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BOLTED MOMENT CONNECTIONS IN COLD-FORMED STEEL BEAM-COLUMN SUB-FRAMES

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2002
ABSTRACT

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Title of thesis: Bolted moment connections in cold-formed steel beam-column sub-frames

Scope of work

This thesis reports the findings of a two-year research project in which an experimental investigation and a numerical study on the structural performance of bolted moment connections in cold-formed steel beam-column sub-frames were performed. The objective of the research project is to demonstrate the high structural performance of bolted moment connections between cold-formed steel sections in practical framing. A total of twenty-five tests with different connection configurations in both internal and external beam-column sub-frames subject to lateral and gravity loads were carried out. Advanced finite element analysis method was also proposed to predict the structural behaviors of the beam-column sub-frames with semi-rigid connections.
Experimental investigation

A total of twenty-five full-scale tests on beam-column sub-frames with bolted moment connections of different configurations were carried out under both lateral and gravity loads. Four different modes of connection failure were identified among the tests:

- BFcsw: Bearing failure in section web around bolt hole;
- FFcs: Flexural failure of connected cold-formed steel section;
- FFgp: Flexural failure of connected gusset plate;
- LTBgp: Lateral torsional buckling of gusset plate.

The moment resistances of the proposed connection configurations were found to range from 62% to 97% of the moment capacities of the connected sections. Among the four failure modes, the flexural failure of connected cold-formed steel section was considered to be the most favourable as over 85% of the moment capacity of the connected section was readily mobilized in the connection.
Numerical study

After data analysis on the test results to generate the moment joint-rotation curves of bolted moment connections, the non-linear finite element analysis program GMNAF was used to model the overall lateral load-deflection curves of the beam-column sub-frames incorporating both geometrical non-linearity and connection flexibility. An incremental-iterative Newton-Raphson procedure was adopted for solution. All of beam-column sub-frames which were failed by flexural failure of connected cold-formed steel section were examined.

The comparison of the numerical results with the test results showed that the finite element models with semi-rigid connections was able to predict correctly the overall behavior of the beam-column sub-frames up to failure. A semi-empirical design rule is also proposed for the connection configurations to generate the moment joint-rotation curves of the connections; the rotational stiffness of the connections were calculated by summation of the flexibilities of individual components.

Conclusions

Based on the findings of the experimental investigation and the numerical study, it is concluded that bolted moment connections between cold-formed steel sections may
be readily achieved using the proposed connection configurations. The bolted moment connections are demonstrated to be effective in transmitting moment between the connected sections under both lateral and gravity loads, and thus enabling effective moment framing in cold-formed steel structures. The proposed non-linear finite element analysis with semi-rigid joints may be used to predict the overall behavior of cold-formed steel beam-column sub-frames with bolted moment connections correctly up to failure. Engineers are encouraged to build short to medium span cold-formed steel portal frames with bolted moment connections for improved buildability when compared with timber or reinforced concrete frames.
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Appendix B

Published technical papers in international conferences

B.1 Chung KF, Chan SL, Wong MF and Yu WK. Structural behaviour of cold-formed steel portal frames with lipped C sections. Proceedings of the Sixth International Conference on Steel and Space Structures, Singapore, September 1999, pp321-328.


CHAPTER ONE

INTRODUCTION

1.1 Cold-formed steel building construction

Cold-formed steel sections are light-weight materials and suitable for building construction owing to their high structural performance. Conventionally, they are used as purlins and side rails in the building envelopes of industrial buildings. The most common sections are lipped C-sections and lipped Z-sections, and the thickness typically ranges from 1.2 mm to 3.2 mm. Common yield strengths are 280 N/mm² and 350 N/mm². In general, cold-formed steel building products such as purlins are developed through extensive tests in order to achieve high structural economy per unit steel. For general applications of cold-formed steel sections, there are a number of codes of practice available in the literature together with complementary design guides and worked examples.

Since 1990, there is a growing trend to use cold-formed steel sections as primary structural members in building construction, such as low to medium rise residential houses and portal frames of modest span. In order to extend the usage of cold-formed
steel sections in building construction, and it is highly desirable to develop efficient moment connections in cold-formed steel sections for efficiency structural framing.

1.2 Bolted moment connections in practical building construction

Consider a typical medium-rise moment framed building under lateral load and gravity load, as shown in Figures 1.1 and 1.2 respectively. It is shown that in both cases, the maximum moment always occurs at the beam-column connections. Consequently, it is important to examine the local structural performances of these beam-column connections in order to understand the global structural behaviour of cold-formed steel structures with moment connections.

It should be noted that for the moment framed building under lateral load, there are two types of member configurations, namely, internal beam-column sub-frame and external beam-column sub-frame. Consequently, it is desirable to examine the structural behaviour of both internal and external beam-column sub-frames under lateral load. The general layout of the beam-column sub-frame tests under lateral load is illustrated in Figure 1.3. Both the internal and the external beam-column sub-frames under lateral loads are statically determine structures. For the moment framed building under gravity load, however, only one beam-column sub-frame test is
possible; the general layout of the beam-column sub-frame under gravity load is illustrated in Figure 1.4. The beam-column sub-frame under gravity load is a statically determine structure.

1.3 Basic configuration of bolted moment connections between cold-formed steel sections

In order to enable moment framing for cold-formed steel sections in building construction, a connection configuration for two lipped C sections back-to-back is proposed as follows and illustrated in Figure 1.5:

- All structural members such as beams and columns are formed with two lipped C-sections back-to-back with interconnections at regular intervals.
- Moment connections between beams and columns are formed with hot-rolled steel gusset plates. In general, only the column members are continuous over the connections.
- Only the webs of lipped C-sections are bolted onto gusset plates for ease of buildability.
- Four bolts per member are used as a minimum configuration.
It should be noted that all bolts are 16mm in diameter, they are all installed with a torque of 50Nm.

The connection details are rationalized after considering ease of fabrication and installation. In general, the proposed moment connections are not able to develop full moment capacity of the connected sections due to discontinuity of load paths along section flanges in the sections.

1.4 Scope of work

The main objectives of the research project are:

- To establish the use of bolted moment connections in cold-formed steel structures for general applications.

- To demonstrate the high structural performance of these bolted moment connections in terms of both strength and stiffness between cold-formed steel sections in practical framing.
- To establish design and analysis methods for cold-formed steel structures with bolted moment connections.

This thesis reports the findings of a two-year research project in which an extensive experimental investigation and an advanced numerical study on the structural performance of cold-formed steel beam-column sub-frames with bolted moment connections were performed. The research project may be divided into the following parts of investigations:

**Part I  Review of literature**

A review on current design rules on bolted moment connections between cold-formed steel sections was carried out. Furthermore, research projects on physical tests and numerical study on the structural behaviour of cold-formed steel structures with bolted moment connections were also examined.

**Part II  Experimental investigation**

An extensive experimental investigation on bolted moment connections between cold-formed steel sections was carried out with two lipped C-
sections back-to-back according to the proposed basic configuration. The key parameters for the structural behaviour of bolted moment connections are:

- Arrangement of bolts or bolt pitches
- Shape of gusset plates
- Relative steel grade and dimensions of cold-formed steel sections to hot rolled steel gusset plates.

Two test series with a total of sixteen internal and external beam-column sub-frames covering a wide range of connection configurations were examined under lateral loads. Moreover, a total of nine internal beam-column sub-frames with various connection configurations were tested under gravity loads.

**Part III Numerical study**

Based on the test results of the cold-formed steel beam-column sub-frames, the measured rotational characteristics of bolted moment connections were adopted to investigate the overall structural behaviour of the sub-frames through non-linear analysis with semi-rigid connections. The measured
moment-rotation curves of the bolted connections were inputted into a non-linear analysis software as local connection characteristics to generate the overall lateral deflection history of the beam-column sub-frames for comparison with test data.

1.5 Layout out of thesis

Chapter 2 Literature review

A detailed review on the technical literature of the structural behaviour and design of cold-formed steel connections is presented. Relevant design and analysis methods incorporating semi-rigid connections are also described.

Chapter 3 Experimental investigation of beam-column sub-frames under lateral load - Test Series I

A test series with a total of eight beam-column sub-frames covering three basic connection configurations were carried out under lateral loads. Both the test program and the test results are fully presented. Details of the proposed connection configurations together with parameters of interest are fully presented. Moreover, both the test
program and the test results are also described in details. Among the eight tests, three different modes of failure are observed. The proposed connections are shown to be structurally efficient and the moment resistances of the connections attain typically at least 70% of the moment capacities of the connected sections.

Chapter 4  Experimental investigation of beam-column sub-frames under lateral load - Test Series II

In order to improve the structural behaviour of the proposed connections, another test series with a total of eight beam-column sub-frame tests covering three engineered connection configurations were also carried out under lateral loads. Details of the proposed engineered connection configurations together with parameters of interest are fully presented. It is found that among the eight tests, two different modes of failure are observed; the test results are described in details. Moreover, comparison with the test results obtained in the first test series is also presented. The moment resistances of the proposed engineered connections are found to attain typically 85% of the moment capacity of the connected sections.
**Chapter 5**  
*Experimental investigation of beam-column sub-frames under gravity load - Test Series III*

In order to confirm the structural behaviour of the proposed connection configurations in beam-column sub-frames under gravity loads, another test series with a total of nine beam-column sub-frame tests were carried out. The connection configurations together with the parameters of interest are fully described. Among the nine tests, two different modes of failure are observed. The moment resistances of the proposed connections are found to range from 77% to 99% of the moment capacities of the connected sections, confirming the structural efficiency of the proposed connection configurations.

**Chapter 6**  
*Non-linear analysis of beam-column sub-frames with semi-rigid connections*

After data analysis of the experimental investigation, the non-linear finite element analysis program ‘GMNAF’ was used to model the overall load-deflection curves of the beam-column sub-frames based on the local moment-joint rotation characteristics obtained from tests. Comparison of the numerical results with the test results is fully
presented and discussed. A rationalized design rule for the stiffness of
the proposed connection configurations is also proposed to predict the
moment joint-rotation characteristic of the connections through
summation of flexibilities of individual components of the connections.

Chapter 7 Conclusions

The findings of both the experimental investigation and the numerical
study are fully discussed. The moment resistances of the proposed
connection configurations are presented as design data for practical
design of bolted moment connections between cold-formed steel
sections. Consequently, it is demonstrated that through rational design
and construction, effective moment connections between cold-formed
steel sections may be readily achieved.

1.6 List of publications

A total of five conference papers and two journal papers were published and
submitted for publication during the research project. Details of the papers are
listed as follows:
Refereed Conference Papers


4. Wong MF and Chung KF. Experimental investigation on bolted moment connections in beam-column sub-frames - Comparative study.


International Journal Papers


A copy of each paper is provided in Appendix A for easy reference.
Figure 1.1 Typical beam-column frames under lateral loads.
Figure 1.2 Typical beam-column framing under gravity loads.
a) Internal beam-column sub-frames

b) External beam-column sub-frames

Figure 1.3 General layout of beam-column sub-frame tests.
Figure 1.4 General layout of beam-column sub-frame tests under gravity loads.
For interconnection, washer as spacer and thickness of gap equal to thickness of connected gusset plate

SECTION OF X-X

Figure 1.5 General layout of beam-column sub-frame tests connection configuration.
CHAPTER TWO

LITERATURE REVIEW

2.1 Background

2.1.1 Introduction to the use of cold-formed steel structures

Cold-formed steel sections are light-weight materials. The most common sections are lipped C and Z sections, and the thickness typically ranges from 1.2 mm to 3.2 mm. The yield strengths typically range from 280 N/mm² to 350 N/mm². Moreover, there are a whole range of variants of these basic shapes, including sections with single and double lips, and sections with internal stiffeners. Owing to its high buildability characteristic, cold-formed steel sections are suitable for many applications as follows:

- In plane and space truss construction, lipped C sections or tubular sections are used as structural members such as top and bottom chords, and diagonals.

- In roof and wall construction, C and Z sections are commonly used as purlins and side rails which support profiled steel sheeting to form building envelopes.

- In low-rise building construction, lipped C sections back-to-back are commonly used with interconnections at regular intervals to form structural members, such as beam and column members.
Moreover, cold-formed steel sections are also commonly used as secondary structural frames, floor bearers and joists, and steel decking for composite construction.

Both bolts and self-drilling self-tapping screws are common fasteners in cold-formed steel construction while welding is seldom used due to the thinness of cold-formed steel sections and also the presence of galvanised coatings.

2.1.2 Stability of cold-formed steel structures

Cold-formed steel sections are thin-walled structures, and the primary consideration on their structural behaviour is stability. Due to the thinness of cold-formed steel sections, local buckling is always critical when the sections with high width-to-thickness ratios are under compression. In order to overcome this problem, edge stiffeners or intermediate stiffeners of sufficient sizes may be provided to the cross-sections of cold-formed steel sections. Furthermore, as they are weak in torsion, torsional flexural buckling in columns and lateral torsional buckling in beams may be critical. There are a number of codes of practice \cite{1-6} on the design of cold-formed steel structures together with complementary design guides and worked examples\cite{7-10} to assist practising engineers.
2.1.3 Reviews of standards and codes on cold-formed steel bolted connections

Most of the codified design rules are only applicable in assessing the load carrying capacities of individual fasteners such as bolts and screws rather than the structural performance of connections between cold-formed steel sections. While it is important to assess the load carrying capacity of each fastener, it is also necessary to examine the structural behaviour of the connectors such as web cleats and gusset plates, and also of the connected parts of cold-formed steel sections under highly localised forces and bending moments. In general, there is a lack of design information of simple connections and moment connections between cold-formed steel sections. The minimum configuration of bolted connections with two bolts per member is commonly regarded as simple (or shear) connections.

Moreover, most of the modern codes and standards do not consider connections between cold-formed steel sections to be moment resisting, and thus many new cold-formed steel products are developed from experimental testing rather than from design methods due to the lack of relevant design recommendations.
2.2 Experimental works

Many researchers have conducted experimental investigation on the structural behaviour of connections between cold-formed steel sections. Much research work on the design development of simple and moment connections has been reported in literature, such as shear resisting connections, purlin-rafter connections, column-base connections and beam-column moment connections.

2.2.1 Simple connections

In order to improve the buildability of cold-formed steel sections, it is highly desirable to use folded cold-formed steel strips as web cleats rather than hot rolled angles to form shear resisting connections. This will allow greater compatibility in materials and connection methods, and both the fabrication and the installation processes will also be simplified. Furthermore, the use of self-drilling self-tapping screws does not require pre-drilled holes and thus, the problem of construction tolerance on site may be reduced significantly.

An experimental investigation on the structural performance of shear resisting connections between cold-formed steel sections are reported in the literature\textsuperscript{[22]} where web cleats of cold-formed steel strips are used to attach beams to supporting beams.
and columns. A total of 24 connection tests with four different connection configurations were carried out; three modes of failure were identified:

- failure of fasteners
- shear buckling of cold-formed steel web cleats or webs of supported beams, and
- lateral torsional buckling of cold-formed steel web cleats

The structural performance of the cold-formed steel web cleats in four different connection configurations is examined in detail. It is demonstrated that typical shear resistances of the proposed connections with cold-formed steel web cleats range from 9 kN to 20 kN while the end deflection of the cold-formed steel web cleat is always less than 5 mm, which may be considered to be acceptable in building construction. A set of design rules is formulated in accordance with both BS5950: Part 5 and Eurocode 3: Part 1.3 after calibration against test data. The rationalised usage of cold-formed steel web cleats allows simple and effective connections to be formed between cold-formed steel sections leading to improved buildability.
2.2.2 Moment connections: Beam-to-beam connections

The most common moment connection among cold formed steel members is the purlin-rafter connection in roof construction. Bolted moment connections with high strength and stiffness are essential in safe and economical design and construction of purlin systems, and thus there are many research works reported in the literature on the development of purlin-rafter connections in modern roof systems. They are basically beam-to-beam connections with different degrees of continuity to reduce mid-span moment and deflection. It should be noted that all these configurations are only suitable for roof and wall construction. For details on the structural behaviour of cold-formed steel purlin systems, refer to a survey reported by Trahair.[27]

2.2.3 Moment connections: Beam-to-column connections

Beam-to-column connections may be commonly found in beam-column frames used in low-rise buildings such as portal frames. Conventionally, fabricated or hot-rolled I sections are used as the primary members of the portal frames with welded or bolted connections, depending on spanning requirements and architectural layouts. While there are over hundreds of technical papers reporting experimental and theoretical investigations on portal frames in the literature, over 95% of them are related to hot-rolled steel portal frames covering all aspects of behaviour, design and construction
such as different types of structural analyses, structural forms, effective length assessment, restraining stiffnesses and connection details, and also lateral and shear restraints offered by sheetings. However, cold-formed steel portal frames have also been investigated by a number of researchers over the past two decades, and a number of selected research work for cold-formed steel portal frames with moment connections is summarised as follows.

a) Cold-formed steel portal frames with single sections

The first systematic study of cold-formed steel portal frames with single C-sections and Z-sections was reported by Hancock and Baigent\(^{11,12}\) between 1980 and 1985. A number of pitched-roof portal frames of 3 m height and 6 m span were tested under three different loading conditions and local plastic buckling in the sections were found at the member connections. It was suggested\(^{11}\) that the load carrying capacities of the portal frames depended primarily on the connection details between members. Moreover, due to low torsional rigidities in single cold-formed steel sections, lateral deformations were found to be significantly larger than those found in portal frames with equivalent box sections. Moreover, finite strip models with cross section distortion capability were established to predict the stress distributions of the sections. Comparing the stresses between the two types of portal frames with C-sections and Z-
sections, it was interesting to note that the bi-moment distribution in the Z-portals has a similar pattern to the minor axis moment distribution in the C-portals\textsuperscript{[10]}.

There was another research programme reported in the literature by Mills\textsuperscript{[31]} where a number of knee joints with different configurations were tested to failure. These knee joints were parts of short span cold-formed steel portal frames widely used as agricultural and industrial sheds in Australia. It was shown that most knee joints with conventional configurations using knee braces and bolted end plates were not able to develop full moment resistances of the connected sections, and they failed at moments significantly lower than design values in accordance with current design practice in Australia. It was suggested that by the use of multiple screw knee joints, a connection configuration similar to the multiple nail knee joints commonly used in timber portal frames, the stress concentration problem around connected section web during load transfer was eliminated successfully. The multiple screw knee joints were demonstrated to be able to develop full moment resistances of the connected sections. The use of self-drilling self-tapping screws was considered to be highly desirable as it was faster to fabricate and easily constructed on site.
b) Cold-formed steel portal frames with double sections back-to-back

In 1986, a building system with portal frames composed of two lipped C-sections back-to-back was reported by Kirk\textsuperscript{[13]}. With longitudinal ribs in the section webs and profiled gusset plates, mechanical enhancement was devised to form beam-column connections of high rigidities, and thus greatly improved the strength and the stiffness of the portal frames against both gravity and lateral loads. In order to mobilise the enhanced moment resistances of the connections readily, tight fabrication and installation tolerances were required; however, this may not be easily achieved on sites due to relatively large construction tolerances.

In order to develop cold-formed steel portal frames using generic cold-formed steel sections with simple connection configurations and high structural performance, a total of 9 component tests and 7 portal frame tests with two lipped C-sections back-to-back were conducted by Chung and Lau in 1997\textsuperscript{[17]}. The portal frames were 1.4 m x 1.4 m (plan area) x 2 m (height) in dimensions and subjected to lateral point loads; they were developed as hoarding frames for protection to the public around construction sites. A number of connection configurations with gusset plates of both hot rolled steel and cold-formed steel were proposed to form bolted moment connections to accommodate members in practical orientations. Only the webs of
lipped C sections were connected with bolts; the section flanges are not connected for ease of construction.

Among all the tests, four modes of failure were identified as follows:

- BFcsw  \textit{bearing failure in section web around bolt hole}
- LTBgp \textit{lateral torsional buckling of gusset plate}
- FFcs \textit{flexural failure of connected member, and}
- CBcols \textit{combined compression and bending failure of column member}

It was shown that bearing failure was a ductile mode with large deformation capacity, and other failure modes might creep in to cause sudden collapse. While lateral torsional buckling of gusset plates caused pre-mature failure of the connections at low applied load, flexural failure was more desirable as over 80\% of the moment capacity of the connected members might be safely mobilised at the connections. In the absence of effective torsional restraint at the column ends, the column members might fail in combined compression and bending.

Among sixteen component and system tests, the moment resistance of bolted moment connections with four bolts per member was found to lie between 42\% and 84\% of
the moment capacities of the connected members. Thus, it was demonstrated that moment connections among cold-formed steel members were structurally feasible and economical through rational design. Relevant design rules on bolted moment connections specifically developed for cold-formed steel portal frames were also reported \(^{(18,19)}\). Consequently, it was demonstrated that bolted moment connections between cold-formed steel members and thus cold-formed steel portal frames were structurally feasible and economical through rational design. However, lateral deflection is found to be excessive in some tests. A preliminary set of design rules against lateral torsional buckling of gusset plates in bolted moment connections between cold-formed steel members was also reported by Chung and Shi. \(^{(19)}\)

c) Cold-formed steel portal frames with other sections

Other similar portal frames using cold-formed hollow flange beams (HFB: 1995\(^{(14)}\) & 1999\(^{(15)}\)) and cold-formed rectangular hollow sections (RHS: 1999\(^{(16)}\)) were also reported in the literature. In both HFB and RHS sections, end plates were welded onto the end of the sections which were in turn bolted together to form effective beam-column connections. However, the connection detail was not suitable for cold-formed steel sections with thickness less than 3.2 mm as welding were likely to burn-through the sections.
2.3 Analytical investigations

Steel structures are usually designed by assuming that the connections are either fully rigid or ideally pinned. The first assumption implies that there is no relative rotation between the connected members, so that at any beam-to-column joint the distribution of the moments occurs according to the flexural stiffness of the connected members. Conversely, the assumption of pinned connections implies that the end rotation of members is free to occur, so that the beam end moment is always zero. However, it is recognised that all the connections, despite their apparent rigidity, undergo a certain flexural deformation, and only provide a certain degree of rotational restraint. In other words, all actual beam-to-column joints behave as semi-rigid joints.

2.3.1 Semi-rigid connections

Ahmed and Kirby[32] reported their investigation on the behavior of semi-rigid connections in hot-rolled steel frames where a realistic consideration of joint response taking into account of the rotational flexibility of beam-to-column connections was considered. The semi-rigid connections are usually expressed in terms of their moment-rotation relationship, and an analytical formulation widely adopted by many researchers is presented as follows:
The displacement formulation requires solution of the equilibrium equation:

\[ \{F\} = [K_T] \{\delta\} \]

where

\(\{F\}\) is the load vector,

\([K_T]\) is the tangential stiffness matrix, and

\(\{\delta\}\) is the vector of unknown displacements.

The \([K_T]\) matrix is defined as:

\[ [K_T] = [K_E] + [K_G] + [K_L] \]

The \([K_E]\) matrix is the stiffness matrix of a beam-column member including the effect of connection flexibility at the end of the member. The matrix \([K_G]\) is the geometric stiffness matrix and \([K_L]\) is the large displacement stiffness matrix.

It already has been emphasised that the primary flexibility of a steel beam-to-column joint is its rotational deformation \(\theta\), caused by the in-plane bending moment \(M\).
Therefore, the joint behaviour is substantially represented through the moment versus joint-rotation curve, M-θ, which is the most important input data for the analysis and design of semi-rigid frames. In general, advanced methods of structural analysis require a very accurate modelling of the beam-to-column joint behaviour. They rely on an accurate modelling of the joint behaviour, through linear, bi-linear, multi-linear or curvilinear moment-rotation curves M-θ. On the contrary, the use of linear or bi-linear spring elements modelling the joint behaviour is limited to simplified methods of structural analysis.

Furthermore, different levels of sophistication correspond to the above modelling alternatives depending on the behaviour assumed for the spring elements which can be linear, bi-linear, multi-linear or curvi-linear. Moreover, it is recognised that there is a very important interaction between the joint modelling, the method of global structural analysis and the joint classification.

It should be noted that if the rigid-plastic method of global structural analysis is used, only the joint flexural resistance is of concern and the joint modelling can be based on a bi-linear rigid-plastic moment-rotation curve. In addition, the joint classification
based on the strength criterion has to be considered by distinguishing nominally pinned, partial strength and full-strength beam-to-column joints.

2.3.2 Moment joint-rotation characteristic

In general, it is widely accepted that the most reliable and accurate information the structural behaviour of bolted connections is obtained from experimental investigation. However, this method is expensive and time-consuming for construction practice, and thus it is only suitable for research works. Consequently, it is always desirable to use analytical methods to predict the structural behaviour of bolted moment connections. Many researchers have studied semi-rigid connections in beam-column frames or column base connections in hot-rolled steel structures, and a large number of design methods and recommendations may be found in the literature.

The methods for predicting the beam-to-column joint behaviour can be divided into five different categories as follows:

- empirical models;
- analytical models;
- mechanical models;
• finite element analysis;

• experimental testing.

2.3.3 Empirical models

Empirical models are based on the use of empirical formulations relating the parameters involved in the mathematical representation of the empirical formulation of the M - θ curve to the geometrical and mechanical properties of beam-to-column joints. These empirical formulations can be obtained by means of regression analyses of data which are derived in different ways such as analytical models, mechanical models, parametric analyses developed by means of finite element models, and experimental testing.

Miyashita and Goto\(^{[33]}\) proposed a set of design recommendations for the classification of semi-rigid connections based on the elasto-plastic behaviour of steel moment frames. Some typical sub-assemblages of multi-storey frames were chosen and examined. The classification rule is able to classify the connection quantitatively according to the ultimate moment capacities of connection (\(M_U\)) at ultimate limit state, and also the initial connection stiffness (\(K_i\)) at serviceability limit state. In general, the generalised form of the semi-rigid connection model is given by:
\[ m = \frac{\theta}{(1 + \theta^2)^{3/2}} \]

where

\[ m = \frac{M}{M_c} ; \]
\[ \theta = \frac{\theta_r}{\theta_b} ; \]
\[ \theta_o = \frac{M_u}{K_i} . \]

- \( M \) is the connection moment
- \( M_u \) is the ultimate moment capacity of connection
- \( \theta_r \) is the relative rotation between beam and column
- \( K_i \) is the initial connection stiffness, and
- \( n \) is a shape factor

This equation is widely adopted by researchers as an effective means to establish the moment-joint rotation characteristics of semi-rigid connections. The shape factor, \( n \), is usually adjusted for each connection configuration when comparing with measured or predicted data in order to obtain a good representation of the moment joint-rotation characteristics of the connection.
2.3.4 Analytical models

In order to predict the moment-rotation curves of different connection configuration directly from geometrical and mechanical properties, several authors have applied the basic concepts of elastic structural analysis and limit design to simplified models of various types of beam-column connections. This approach starts with the observation of test behaviour to identify the main deformation sources and the collapse mechanism of the connections. Therefore, on the basis of the experimental evidence, a simplified connection model is assumed for predicting the initial connection stiffness by means of an elastic analysis. In addition, the observed plastic mechanism is modelled to predict the ultimate moment resistance through the balance between internal and external work. The reliability of the results obtained through the assumed models is controlled by verifying the agreement with test data. Finally, the mathematical representation of the M-θ curve is provided using the predicted initial stiffness and ultimate moment capacity. Many researchers have devoted much effort towards predicting the response of the connections directly from their geometrical and mechanical properties.

The main advantage of analytical models is their ability to provide an approximate evaluation of the key parameters describing the moment-rotation curves, without
resort to testing. Curve fitting is required only when the complete moment-rotation curve is desired and, in such a case, is generally limited to the calibration of a shape factor. The reliability of the analytical models is generally investigated by comparison with the available experimental test results. Therefore, the designer must be aware that the analytical models are only accurate for those connections with similar constructional details of the connections confirmed by tests.

2.3.5 Mechanical models

Mechanical models, also namely spring models, are based on the simulation of the connection through a set of rigid and flexible components. The non-linearity of the response is obtained by means of inelastic constitutive laws adopted for the spring elements.

The first difference between analytical and mechanical models is that, in analytical models the joint components are characterised by means of stiffness and resistance values derived from the basic concepts of elastic structural analysis and limit design respectively. Conversely, mechanical models rely also on the modelling of one or more components by means of stiffness and resistance values obtained from empirical relationships.
The second and, probably, the most important difference is that, in analytical models, attention is focused on the prediction of the joint rotational stiffness and the flexural resistance while the curvi-linear modelling of the knee of the M-\( \theta \) curve still requires a curve fitting, even though limited to the calibration of merely a shape factor. Conversely, from the theoretical point of view even in the case of a bi-linear modelling of the single joint components, spring models are able to simulate the knee of the M-\( \theta \) curves through the multi-linear overall behaviour resulting from the progressive yielding of the joint components and, therefore, without resorting to any curve fitting.

Mechanical models have been developed by several researchers with reference to specific connection configurations as well as to the whole connections with the aim of directly providing the response of the connections.

2.3.6 Finite element analysis

The finite element technique seems, in principle, to be the most suitable tool to investigate the response of a connection. Nevertheless, it has to be recognised that, in spite of the continual progress, some of the requirements needed for an accurate
simulation are still today unsatisfied. In fact, the moment-rotation curves represent
the result of a very complex interaction among various components.

As already pointed out up to now, some of the basic mechanisms of these interactions
need to be fully understood. In fact, the analysis of isolated plates has already
reached a high degree of accuracy with the possibility of accounting for the spread of
plasticity, the strain-hardening, the instability effects and the representation of large
strain and/or displacements. However, all other requirements need a level of
refinement not yet attained. As a consequence, on the one hand, the finite element
technique already represents a sufficiently accurate tool for modelling welded beam-
to-column connections, and on the other hand, it is a very sophisticated approach
whose potentiality for modelling bolted connections is still largely unexplored.

2.3.7 Experimental testing

It is generally agreed that experimental tests provide the most accurate information
about the rotational behaviour of beam-to-column connections used to evaluate the
reliability of empirical, analytical, mechanical and finite element models.
However, it should be noted that even though full scale and carefully conducted experimental tests are the most reliable and direct method for understanding the connection behaviour, it has to be recognised that the rotational behaviour of beam-to-column joints in building frames can exhibit some differences with respect to tested specimens. This is mainly due to the interaction between internal actions whose distribution in joints in building frames can be different from that occurring in structural schemes adopted for testing specimens. Furthermore, there is another difference between the joint behaviour in tested specimens and the joint behaviour in building frames as the latter can be affected by the pre-loading due to internal actions occurring during building erection. Further differences are related to the internal actions developed during the building erection, the geometrical imperfections, the manufacturing tolerances, and also the variability of the loading conditions.

Consequently, experimental tests have to be regarded as the most reliable method for predicting the joint behaviour in building frames in practice rather than regarded as the actual joint behaviour. Both the applied moment and the corresponding member and joint rotations are obtained from tests and then analysed to establish the moment joint-rotation characteristics of the connection configurations for subsequent analysis and design.
CHAPTER THREE

EXPERIMENTAL INVESTIGATION OF BEAM-COLUMN SUB-FRAMES UNDER LATERAL LOAD - TEST SERIES I

3.1 Objectives of investigation

A total of eight beam-column sub-frame tests with three different connection configurations were carried out. The objectives of this test series are:

- To establish the structural performance of the proposed bolted moment connections in cold-formed steel beam-column sub-frames, in particular, the moment resistances of the connections.

- To quantify the structural efficiency of the proposed bolted moment connections in terms of the moment capacities of the connected sections.

3.2 Test program

All the test specimens are constructed according to the proposed basic configurations with systematic variations in the connection details, i.e. bolt arrangement (or bolt pitch),
and shape and thickness of gusset plates.

Details of the eight beam-column sub-frames are summarized as follows:

<table>
<thead>
<tr>
<th>Test series</th>
<th>Test specimen</th>
<th>Beam column connection</th>
<th>Gusset plate</th>
<th>No of bolts per member</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Shape</td>
<td>Thickness</td>
</tr>
<tr>
<td>I-1</td>
<td>S090A1</td>
<td>Cross</td>
<td>10 mm</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>S180A1</td>
<td>Cross</td>
<td>10 mm</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>S240A1</td>
<td>Cross</td>
<td>10 mm</td>
<td>4</td>
</tr>
<tr>
<td>I-2</td>
<td>E180C1</td>
<td>Tee</td>
<td>10 mm</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>E240C1</td>
<td>Tee</td>
<td>10 mm</td>
<td>4</td>
</tr>
<tr>
<td>I-3</td>
<td>S090A2</td>
<td>Cross</td>
<td>16 mm</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>S180A2</td>
<td>Cross</td>
<td>16 mm</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>E180C2</td>
<td>Tee</td>
<td>16 mm</td>
<td>4</td>
</tr>
</tbody>
</table>

The connection configurations are specified by the following parameters:

<table>
<thead>
<tr>
<th>Type of beam-column</th>
<th>S</th>
<th>Internal</th>
<th>E</th>
<th>External</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bolt pitch</td>
<td>090</td>
<td>90 mm (0.6 D)</td>
<td>180</td>
<td>180 mm (1.2 D)</td>
</tr>
<tr>
<td></td>
<td>240</td>
<td>240 mm (1.6 D)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Corner of gusset plate</td>
<td>A</td>
<td>Sharp corner</td>
<td>C</td>
<td>Sharp corner</td>
</tr>
<tr>
<td>Thickness of gusset plate</td>
<td>1</td>
<td>10 mm (6.25 t, 5.00 t)</td>
<td>2</td>
<td>16 mm (8.00 t)</td>
</tr>
</tbody>
</table>

For example, for internal beam-column sub-frames, a beam-column connection with a symmetrical cross gusset plate of 10 mm thick using 4 bolts per member at 90 mm bolt pitch is referred as "S090A1".
In this test series, two section sizes are used, namely C15016 and C15020. All the members of the test specimens are double sections which are designated as DS. The design yield strength of the sections is 450 N/mm² which is designated as G450. The bolts are of 16 mm in diameter and of grade 8.8. The nominal section dimensions of the sections are summarized as follows:

<table>
<thead>
<tr>
<th>Designation</th>
<th>Depth (mm)</th>
<th>Flange width (mm)</th>
<th>Lip (mm)</th>
<th>Thickness (mm)</th>
<th>Internal radius (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C15016DS G450</td>
<td>152</td>
<td>64</td>
<td>15</td>
<td>1.6</td>
<td>3.2</td>
</tr>
<tr>
<td>C15020DS G450</td>
<td>150</td>
<td>60</td>
<td>14</td>
<td>2.0</td>
<td>4.0</td>
</tr>
</tbody>
</table>

3.2.1 Test series I-1 - S090A1, S180A1 and S240A1

In order to examine the structural performance of bolted moment connections against various pitches, three internal beam-column sub-frames with different pitches of 90 mm, 180 mm and 240 mm are selected for investigation as shown in Figure 3.1a; the thickness of the gusset plates is 10 mm. Refer to Figure 3.2 for details.

3.2.2 Test series I-2 - E180C1 and E240C1

A similar connection configuration is adopted but the member configuration is changed from the internal beam-column sub-frame to the external beam-column sub-frame as shown in Figure 3.1b. This enables comparison on the structural performance of
similar connection configurations between the internal and the external beam-column sub-frames. Refer to Figure 3.2 for details.

3.2.3 Test series I-3 - S090A2, S180A2 and E180C2

Furthermore, in order to examine the structural performance of bolted moment connections against gusset plate thickness, two internal beam-column sub-frame tests with gusset plates of 10 mm and 16 mm thick were carried. One additional external beam-column sub-frame with a gusset plate of 16 mm thick was also carried out for comparison. Refer to Figures 3.2 for details.

3.3 Test instrumentation and procedure

The general layout of the test set-up is shown in Figure 3.3. During the tests, the applied loads and the displacements of the test specimens were measured using load cells and transducers as shown in Figure 3.3. Any out-of-plane deformation was prohibited by two sets of restraining rollers that were installed in front of and behind the beam members at two sides. The tests would be terminated when large deformation occurred in the tests, or section failure with yielding or buckling was observed. In most cases, a pre-load of 2 kN was applied before the tests to ensure that all bolts were in contact with the section webs of connected members despite all the bolt holes were
'perfect-fitted' to 16 mm diameter bolts.

3.4 Failure modes

Three modes of failure on the connections of the beam-column sub-frames are identified among the eight tests:

- **BFcsw**  
  *Bearing failure in section web around bolt hole.*

- **FFcs**  
  *Flexural failure of connected cold-formed steel section.*

- **FFgp**  
  *Flexural failure of connected gusset plate.*

3.4.1 Bearing failure in section web around bolt hole (BFcsw)

It is a ductile failure mode where the load carrying capacity of the test specimen does not drop and the deformation is generally large. For those connections failed by BFcsw, they are not recommended to use as the connections are found to give low moment capacities with large members rotation and deformation. Refer to Figure 3.4 for typical failure.

3.4.2 Flexural failure of connected cold-formed steel section (FFcs)

This failure mode only occurs when the applied moment approaches the moment
resistances of the sections. Once the failure mode is apparent, the test specimen unloads quickly. The flexural failure of the connected cold-formed steel section is a desirable and favourable failure mode because high moment resistance in the connections may be readily mobilized. Refer to Figure 3.5 for typical failure.

3.4.3 Flexural failure of connected gusset plate (FFgp)

This failure mode always occurs in the presence of defective cracks in the corner edges of hot rolled steel gusset plates. It is believed to be caused by poor workmanship in plate-cutting. The associated lateral deformation of the test specimens is generally large. It should be eliminated with proper control of plate-cutting or a round corner should be used. Refer to Figure 3.6 for typical failure.

3.5 Test results

Table 3.1 summarizes the results of the coupon tests of both cold-formed steel sections and gusset plates of all the test specimens. Both the load-deflection curves and the moment-rotation curves of the eight test specimens are plotted in Figures 3.7 to 3.14. All the load-deflection curves and the moment rotation curves are grouped together for direct comparison in Figures 3.15 and 3.16 for internal and external beam-column sub-frames respectively. The rotation calculations for both the internal and the external
beam-column sub-frame based on measured displacements at specific locations of the
test specimens are shown in Figure 3.3. The results of the test series are summarized in
Table 3.2.

In order to assess the effectiveness of a connection moment, a moment resistance ratio
($\Psi$) is established which is defined as follows:

$$\Psi = \frac{\text{Measured moment resistance of connection}}{\text{Measured moment capacity of connected member}}$$

(3.1)

For flexible test specimens, the moment resistances of the connections are limited to the
applied moment at a rotation of 0.05 radian in order to avoid excessive deformation of
the connected members.

The measured moment capacities of C15016DS G450 and C15020DS G450 are 16.95
kNm and 21.36 kNm respectively after normalized to design thickness and yield
strength; the moment capacities are measured from four-point load tests. The
normalised moment resistances of the test specimens are evaluated at the failure
positions of the connections. In the present analysis, the moment resistances are first
evaluated at the centreline of the connections, and different level arm coefficients are
then applied according to the associated failure modes to give the moment resistances of the connections at the failure positions. Refer to Figure 3.17 for details of the level arm coefficients.

3.5.1 Test Series I-1 —Tests S090A1, S180A1 and S240A1

In this test series, large deflections and rotations of the test specimens were observed during load application. The normalised moment resistances of the bolted moment connections in tests S090A1, S180A1 and S240A1 at the failure positions of the beam-column connections are found to be 6.18 kNm, 14.68 kNm and 12.47 kNm respectively. The initial stiffness of the connections in tests S090A1, S180A1 and S240A1 are 750 kNm/rad., 1000 kNm/rad. and 1500 kNm/rad. respectively. The rotation capacities of the beam-column connections in tests S090A1, S180A1 and S240A1 obtained from the moment-rotation curves are 0.040 radian, 0.040 radian and 0.010 radian respectively.

During the tests, there was no distinctive out-of-plane deformation in the test specimens. After the test, all the members of the test specimens were disassembled from the connections for inspection.
In test *S090A1*, significant bearing deformation was observed in the bolt holes of the beam members due to high moment acting at small lever arms. For both tests *S180A1* and *S240A1*, gross bending deformation was apparent in the hot-rolled steel gusset plates and a crack was found at the corner of each of the gusset plates.

The moment resistance ratios, $\Psi$, at the failure positions of the connections in tests *S090A1*, *S180A1* and *S240A1* are found to be 0.36, 0.69 and 0.74 respectively.

### 3.5.2 Test Series I-2 – Tests E180C1 and E240C1

In this test series, large deflections and rotations of the test specimens were observed during load application. The normalised moment resistances of the bolted moment connections in tests *E180C1* and *E240C1* at the failure positions of the beam-column connections are found to be 17.91 kNm and 18.77 kNm respectively. The initial stiffnesses of the connections in tests *E180C1* and *E240C1* are 1300 kNm/rad. and 1600 kNm/rad. respectively. The rotation capacities of the beam-column connections in tests *E180C1* and *E240C1* obtained from the moment-rotation curves were 0.030 radian and 0.041 radian respectively.

During the tests, there was no distinctive out-of-plane deformation in the test specimens.

After the tests, all the members of the sub-frames were disassembled from the
connections for inspection. Gross bending deformation was apparent in the hot-rolled steel gusset plates of both tests. A crack was found at the corner of each of the gusset plates in both test specimens.

The moment resistance ratios, $\Psi$, at the failure positions of the connections in tests $E180C1$ and $E240C1$ are found to be 0.84 and 0.88 respectively.

3.5.3 Test series I-3 – Tests S090A2, S180A2 and E180C2

In this test series, large deflections and rotations were observed in test specimen $S090A2$ during load application while both tests $S180A2$ and $E180C2$ exhibited only little deformation. The normalised moment resistances of the bolted moment connection in tests $S090A2$, $S180A2$ and $E180C2$ at the failure positions of the beam-column connections were found to be 12.13 kNm, 19.72 kNm and 20.71 kNm respectively. The initial stiffnesses of the connections in tests $S090A2$, $S180A2$ and $E180C2$ are 750 kNm/rad., 1600 kNm/rad. and 1600 kNm/rad. respectively. Due to excessive deformation of the test specimen, the moment resistance of the bolted moment connection in test $S090A2$ was restricted to 12.00 kNm at a rotation of 0.05 radian. The rotation capacities of tests $S180A2$ and $E180C2$ are found to be 0.031 radian and 0.012 radian.
During the tests, there was no out-of-plane deformation in the test specimens. After the tests, all the members of the sub-frames were dissembled from the connections for inspection.

In test S090A2, bearing failure was observed at the bolt holes in the cold-formed steel section due to high bending moment acting at small level arms. Both test specimens S180A2 and E180C2 failed suddenly under the applied forces of 20.94 kN and 11.05 kN respectively, with local buckling in the section flanges of the connected sections.

The moment resistance ratios, $\Psi$, at the failure positions of the connections in tests S090A2, S180A2 and E180C2 are found to be 0.57, 0.92 and 0.97 respectively.

3.6 Comparisons

3.6.1 Efficient connections

In test S180A2, the connection failed under large moment at small rotation when compared with test S180A1. The moment resistance of the connection in test S180A2 was found to be 92% of the moment capacity of the connected section, being the most stiff and strong connection among all the internal beam-column sub-frame tests.
3.6.2 Optimal bolt pitch

For internal beam-column sub-frames with 10 mm thick gusset plates, the change in the bolt pitch from 90 mm to 180 mm and then to 240 mm is shown to increase the value of the moment resistance ratio of the proposed connection configuration from 0.36, to 0.69 and then to 0.74, as shown in tests S090A1, S180A1 and S240A1.

Moreover, for external beam-column sub-frames with 10 mm thick gusset plates, the increase in the bolt pitch from 180 mm to 240 mm is also shown to increase the value of the moment resistance ratio of the proposed connection configuration from 0.84 to 0.88 as shown in tests E180C1 and E240C1.

The result shows that connections with large bolt pitch always give high moment resistances. Moreover, with an increase in the moment resistances, flexural failure in the gusset plates rather than bearing failure in the connected section web becomes critical in the connections.

As a bolt pitch of 90 mm is found to give low moment resistance with large connection rotation and member deformation, it is thus not recommended to be used in moment connections.
3.6.3 Thickness of gusset plate

By increasing the thickness of the gusset plates from 10 mm to 16 mm, the moment resistance ratio of the proposed connection configuration is found to be increased from:

- 0.36 to 0.57 as shown in tests S090A1 and S090A2,
- 0.69 to 0.92 as shown in tests S180A1 and S180A2, and
- 0.84 to 0.97 as shown in tests E180C1 and E180C2.

This shows that thick gusset plates always give high moment resistances than thin gusset plates. Moreover, with an increase in the moment resistances of the gusset plates, flexural failure in the connected cold-formed steel sections rather than flexural failure of the gusset plates becomes critical in the connections. The maximum moment resistance of the proposed connection configuration is found to be over 90% of the moment capacities of the connected sections, demonstrating that the proposed connection configuration is effective in transferring moment across the connected members.
3.7 Conclusions

A total of eight beam-column sub-frame tests were carried out. Among the eight tests, three different modes of failure were observed. In general, the connections were not able to develop full moment capacity of the connected sections due to discontinuity of the load path along section flanges in the sections. The moment resistances of the proposed connections were found to range from 36% to 97% of the moment capacity of the connected sections.

For C15016DS and C15020DS sections, the following parameters are considered to be effective in forming bolted moment connections of high structural efficiency:

a) 4 bolts per member;

b) a bolt pitch of 180 mm;

c) a hot-rolled steel gusset plate with a thickness larger than or equal to 10 mm.

The proposed connections are expected to be structurally efficient with moment resistances attaining at least 70% of the moment capacities of the connected sections.
Figure 3.1 General layout of beam-column sub-frames tests.
All Bolts are 16mm in diameter (M8.8)

10mm thick gusset plate

Figure 3.2a: Connection detail of test specimen S090A1.
All Bolts are 16mm in diameter. (M8.8)

10mm thick gusset plate

Figure 3.2b: Connection detail of test specimen S180A1.
All bolts are 16mm in diameter. (M8.8)

10mm thick gusset plate

Figure 3.2c: Connection detail of test specimen S240A1.
All Bolts are 16mm in diameter. (M8.8)

10mm thick gusset plate

Figure 3.2d: Connection detail of test specimen E180C1.
All bolts are 16mm in diameter. (M8.8)

10mm thick gusset plate

Figure 3.2e: Connection detail of test specimen E240C1.
All Bolts are 16mm in diameter (M8.8)

16mm thick gusset plate

Figure 3.2f: Connection detail of test specimen S090A2.
All Bolts are 16mm in diameter. (M8.8)

16mm thick gusset plate

Figure 3.2g: Connection detail of test specimen S180A2.
All Bolts are 16mm in diameter. (M8.8)

16mm thick gusset plate

Figure 3.2h: Connection detail of test specimen E180C2.
Figure 3.3a  Overall view of test setup.

Figure 3.4  BFcsw: Bearing failure in section web around bolt hole.
Figure 3.3b  Details of rotation calculation for internal beam-column sub-frames.
Figure 3.3c  Details of rotation calculation for external beam-column sub-frames.
Figure 3.5a  FFcs: Flexural failure of connected cold-formed steel section in internal beam-column sub-frame.

Figure 3.5b  FFcs: Flexural failure of connected cold-formed steel sections in external beam-column sub-frame.
Figure 3.5c  Local buckling at section flange under compression.

Figure 3.6  Flexural failure of connected gusset plate.
Figure 3.7a Load deflection curve of test specimen S090A1.

Figure 3.7b Moment-joint rotation curves of test specimen S090A1.
Figure 3.8a Load deflection curve of test specimen S180A1.

Figure 3.8b Moment-joint rotation curves of test specimen S180A1.
Figure 3.9a Load deflection curve of test specimen S240A1.

Figure 3.9b Moment-joint rotation curves of test specimen S240A1.
Figure 3.10a  Load deflection curve of test specimen E180C1.

Figure 3.10b  Moment-joint rotation curves of test specimen E180C1.
Figure 3.11a  Load deflection curve of test specimen E240C1.

Figure 3.11b  Moment-joint rotation curves of test specimen E240C1.
Figure 3.12a  Load deflection curve of test specimen S090A2.

Figure 3.12b  Moment-joint rotation curves of test specimen S090A2.
Figure 3.13a  Load deflection curve of test specimen S180A2.

Figure 3.13b  Moment-joint rotation curves of test specimen S180A2.
Figure 3.14a  Load deflection curve of test specimen E180C2.

Figure 3.14b  Moment-joint rotation curves of test specimen E180C2.
Figure 3.15a Load deflection curves of internal beam-column sub-frames.

Figure 3.15b Load deflection curves of external beam-column sub-frames.
Figure 3.16a  Moment rotation curves of internal beam-column sub-frame tests.

Figure 3.16b  Moment rotation curves of external beam-column sub-frame tests.
Mode of failure | Level arm coefficient
--- | ---
FFcs | 1710/2000 = 0.86
FFgp | 1925/2000 = 0.96
BFcsw (beam) | 1800/2000 = 0.90
BFcsw (column) | 2000/2000 = 1.00

Figure 3.17  Level arm coefficient in lateral loading tests.
Table 3.1 Summary of tensile test results

<table>
<thead>
<tr>
<th>Designation</th>
<th>Test specimens</th>
<th>Dimension of specimens</th>
<th>Maximum applied force (kN)</th>
<th>Elongation (%)</th>
<th>$\sigma_{0.2%}$ (N/mm$^2$)</th>
<th>$0.84\ U_s$ (N/mm$^2$)</th>
<th>$U_s$ (N/mm$^2$)</th>
<th>Young's modulus (kN/mm$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C15016D S G450</td>
<td>S090A1</td>
<td>10.03  1.62</td>
<td>9.11</td>
<td>20.0</td>
<td>520</td>
<td>471</td>
<td>561</td>
<td>201</td>
</tr>
<tr>
<td>C15020D S G450</td>
<td>S180A1</td>
<td>9.98   1.98</td>
<td>10.59</td>
<td>16.0</td>
<td>490</td>
<td>451</td>
<td>537</td>
<td>215</td>
</tr>
<tr>
<td>C15016D S G450</td>
<td>S240A1</td>
<td>14.85  1.63</td>
<td>13.40</td>
<td>10.0</td>
<td>515</td>
<td>465</td>
<td>554</td>
<td>200</td>
</tr>
<tr>
<td>C15020D S G450</td>
<td>E180C1</td>
<td>14.97  1.95</td>
<td>15.79</td>
<td>20.0</td>
<td>490</td>
<td>454</td>
<td>541</td>
<td>200</td>
</tr>
<tr>
<td>C15020D S G450</td>
<td>E240C1</td>
<td>14.71  1.94</td>
<td>15.49</td>
<td>16.7</td>
<td>500</td>
<td>456</td>
<td>543</td>
<td>203</td>
</tr>
<tr>
<td>C15020D S G450</td>
<td>S090A2</td>
<td>9.94   1.94</td>
<td>10.53</td>
<td>16.0</td>
<td>500</td>
<td>459</td>
<td>546</td>
<td>217</td>
</tr>
<tr>
<td>C15020D S G450</td>
<td>S180A2</td>
<td>9.98   1.99</td>
<td>10.74</td>
<td>20.0</td>
<td>500</td>
<td>454</td>
<td>541</td>
<td>200</td>
</tr>
<tr>
<td>C15020D S G450</td>
<td>E180C2</td>
<td>9.78   1.94</td>
<td>10.57</td>
<td>20.0</td>
<td>510</td>
<td>468</td>
<td>557</td>
<td>215</td>
</tr>
<tr>
<td>HRS-350</td>
<td>HRS-10</td>
<td>15.91  9.87</td>
<td>72.50</td>
<td>30.0</td>
<td>330</td>
<td>388</td>
<td>462</td>
<td>213</td>
</tr>
<tr>
<td>HRS-350</td>
<td>HRS-16</td>
<td>19.96  15.89</td>
<td>155.00</td>
<td>25.0</td>
<td>322</td>
<td>411</td>
<td>489</td>
<td>190</td>
</tr>
<tr>
<td>Test</td>
<td>Section</td>
<td>Maximum applied force (kN)</td>
<td>Failure mode</td>
<td>Maximum moment resistance Moment (kNm) Rotation (rad.)</td>
<td>Moment resistance at 0.05 rad (kN/m.rad)</td>
<td>Initial stiffness</td>
<td>Member thickness (mm)</td>
<td>Yield strength (N/mm²)</td>
</tr>
<tr>
<td>--------</td>
<td>---------</td>
<td>-----------------------------</td>
<td>--------------</td>
<td>--------------------------------------------------------</td>
<td>----------------------------------------</td>
<td>------------------</td>
<td>-----------------------</td>
<td>-----------------------</td>
</tr>
<tr>
<td>S090A1</td>
<td>C15016DS</td>
<td>6.08</td>
<td>BFcsw</td>
<td>6.72</td>
<td>0.040</td>
<td>-</td>
<td>750</td>
<td>1.62</td>
</tr>
<tr>
<td>S180A1</td>
<td>C15020DS</td>
<td>15.14</td>
<td>BFcsw/FFgp</td>
<td>16.11</td>
<td>0.040</td>
<td>-</td>
<td>1000</td>
<td>1.98</td>
</tr>
<tr>
<td>S240A1</td>
<td>C15016DS</td>
<td>13.57</td>
<td>FFgp</td>
<td>14.45</td>
<td>0.010</td>
<td>-</td>
<td>1500</td>
<td>1.63</td>
</tr>
<tr>
<td>E180C1</td>
<td>C15020DS</td>
<td>9.10</td>
<td>FFgp</td>
<td>19.37</td>
<td>0.030</td>
<td>-</td>
<td>1300</td>
<td>1.95</td>
</tr>
<tr>
<td>E240C1</td>
<td>C15020DS</td>
<td>9.48</td>
<td>FFgp</td>
<td>20.18</td>
<td>0.041</td>
<td>-</td>
<td>1600</td>
<td>1.94</td>
</tr>
<tr>
<td>S090A2</td>
<td>C15020DS</td>
<td>16.81</td>
<td>BFcsw</td>
<td>18.59</td>
<td>0.075</td>
<td>12.00</td>
<td>750</td>
<td>1.94</td>
</tr>
<tr>
<td>S180A2</td>
<td>C15020DS</td>
<td>20.94</td>
<td>FFcs (CFS)</td>
<td>19.80</td>
<td>0.031</td>
<td>-</td>
<td>1600</td>
<td>1.99</td>
</tr>
<tr>
<td>E180C2</td>
<td>C15020DS</td>
<td>11.05</td>
<td>FFcs (CFS)</td>
<td>20.89</td>
<td>0.012</td>
<td>-</td>
<td>1600</td>
<td>1.94</td>
</tr>
</tbody>
</table>

Notes:
- S denotes an internal beam-column sub-frame with a 'cross' sharpened gusset plate under lateral load
- E denotes an external beam-column sub-frame with a 'tee' sharpened gusset plate under lateral load
- 90 denotes a bolt pitch of 90 mm
- 180 denotes a bolt pitch of 180 mm
- 240 denotes a bolt pitch of 240 mm
- A denotes 4 bolts per member
- C denotes 4 bolts per member (same as A)
- 1 denotes the thickness of gusset plate thickness at 10 mm
- 2 denotes the thickness of gusset plate thickness at 16 mm

The measured moment capacities of C15016DS G450 and C15020DS G450 are 16.95 kNm and 21.36 kNm respectively.
CHAPTER FOUR

EXPERIMENTAL INVESTIGATION OF BEAM-COLUMN SUB-FRAMES UNDER LATERAL LOAD - TEST SERIES II

4.1 Objectives of investigation

A total of eight beam-column sub-frame tests with three different connection configurations were carried out. The objectives of this test series are:

- To establish the structural performance of the proposed bolted moment connections with engineered gusset plates in cold-formed steel beam-column sub-frames, in particular, the moment resistances of the connections.

- To quantify the structural efficiency of the proposed bolted moment connections in terms of the moment capacities of the connected sections.
4.2 Test program

All the test specimens are constructed according to the proposed basic configurations with systematic variations in the connection details, i.e. bolt arrangement (or bolt pitch), and shape and thickness of gusset plates. Engineered gusset plates with 50 mm deep chamfers are used in all tests.

Details of the eight beam-column sub-frames are summarized as follows:

<table>
<thead>
<tr>
<th>Test series</th>
<th>Test specimen</th>
<th>Beam column connection</th>
<th>No of bolts per members</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Gusset plate*</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Shape</td>
<td>Thickness</td>
</tr>
<tr>
<td>II-1</td>
<td>S180D1</td>
<td>Cross</td>
<td>10 mm</td>
</tr>
<tr>
<td></td>
<td>S240D1</td>
<td>Cross</td>
<td>10 mm</td>
</tr>
<tr>
<td></td>
<td>E180D1</td>
<td>Tee</td>
<td>10 mm</td>
</tr>
<tr>
<td></td>
<td>E240D1</td>
<td>Tee</td>
<td>10 mm</td>
</tr>
<tr>
<td>II-2</td>
<td>S180D4</td>
<td>Cross</td>
<td>6 mm</td>
</tr>
<tr>
<td></td>
<td>S240D4</td>
<td>Cross</td>
<td>6 mm</td>
</tr>
<tr>
<td></td>
<td>E180D4</td>
<td>Tee</td>
<td>6 mm</td>
</tr>
<tr>
<td></td>
<td>E240D4</td>
<td>Tee</td>
<td>6 mm</td>
</tr>
</tbody>
</table>

*Note: Engineered gusset plates with 50 mm deep chamfers are used.

The connection configurations are specified by the following parameters:

<table>
<thead>
<tr>
<th>Type of beam-column</th>
<th>S</th>
<th>E</th>
<th>Internal</th>
<th>External</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bolt pitch</td>
<td>180</td>
<td>240</td>
<td>180 mm (1.2 D)</td>
<td>240 mm (1.6 D)</td>
</tr>
<tr>
<td>Shape of gusset plate</td>
<td>D</td>
<td></td>
<td>Round corners with 50 mm deep chamfers</td>
<td></td>
</tr>
<tr>
<td>Thickness of gusset plate</td>
<td>1</td>
<td>4</td>
<td>10 mm (5.0 t)</td>
<td>6 mm (3.0 t)</td>
</tr>
</tbody>
</table>
For example, for internal beam-column sub-frames, a beam-column connection with a symmetrical gusset plate of 10 mm thick and 50 mm chamfers using 4 bolts per member at 180 mm bolt pitch is referred as "S180D1".

In this test series, only one section size is used, namely C15020. All the members of the test specimens are double sections which are designated as DS. The design yield strength of the sections is 450 N/mm² which is designated as G450. The bolts are of 16 mm in diameter and of grade 8.8. The nominal section dimensions of the section are summarized as follows:

<table>
<thead>
<tr>
<th>Designation</th>
<th>Depth (mm)</th>
<th>Flange width (mm)</th>
<th>Lip (mm)</th>
<th>Thickness (mm)</th>
<th>Internal radius (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C15020DS G450</td>
<td>150</td>
<td>60</td>
<td>14</td>
<td>2.0</td>
<td>4.0</td>
</tr>
</tbody>
</table>

4.2.1 Test series II-1 ~S180D1, S240D1, E180D1 and E240D1

In order to examine the structural performance of the proposed bolted moment connections using engineered gusset plates against various pitches, two internal beam-column sub-frame tests with bolt pitches of 180 mm and 240 mm are selected for examination, i.e. S180D1 and S240D1 as shown in Figure 4.1a; the thickness of the gusset plates is 10 mm. Two external beam-column sub-frame tests with similar
connection configurations are also carried out for comparison; i.e. E180D1 and E240D1 as shown in Figure 4.1b. Refer also to Figure 4.2 for details of the dimensions of the connections.

4.2.2 Test series II-2 – S180D4, S240D4, E180D4 and E240D4

In order to examine the structural performance of the proposed bolted moment connection against the thickness of gusset plates, two internal beam-column sub-frame tests with engineered gusset plates of 6 mm thick were carried out, i.e. S180D4 and S240D4. Two additional external beam-column sub-frames with engineered gusset plates of 6 mm thick were also carried out for comparison, i.e. E180D4 and E240D4. Refer also to Figure 4.2 for details of the dimensions of the connections.

4.3 Test instrumentation and procedure

The general layout of the test set-up is shown in Figure 4.3. During the tests, the applied loads and the displacements of the test specimens were measured using load cells and transducers as also shown in Figure 4.3. Any out-of-plane deformation was prohibited by two sets of restraining rollers that were installed in front of and behind the beam members at two sides. The tests would be terminated when large deformation occurred in the test specimen, or section failure with yielding or buckling
was observed. In most cases, a pre-load of 2 kN was applied before the tests to ensure that all bolts were in contact with the section webs of connected members despite all the bolt holes were 'perfect-fitted' to 16 mm diameter bolts. Both the instrumentation and the test procedures are similar to those tests in Test Series I reported in Chapter 3.

4.4 Failure modes

Two modes of failure on the connections of the beam-column sub-frames are identified among the eight tests:

- **FFcs**  \textit{Flexural failure of connected cold-formed steel section.}

- **LTBgp**  \textit{Lateral torsional buckling of connected gusset plate.}

4.4.1 \textit{Flexural failure of connected cold-formed steel section (FFcs)}

This failure mode only occurs when the applied moment approaches the moment capacities of the connected sections. Once the failure mode is apparent, the test specimen unloads quickly. The flexural failure of the connected cold-formed steel section is a desirable and favourable failure mode because high moment resistance in the connections may be readily mobilized. Refer to Figure 4.4 for typical failure.
4.4.2 Lateral torsional buckling of connected gusset plate (LTBgp)

This failure mode only occurs in thin gusset plates under a moderate applied moment.

Once the failure mode is apparent, the test specimen unloads immediately. In practice, this failure mode may be eliminated by the provision of sufficient lateral restraints. Refer to Figure 4.5 for typical failure.

4.5 Test results

Table 4.1 summarizes the results of the coupon tests of both cold-formed steel sections and gusset plates of all the test specimens. Both the load-deflection curves and the moment-rotation curves of the eight test specimens are plotted in Figures 4.6 to 4.13. All the load-deflection curves and the moment rotation curves are grouped together for direct comparison in Figures 4.14 and 4.15 for connections with 6 mm and 10 mm thick gusset plates respectively. The rotation calculations for the internal and the external beam-column sub-frames based on measured displacements at specific locations of the test specimens are shown in Figure 4.3. The results of the test series are summarized in Table 4.2.

In order to assess the effectiveness of a connection moment, a moment resistance ratio ($\Psi$) is established which is defined as follows:
\[ \psi = \frac{\text{Measured moment resistance of connection}}{\text{Measured moment capacity of connected member}} \] (4.1)

For flexible test specimens, the moment resistances of the connections are limited to the applied moment at a rotation of 0.05 radian in order to avoid excessive deformation of the connected members.

The measured moment capacity of C15020DS G450 is 21.36 kNm after normalized to design thickness and yield strength; the moment capacity is measured from four-point load tests. The normalised moment resistances of the test specimens are evaluated at the failure positions of the connections. In the present analysis, the moment resistances are first evaluated at the centreline of the connections, and different level arm coefficients are then applied according to the associated failure modes to give the moment resistances of the connections at the failure positions. Refer to Figure 4.16 for details of the level arm coefficients.

4.5.1 Test Series II-1 —Tests S180D1, S240D1, E180D1 and E240D1

In this test series, large deflections and rotations of the test specimens were observed during load application. The normalised moment resistances of the bolted moment connections in tests S180D1, S240D1, E180D1 and E240D1 at the failure positions of
the beam-column connections are found to be 18.61 kNm, 19.59 kNm, 19.71 kNm and 18.66 kNm respectively. The initial stiffnesses of the connections in tests S180D1, S240D1, E180D1 and E240D1 are 1500 kNm/rad., 2000 kNm/rad., 1500kNm/rad. and 2000 kNm/rad. respectively. The rotation capacities of the beam-column connections in tests S180D1, S240D1, E180D1 and E240D1 obtained from the moment-rotation curves are 0.040 radian, 0.036 radian, 0.043 radian and 0.038 radian respectively.

During the tests, there was no out-of-plane deformation in the test specimens. All test specimens failed suddenly in flexural failure at the connected sections. Little bearing around bolt holes was observed and little bending deformation was observed in the hot-rolled steel gusset plates. After the tests, all the members of the test specimens were disassembled from the connections for inspection.

The moment resistance ratios, Ψ, at the failure positions of the connections in tests S180D1, S240D1, E180D1 and E240D1 are found to be 0.87, 0.92, 0.92 and 0.87 respectively.
4.5.2 Test Series II-2 — Tests S180D4, S240D4, E180D4 and E240D4

In this test series, large deflections and rotations of the test specimens were observed during load application. The normalised moment resistances of the bolted moment connections in tests S180D4, S240D4, E180D4 and E240D4 at the failure positions of the beam-column connections are found to be 13.78 kNm, 14.33 kNm, 13.65 kNm and 13.23 kNm respectively. The initial stiffnesses of the connections in tests S180D4, S240D4, E180D4 and E240D4 are 1300 kNm/rad., 1500 kNm/rad., 1300 kNm/rad. and 1200 kNm/rad. respectively. Due to excessive deformation of test specimens S180D4, S240D4 and E180D4, the rotation capacities of the beam-column connections were limited to be 0.05 radian. The rotation capacity of the beam-column connection in test E240D4 was 0.020 radian.

During the tests, significant out-of-plane deformation of the connected sections was observed when the applied moment at the failure positions reached about 15 kNm. Upon further loading, large out-of-plane deformation of the gusset plates were apparent.
After the tests, all the members of the sub-frames were disassembled from the connections for inspection. Little bearing around bolt holes was observed while twisting in the hot-rolled steel gusset plates were apparent.

The moment resistance ratios, $\Psi$, at the failure positions of the connections in tests $S180D4$, $S240D4$, $E180D4$ and $E240D4$ are found to be 0.65, 0.67, 0.64 and 0.62 respectively.

4.6 Comparisons

4.6.1 Connections with thick gusset plates

For connections with thick gusset plates, it is shown that flexural failure of connected cold-formed steel sections is always critical, and the corresponding moment resistance ratio of the connections, $\Psi$, is at least 0.85. This is regarded as a favourable mode of failure with high structural efficiency.

4.6.2 Connections with thin gusset plates

For connections with thin gusset plates, lateral torsional buckling of hot-rolled steel gusset plate is always critical, and the corresponding moment resistance ratio, $\Psi$, of
the connections is about 0.60. This failure mode is not considered to be efficient with limited structural efficiency.

4.6.3 Optimal bolt pitch

For connections with similar configurations but with bolt pitches of 180 mm and 240 mm, it is shown that there is little difference among the moment resistance ratios. Thus, the bolt pitch of 180 mm may be considered as an optimal value in mobilising high moment resistance of the connections.

4.6.4 Connections of high structural efficiency

The overall structural performance of the proposed bolted moment connections with a bolt pitch of 180 mm using 4 bolts per member under lateral loadings may be summarized as follows:

<table>
<thead>
<tr>
<th>Test Series</th>
<th>Gusset plate</th>
<th>Gusset plate</th>
<th>Moment resistance ratio ($\psi$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>16 mm cross</td>
<td>FFcs</td>
<td>0.95</td>
</tr>
<tr>
<td>II</td>
<td>10 mm chamfer</td>
<td>FFcs</td>
<td>0.90</td>
</tr>
<tr>
<td>I</td>
<td>10 mm cross</td>
<td>FFgp</td>
<td>0.70</td>
</tr>
<tr>
<td>II</td>
<td>6 mm chamfer</td>
<td>LTBgp</td>
<td>0.65</td>
</tr>
</tbody>
</table>

For connections with thick gusset plates, it is shown that a high moment resistance ratio of 0.95 is achieved and the critical failure mode is Mode FFcs. By reducing the
plate thickness from 16 mm to 10 mm, the moment resistance ratio is reduced to 0.70 and the critical failure mode is switched to Mode FFgp. By the introduction of 50 mm chamfers to the gusset plates, the moment resistance ratio is then increased to 0.90, and flexural failure occurs in the connected section rather than the gusset plate.

Consequently, it is shown that the use of 50 mm chamfers is effective in strengthening the gusset plates with significant enhancement to the moment capacity of the connections.

It should also be noted that the use of 50 mm chamfers in 6 mm thick gusset plates is also effective, and the moment resistance ratio is shown to be similar to that of a connection with a gusset plate of 10 mm thickness but without chamfers.

4.7 Conclusions

A total of eight beam-column sub-frame tests using engineered gusset plates were carried out. Among the eight tests, two different modes of failure were observed. In general, the connections were not able to develop full moment capacity of the connected sections due to discontinuity of the load path along section flanges in the sections. The moment resistances of the proposed connections using engineered
gusset plates were found to range from 62% to 92% of the moment capacity of the connected sections.

For C15020DS sections, the following parameters are considered to be effective in forming bolted moment connections of high structural efficiency:

a) 4 bolts per member;

b) a bolt pitch of 180 mm;

c) an engineered hot-rolled steel gusset plate with a thickness larger than or equal to 10 mm; 50 mm deep chamfers are provided in the gusset plate.

The proposed connections are expected to be structurally efficient with connection resistances attaining at least 85% of the moment capacity of the connected sections.
a) Internal beam-column sub-frames with 'chamfer' shaped gusset plates.

b) External beam-column sub-frames with 'chamfer' shaped gusset plates.

Figure 4.1 General layout of beam-column sub-frame tests.
All Bolts are 16mm in diameter. (M8.8)

10mm thick gusset plate with chamfer sections

Figure 4.2a: Connection detail of test specimen S180D1.
All bolts are 16mm in diameter. (M8.8)

10mm thick gusset plate with chamfer sections

Figure 4.2b: Connection detail of test specimen S240D1.
All Bolts are 16mm in diameter. (M8.8)

10mm thick gusset plate with chamfer sections

Figure 4.2c: Connection detail of test specimen E180D1.
All bolts are 16mm in diameter. (M8.8)

10mm thick gusset plate with chamfer sections

Figure 4.2d: Connection detail of test specimen E240D1.
All Bolts are 16mm in diameter. (M8.8)

6mm thick gusset plate with chamfer sections

Figure 4.2e: Connection detail of test specimen S180D4.
All bolts are 16mm in diameter. (M8.8)

6mm thick gusset plate with chamfer sections

Figure 4.2f: Connection detail of test specimen S240D4.
All Bolts are 16mm in diameter. (M8.8)

6mm thick gusset plate with chamfer sections

Figure 4.2g: Connection detail of test specimen E180D4.
All bolts are 16mm in diameter. (M8.8)

6mm thick gusset plate with chamfer sections

Figure 4.2h: Connection detail of test specimen E240D4.
Figure 4.3a  Overall view of test set-up of beam-column sub-frame tests.

Figure 4.3b  General layout of instrumentations in beam-column sub-frame tests.
Figure 4.3c  General layout of test set-up.
Figure 4.3d  Details of rotation calculation for internal beam-column sub-frames.
Relative rotation:

\[ R_{ae} = \frac{\Delta 16 \times \sqrt{2}}{370} \]

\[ R_{bd} = \frac{\Delta 17 \times \sqrt{2}}{370} \]

Figure 4.3e  Details of relative rotation calculation for beam-column sub-frames.
Figure 4.4a  FFCs: Flexural failure of connected sections in internal beam-column sub-frame tests.

Figure 4.4b  FFCs: Flexural failure of connected sections in external beam-column sub-frame tests.
Figure 4.5a  LTBgp: Lateral torsional buckling of connected gusset plate.

Figure 4.5b  LTBgp: Top views of Lateral torsional buckling of connected gusset plate.
Figure 4.6a  Load deflection curve of test specimen S180D1.

Figure 4.6b  Moment-joint rotation curves of test specimen S180D1.
Figure 4.7a  Load deflection curve of test specimen E180D1.

Figure 4.7b  Moment-joint rotation curves of test specimen E180D1.
Figure 4.8a  Load deflection curve of test specimen S240D1.

Figure 4.8b  Moment-joint rotation curves of test specimen S240D1.
Figure 4.9a  Load deflection curve of test specimen E240D1.

Figure 4.9b  Moment-joint rotation curves of test specimen E240D1.
Figure 4.10a  Load deflection curve of test specimen S180D4.

Figure 4.10b  Moment-joint rotation curves of test specimen S180D4.
Figure 4.11a  Load deflection curve of test specimen E180D4.

Figure 4.11b  Moment-joint rotation curves of test specimen E180D4.
Figure 4.12a  Load deflection curve of test specimen S240D4.

Figure 4.12b  Moment-joint rotation curves of test specimen S240D4.
Figure 4.13a  Load deflection curve of test specimen E240D4.

Figure 4.13b  Moment-joint rotation curves of test specimen E240D4.
Figure 4.14a  Load deflection curves of connections with 10 mm thick gusset plates.

Figure 4.14b  Load deflection curves of connections with 6 mm thick gusset plates.
Figure 4.15a  Moment-joint rotation curves of connections with 10 mm thick gusset plates.

Figure 4.15b  Moment-joint rotation curves of connections with 6 mm thick gusset plates.
Mode of failure

FFcs (For bolt pitch = 180 mm)
FFcs (For bolt pitch = 240 mm)
LTBgp

Level arm coefficient

1710/2000 = 0.86
1650/2000 = 0.83
1925/2000 = 0.96

Figure 4.16 Level arm coefficient in lateral loading tests.
<table>
<thead>
<tr>
<th>Designation</th>
<th>Specimen</th>
<th>Width (mm)</th>
<th>Thickness (mm)</th>
<th>Maximum applied force (N)</th>
<th>Elongation (%)</th>
<th>Young's modulus (N/mm²)</th>
<th>0.84 U_s (N/mm²)</th>
<th>U_s (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C150200G50</td>
<td>S180D01-1</td>
<td>10.00</td>
<td>9.00</td>
<td>2.01</td>
<td>9.98</td>
<td>20.0</td>
<td>482</td>
<td>417</td>
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<tr>
<td>C150200G50</td>
<td>S180D01-2</td>
<td>10.55</td>
<td>9.50</td>
<td>2.05</td>
<td>9.50</td>
<td>15.0</td>
<td>488</td>
<td>433</td>
</tr>
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<td>C150200G50</td>
<td>S180D01-3</td>
<td>9.78</td>
<td>9.78</td>
<td>2.02</td>
<td>9.78</td>
<td>15.0</td>
<td>487</td>
<td>433</td>
</tr>
<tr>
<td>C150200G50</td>
<td>S180D01-4</td>
<td>10.05</td>
<td>10.05</td>
<td>2.02</td>
<td>10.05</td>
<td>15.0</td>
<td>492</td>
<td>436</td>
</tr>
<tr>
<td>C150200G50</td>
<td>S180D02-1</td>
<td>10.05</td>
<td>9.50</td>
<td>2.00</td>
<td>10.05</td>
<td>15.0</td>
<td>496</td>
<td>437</td>
</tr>
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<td>S180D02-2</td>
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<td>10.05</td>
<td>2.00</td>
<td>10.05</td>
<td>15.0</td>
<td>492</td>
<td>436</td>
</tr>
<tr>
<td>C150200G50</td>
<td>E180D01</td>
<td>10.00</td>
<td>10.00</td>
<td>2.05</td>
<td>10.00</td>
<td>15.0</td>
<td>478</td>
<td>424</td>
</tr>
<tr>
<td>C150200G450</td>
<td>S240D01</td>
<td>10.06</td>
<td>10.06</td>
<td>2.06</td>
<td>10.06</td>
<td>15.0</td>
<td>478</td>
<td>424</td>
</tr>
<tr>
<td>C150200G450</td>
<td>S240D02</td>
<td>9.87</td>
<td>9.87</td>
<td>2.03</td>
<td>9.87</td>
<td>15.0</td>
<td>484</td>
<td>425</td>
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<tr>
<td>C150200G450</td>
<td>S240D04</td>
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<td>10.04</td>
<td>2.04</td>
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<td>15.0</td>
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<td>428</td>
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<td>10.03</td>
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<td>487</td>
<td>427</td>
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<td>10.03</td>
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<td>2.01</td>
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<td>15.0</td>
<td>487</td>
<td>427</td>
</tr>
</tbody>
</table>

Table 4.1: Summary of tensile test results
<table>
<thead>
<tr>
<th>Test</th>
<th>Section</th>
<th>Maximum applied force (kN)</th>
<th>Failure mode</th>
<th>Maximum moment resistance Moment (kNm)</th>
<th>Rotation (rad.)</th>
<th>Moment resistance at 0.05 rad (kNm)</th>
<th>initial stiffness (kNm/rad.)</th>
<th>Member Thickness (mm)</th>
<th>Yield strength (N/mm²)</th>
<th>Gusset plate Thickness (mm)</th>
<th>Yield strength (N/mm²)</th>
<th>Normalised moment (kNm)</th>
<th>Failure position of connection</th>
<th>Ψ (CFS)</th>
<th>Ψ (HRS)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S180D1</td>
<td>C15020DS</td>
<td>21.47</td>
<td>FFcs</td>
<td>20.31</td>
<td>0.040</td>
<td>-</td>
<td>1500</td>
<td>2.02</td>
<td>486</td>
<td>9.96</td>
<td>302</td>
<td>18.61</td>
<td>-</td>
<td>0.87</td>
<td></td>
</tr>
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<td>S240D1</td>
<td>C15020DS</td>
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<td>FFcs</td>
<td>20.91</td>
<td>0.036</td>
<td>-</td>
<td>2000</td>
<td>2.01</td>
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<td>9.96</td>
<td>302</td>
<td>19.59</td>
<td>-</td>
<td>0.92</td>
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<td>C15020DS</td>
<td>22.70</td>
<td>FFcs</td>
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<td>-</td>
<td>1500</td>
<td>2.06</td>
<td>476</td>
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<td>302</td>
<td>19.71</td>
<td>-</td>
<td>0.92</td>
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<td>C15020DS</td>
<td>20.86</td>
<td>FFcs</td>
<td>19.03</td>
<td>0.038</td>
<td>-</td>
<td>2000</td>
<td>2.04</td>
<td>450</td>
<td>9.96</td>
<td>302</td>
<td>18.66</td>
<td>-</td>
<td>0.87</td>
<td></td>
</tr>
<tr>
<td>S180D4</td>
<td>C15020DS</td>
<td>17.82</td>
<td>LTBgp</td>
<td>18.97</td>
<td>0.099</td>
<td>16.65</td>
<td>1300</td>
<td>2.02</td>
<td>490</td>
<td>5.96</td>
<td>329</td>
<td>13.78</td>
<td>-</td>
<td>0.65</td>
<td>0.88</td>
</tr>
<tr>
<td>S240D4</td>
<td>C15020DS</td>
<td>16.82</td>
<td>LTBgp</td>
<td>17.90</td>
<td>0.058</td>
<td>17.40</td>
<td>1500</td>
<td>2.03</td>
<td>468</td>
<td>5.96</td>
<td>329</td>
<td>14.33</td>
<td>-</td>
<td>0.87</td>
<td>0.92</td>
</tr>
<tr>
<td>E180D4</td>
<td>C15020DS</td>
<td>9.35</td>
<td>LTBgp</td>
<td>19.90</td>
<td>0.130</td>
<td>16.50</td>
<td>1300</td>
<td>2.02</td>
<td>477</td>
<td>5.96</td>
<td>329</td>
<td>13.65</td>
<td>-</td>
<td>0.64</td>
<td>0.87</td>
</tr>
<tr>
<td>E240D4</td>
<td>C15020DS</td>
<td>7.75</td>
<td>LTBgp</td>
<td>16.07</td>
<td>0.020</td>
<td>-</td>
<td>1200</td>
<td>2.03</td>
<td>482</td>
<td>5.96</td>
<td>329</td>
<td>13.23</td>
<td>-</td>
<td>0.62</td>
<td>0.85</td>
</tr>
</tbody>
</table>

Notes:
- **S** denotes an internal beam-column sub-frame with a 'cross' sharpened gusset plate under lateral load
- **E** denotes an external beam-column sub-frame with a 'tee' sharpened gusset plate under lateral load
- **180** denotes a bolt pitch of 180 mm
- **240** denotes a bolt pitch of 240 mm
- **D** denotes 4 bolts per member with chamfers in gusset plate
- **1** denotes the thickness of gusset plate thickness at 10 mm
- **4** denotes the thickness of gusset plate thickness at 6 mm

The measured moment capacity of C15020DS G450 is 21.36 kNm.
CHAPTER FIVE

EXPERIMENTAL INVESTIGATION OF BEAM-COLUMN SUB-FRAMES UNDER GRAVITY LOAD – TEST SERIES III

5.1 Objectives of investigation

A total of nine beam-column sub-frame tests with different connection configurations were carried out under gravity load. The objectives of this test series are:

- To establish the structural performance of the proposed bolted moment connections in cold-formed steel beam-column sub-frames under gravity load, in particular, the moment resistances of the connections.

- To quantify the structural efficiency of the proposed bolted moment connections in terms of the moment capacities of the connected sections.
5.2 Test program

All the test specimens are constructed according to the proposed basic configuration with systematic variations in the connection details, i.e. bolt pitch, and shape and thickness of gusset plates.

Details of the ten beam-column sub-frames are summarized as follows:

<table>
<thead>
<tr>
<th>Test series</th>
<th>Test specimen</th>
<th>Beam column connection</th>
<th>Gusset plate</th>
<th>No of bolts per member</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Shape</td>
<td>Thickness</td>
</tr>
<tr>
<td>III-1</td>
<td>G180A1S</td>
<td>Cross</td>
<td>10 mm</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>G180A2S</td>
<td>Cross</td>
<td>16 mm</td>
<td>4</td>
</tr>
<tr>
<td>III-2</td>
<td>G180D1S</td>
<td>Cross with chamfers</td>
<td>10 mm</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>G180D4S</td>
<td>Cross with chamfers</td>
<td>6 mm</td>
<td>4</td>
</tr>
<tr>
<td>III-3</td>
<td>G440A2L</td>
<td>Cross with chamfers</td>
<td>16 mm</td>
<td>4</td>
</tr>
<tr>
<td>III-4</td>
<td>G340D1L</td>
<td>Cross with chamfers</td>
<td>10 mm</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>G340D1S</td>
<td>Cross with chamfers</td>
<td>10 mm</td>
<td>4</td>
</tr>
<tr>
<td>III-5</td>
<td>G340D4L</td>
<td>Cross with chamfers</td>
<td>6 mm</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>G440D4S</td>
<td>Cross with chamfers</td>
<td>6 mm</td>
<td>4</td>
</tr>
</tbody>
</table>

The connection configurations are specified by the following parameters:

<table>
<thead>
<tr>
<th>Shape of gusset plate</th>
<th>G</th>
<th>Symmetrical cross section under gravity load</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bolt pitch</td>
<td></td>
<td></td>
</tr>
<tr>
<td>180</td>
<td></td>
<td>180 mm (1.2 D)</td>
</tr>
<tr>
<td>340</td>
<td></td>
<td>340 mm (1.36 D)</td>
</tr>
<tr>
<td>440</td>
<td></td>
<td>440 mm (1.76 D)</td>
</tr>
<tr>
<td>Shape of gusset plate</td>
<td>A</td>
<td>Round corners</td>
</tr>
<tr>
<td></td>
<td>D</td>
<td>Round corners with 50 mm deep chamfers</td>
</tr>
<tr>
<td>Thickness of gusset plate</td>
<td>1</td>
<td>10 mm (6.25 t, 5.0 t, 4.0 t)</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>16 mm (10.0 t, 8.0 t)</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>6 mm (3.75 t, 3.0 t)</td>
</tr>
</tbody>
</table>
For example, for internal beam-column sub-frames, a beam-column connection with a symmetrical gusset plate of 6 mm thick and 50 mm deep chamfers using 4 bolts per member at 180 mm bolt pitch subjected to gravity loading is referred as "G180D4".

In this test series, three section sizes are used, namely C15016, C25020 and C25025. All the members of the test specimens are double sections which are designated as DS. The design yield strengths of the sections are 300 N/mm² and 450 N/mm² which are designated as G300 and G450 respectively. The bolts are of 16 mm in diameter and of grade 8.8. The nominal section dimensions of the section are summarized as follows:

<table>
<thead>
<tr>
<th>Designation</th>
<th>Depth (mm)</th>
<th>Flange width (mm)</th>
<th>Lip (mm)</th>
<th>Thickness (mm)</th>
<th>Internal radius (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C15016DS G300</td>
<td>150</td>
<td>64</td>
<td>17</td>
<td>1.56</td>
<td>3.2</td>
</tr>
<tr>
<td>C25020DS G300</td>
<td>250</td>
<td>78</td>
<td>20</td>
<td>2.01</td>
<td>4.0</td>
</tr>
<tr>
<td>C25025DS G450</td>
<td>254</td>
<td>78</td>
<td>23</td>
<td>2.50</td>
<td>5.0</td>
</tr>
</tbody>
</table>

5.2.1 Test series III-1: G180A1 and G180A2

In order to examine the structural performance of the proposed bolted moment connections against thickness of gusset plates, two internal beam-column sub-frame tests with a bolt pitch of 180 mm were carried out as shown in Figure 5.1; the
thickness of the gusset plates is 10 mm and 16 mm respectively. Refer to Figure 5.2 for details.

5.2.2 Test series III-2: G180D1 and G180D4

In order to examine the structural performance of the proposed bolted moment connections against thickness of engineered gusset plates with 50 mm deep chamfers, two internal beam-column sub-frame tests with engineered gusset plates of 10 mm and 6 mm thick were carried out as shown in Figure 5.1. Thus, the effect of the chamfers on the structural performance of the proposed connection configurations may be assessed. Refer to Figure 5.2 for details.

5.2.3 Test series III-3: G440A2L

In order to examine the structural performance of the proposed bolted moment connections for 250 mm deep cold-formed steel sections, one internal beam-column sub-frame was carried out with a regular gusset plate of 16 mm under high bending moment and low shear force. The overall beam span is 4 m. A bolt pitch of 440 mm is used. Thus, the moment resistance of the proposed connection configuration for 250 mm deep sections may be established. Refer to Figure 5.2 for details.
5.2.4 Test series III-4: G340D1L and G340D1S

In order to examine the structural performance of the proposed bolted moment connections with engineered gusset plates, two internal beam-column sub-frames were carried out with different beam spans of 4 m and 2.5 m, i.e. under different moment shear ratios. The engineered gusset plates are 10 mm thick with 50 mm deep chamfers, and a bolt pitch of 340 mm is used in both tests. Thus, the effect of moment shear ratio on the structural performance of the proposed connection configurations may be assessed. Refer to Figure 5.2 for details.

5.2.5 Test series III-5: G340D4L and G440D4S

In order to examine the structural performance of the proposed bolted moment connections with engineered gusset plates against reduced gusset plate thickness, two internal beam-column sub-frames were carried out. For test G340D4L, the bolt pitch of the connections is 340 mm while the overall span is 4 m. For test G440D4S, the bolt pitch of the connections is increased to 440 mm while the overall span is reduced to 2.5 m. Thus, the effect of bolt pitches in thin engineered gusset plates of the proposed connection configurations may be assessed under different moment shear ratios. Refer to Figure 5.2 for details.
5.3 Test instrumentation and procedure

The general layout of the test set-up is shown in Figure 5.3. During the tests, the applied loads and the displacements of the test specimens were measured using load cells and transducers as shown in Figure 5.4. Any out-of-plane deformation was prohibited by two sets of restraining rollers that were installed in front of and behind the column members at both sides and also at the top and the bottom ends. The tests would be terminated when large deformation occurred in the test specimens, or section failure with yielding or buckling was observed. In most cases, a pre-load of 2 kN was applied before the tests to ensure that all bolts were in contact with the section webs of connected members despite all the bolt holes were 'perfect-fitted' to 16 mm diameter bolts. Both the test instrumentation and the test procedure are similar to these tests in Test Series I reported in Chapter 3.

5.4 Failure modes

Two modes of failure on the connections of the beam-column sub-frames are identified among the ten tests:

- **FFcs** *Flexural failure of connected cold-formed steel section.*
- **LTBgp** *Lateral torsional buckling of connected gusset plate.*
5.4.1 Flexural failure of connected cold-formed steel section (FFcs)

This failure mode only occurs when the applied moment approaches the moment resistances of the sections. Once the failure mode is apparent, the test specimen unloads quickly. The flexural failure of the connected cold-formed steel section is a desirable and favourable failure mode because high moment resistance in the connections may be readily mobilized. Refer to Figure 5.5 for typical failure.

5.4.2 Lateral torsional buckling of connected gusset plate (LTBgp)

This failure mode only occurs in thin gusset plates under a moderate applied moment. Once the failure mode is apparent, the test specimen unloads immediately. In practice, this failure mode may be eliminated by the provision of sufficient lateral restraints. Refer to Figure 5.6 for typical failure.

5.5 Test results

Table 5.1 summarizes the results of the coupon tests of both cold-formed steel sections and gusset plates of all the test specimens. Both the load-deflection curves and the moment-rotation curves of the ten test specimens are plotted in Figures 5.7 to 5.15. All the load-deflection curves and the moment rotation curves are grouped together for direct comparison in Figures 5.16 to 5.17. The rotation calculations for
the internal beam-column sub-frames based on measured displacements at specific location of the test specimens are shown in Figure 5.4. The results of the test series are summarized in Table 5.2.

In order to assess the effectiveness of a connection moment, a moment resistance ratio ($\Psi$) is established which is defined as follows:

\[
\Psi = \frac{\text{Measured moment resistance of connection}}{\text{Measured moment capacity of connected member}}
\]  \hspace{1cm} (5.1)

For flexible test specimens, the moment resistances of the connections are limited to the applied moment at a rotation of 0.05 radian in order to avoid excessive deformation of the connected members.

The measured moment capacities of C15016DS G300, C25020DS G300 and C25025DS G450 are 10.86 kNm, 34.79 kNm and 62.20 kNm after normalized to design thickness and yield strength; the moment capacity is measured from four-point load tests. The normalised moment resistances of the test specimens are evaluated at the failure positions of the connections. In the present analysis, the moment resistances are first evaluated at the centreline of the connections, and different level
arm coefficients are then applied according to the associated failure modes to give the moment resistances of the connections at the failure positions. Refer to Figure 5.18 for details of the level arm coefficients. For all test specimens in this program, combined bending and shear interaction is expected to be significant due to short shear span.

5.5.1 Test Series III-1: Tests G180A1 and G180A2

In this test series, limited deflections and rotations of the test specimens were observed during load application. The normalised moment resistances of the bolted moment connections in tests G180A1 and G180A2 at the failure positions of the beam-column connections were 8.84 kNm and 8.41 kNm respectively. The initial stiffnesses of the connections in tests G180A1 and G180A2 are 2600 kNm/rad. and 1600 kNm/rad. respectively. The rotation capacities of the beam-column connections in tests G180A1 and G180A2 obtained from the moment-rotation curves are 0.021 radian and 0.014 radian respectively.

During the tests, there was no out-of-plane deformation in the test specimens. After the tests, all the members of the test specimens were disassembled from the connections
for inspection. Little bearing failure was observed around the bolt holes of the section webs.

Both test specimens failed suddenly in flexural failure at the connected sections when the applied forces reached 36.93 kN and 37.14 kN respectively.

The moment resistance ratios, $\Psi$, at the failure positions of the connections in tests $G180A1$ and $G180A2$ are found to be 0.81 and 0.77 respectively.

No increase in the moment resistance ratio was found when the thickness of the gusset plates is increased from 10 mm to 16 mm, and the critical mode of failure in both tests is the flexural failure of connected cold-formed steel section.

5.5.2 Test Series III-2: Tests $G180D1$ and $G180D4$

In this test series, little deflections and rotations of the test specimens were observed during load application. The normalised moment resistances of the bolted moment connections in tests $G180D1$ and $G180D4$ at the failure positions of the beam-column connections were 8.52 kNm and 8.98 kNm respectively. The initial stiffnesses of the connections in tests $G180D1$ and $G180D4$ are 2200 kNm/rad. and 2700 kNm/rad. 

5-10
respectively. The rotation capacities of the beam-column connections in tests

$G180D1$ and $G180D4$ obtained from the moment-rotation curves are 0.019 radian and

0.020 radian respectively.

During the tests, there was no out-of-plane deformation on the test specimens. After the tests, all the members of the sub-frames were disassembled from the connections for inspection. Little bearing failure was observed around the bolt holes in the section webs.

All test specimens failed suddenly in flexural failure at the connected section while the applied forces reached 36.32 kN and 37.38 kN respectively.

The moment resistance ratios, $\Psi$, at the failure positions of the connections in tests $G180D1$ and $G180D4$ are found to be 0.78 and 0.83 respectively.

In the presence of chamfers, no reduction in the moment resistance ratio is found when the thickness of the gusset plate is reduced from 10 mm to 6 mm, and the critical mode of failure in both tests is the flexural failure of the connected sections.
5.5.3 Test Series III-3: Test G440A2L

In this test series, little deflections and rotations of the test specimens were observed during load application. The normalised moment resistances of the bolted moment connections in tests G440A2L at the failure positions of the beam-column connections were 34.51 kNm. The initial stiffness of the connection in test G440A2L is 3000 kNm/rad. The rotation capacity of the beam-column connections in test G440A2L obtained from the moment-rotation curve was 0.038 radian.

During the test, no out-of-plane deformation of the connected sections was observed. The test specimen failed suddenly in flexural failure at the connected section while the applied force reached 62.83 kN.

The moment resistance ratio, \( \Psi \), at the failure position of the connection was 0.99.

5.5.4 Test Series III-4: Tests G340D1L and G340D1S

In this test series, little deflections and rotations of the test specimens were observed during load application. The normalised moment resistances of the bolted moment connections in tests G340D1L and G340D1S at the failure positions of the beam-column connections are found to be 56.78 kNm and 29.17 kNm respectively. The
initial stiffnesses of the connections in tests $G340D1L$ and $G340D1S$ are 4000 kNm/rad. and 2400 kNm/rad. respectively. The rotation capacities of the beam-column connections in tests $G340D1L$ and $G340D1S$ obtained from the moment-rotation curve are 0.033 radian and 0.014 radian respectively.

During the tests, significant out-of-plane deformation of the connected sections was observed in tests $G340D1L$ and $G340D1S$ when the applied moments at the failure positions reached 54 kNm and 27 kNm respectively. Upon further loading, large out-of-plane deformation of the gusset plates were apparent.

The moment resistance ratios, $\Psi$, at the failure positions of the connections in tests $G340D1L$ and $G340D1S$ are found to be 0.91 and 0.83 respectively.

5.5.5 Test Series III-5: Tests G340D4L and G440D4S

In this test series, little deflections and rotations of the test specimens were observed during load application. The normalised moment resistances of the bolted moment connections in tests $G340D4L$ and $G440D4S$ at the failure positions of the beam-column connections are found to be 27.77 kNm and 30.80 kNm respectively. The initial stiffnesses of the connections in tests $G340D4L$ and $G440D4S$ are 3500
kNm/rad. and 2100 kNm/rad. respectively. The rotation capacities of the beam-column connections in tests $G340D4L$ and $G440D4S$ obtained from the moment-rotation curve are 0.014 radian and 0.016 radian respectively.

During the tests, significant out-of-plane deformation of the connected sections was observed in tests $G340D4L$ and $G440D4S$ when the applied moments at the failure positions reached 25 kNm and 28 kNm respectively. Upon further loading, large out-of-plane deformation of the gusset plate was apparent.

The moment resistance ratios, $\Psi$, at the failure positions of the connections in tests $G340D4L$ and $G440D4S$ are found to be 0.79 and 0.88 respectively.

5.6 Comparisons for 150 mm deep sections

5.6.1 Efficient connections - G180A1S, G180A2S, G180D1S and G180D4S

In both test series III-1 and III-2, the moment resistance ratios are found roughly to be 0.80 for connections with gusset plates of different thicknesses with and without chamfers. It is shown that very similar moment resistances are obtained from both connections with thick regular gusset plates and connections with thin engineered gusset plates. This confirms the effectiveness of engineered gusset plates that thin
gusset plates may be used to replace thick gusset plates provided that chamfers of suitable sizes are introduced.

5.7 Comparisons for 250 mm deep sections

5.7.1 Connections with thick gusset plate - Test G440A2L

It is shown that in test G440A2L, the moment resistance ratio is 0.99 with flexural failure in the connected sections. Thus, the proposed connection configuration is shown to be effective in mobilizing the moment capacity of the connected sections.

5.7.2 Connections with engineered gusset plates - Tests G340D1L and G340D1S

It is shown that by reducing the thickness of gusset plate from 16 mm to 10 mm, the moment resistance ratio is reduced slightly from 0.99 to 0.91, provided that chamfers are provided. In the presence of high shear force, the moment resistance ratio is further reduced to 0.83. In both cases, the critical failure mode is lateral torsional buckling of the gusset plates.
5.7.3 Connections with thin engineered gusset plates - Tests G340D4L and G440D4S

By reducing the thickness of the engineered gusset plates from 10 mm to 6 mm as shown in tests G340D1L and G340D4L, it is shown the moment resistance ratio of the connections is reduced from 0.91 to 0.79. By increasing the bolt pitch from 340 mm to 440 mm, the moment resistance ratio is increased to 0.88 despite of increased shear force, as shown in test G440D4S. In both cases, the critical failure mode is lateral torsional buckling of the gusset plates.

5.7.4 Efficient connections with engineered gusset plates

It should be noted that for all connections with 10 mm thick engineered gusset plates, the moment resistances reach 80% of the moment capacities of the connected sections under both high and low moment shear ratios. Moreover, a bolt pitch of 340 mm is found to be sufficient.

5.8 Conclusions

A total of nine internal beam-column tests under gravity loads were carried out. From the tests, two different modes of failure were observed. In general, the connections do not able to develop full moment capacity of the connected sections due to
discontinuity of the load path along section flanges in the sections. The moment resistances of the proposed connections were found to range from 77% to 99% of the moment capacities of the connected sections.

For C15016DS sections, the following parameters are considered to be effective in forming bolted moment connections of high structural efficiency:

a) 4 bolts per member;

b) a bolt pitch of 180 mm,

c) an engineered hot-rolled steel gusset plate with a thickness larger than or equal to 10 mm.

The proposed connections are expected to be structurally efficient with moment resistances attaining at least 75% of the moment capacities of the connected sections under gravity loads.

For C25020DS sections, the following parameters are considered to be effective in forming bolted moment connections of high structural efficiency:
d) 4 bolts per member;

e) a bolt pitch of 340 mm,

f) an engineered hot-rolled steel gusset plate with a thickness larger than or equal to 10 mm.

The proposed connections are expected to be structurally efficient with moment resistances attaining at least 80% of the moment capacities of the connected sections under gravity loads.
Figure 5.1 General layout of beam-column sub-frame tests.
All Bolts are 16mm in diameter. (M8.8)

10mm thick gusset plate

Figure 5.2a: Connection detail of test specimen G180A1S.
All Bolts are 16mm in diameter. (M8.8)

16mm thick gusset plate

Figure 5.2b: Connection detail of test specimen G180A2S.
All Bolts are 16mm in diameter. (M8.8)

10mm thick gusset plate with chamfer sections

Figure 5.2c: Connection detail of test specimen G180D1S.
All Bolts are 16mm in diameter. (M8.8)

6mm thick gusset plate with chamfer sections

Figure 5.2d: Connection detail of test specimen G180D4S.
All Bolts are 16mm in diameter. (M8.8)

16mm thick gusset plate

Figure 5.2e  Connection detail of test specimen G440A2L.
All Bolts are 16mm in diameter. (M8.8)

10mm thick gusset plate

Figure 5.2f  Connection detail of test specimens G340D1S and G340D1L.
All Bolts are 16mm in diameter. (M8.8)

6mm thick gusset plate

Figure 5.2g  Connection detail of test specimens G340D4L.
All Bolts are 16mm in diameter. (M8.8)

6mm thick gusset plate

Figure 5.2h  Connection detail of test specimen G440D4S.
Figure 5.3a  Overall view of C150 section test specimen.

Figure 5.3b  Overall view of C250 section short span test specimen.
Figure 5.3c  Overall view of C250 section long span test specimen.

Figure 5.5a  FFcs: Flexural failure of connected cold-formed steel sections.
Gravity load

Moment rotation:
\[ \theta_G = \frac{\Delta}{L}; \]

Member rotation:
\[ \theta_{CE} = \theta_{87} - \theta_{76} = \left[ (\frac{\Delta_8 - \Delta_7}{l_1}) - (\frac{\Delta_7 - \Delta_6}{l_1}) \right]; \]

\[ \theta_{CE} = \theta_{910} - \theta_{1011} = \left[ (\frac{\Delta_9 - \Delta_{10}}{l_1}) - (\frac{\Delta_{10} - \Delta_{11}}{l_1}) \right]; \]

\[ \theta_{DE} = \theta_{14} - \theta_{45} = \left[ (\frac{\Delta_1 - \Delta_4}{l_1}) - (\frac{\Delta_4 - \Delta_5}{l_1}) \right]; \]

\[ \theta_{DE} = \theta_{1413} - \theta_{1312} = \left[ (\frac{\Delta_{14} - \Delta_{13}}{l_1}) - (\frac{\Delta_{13} - \Delta_{12}}{l_1}) \right]; \]

Figure 5.4 Details of rotation calculation for internal beam-column sub-frames.
Figure 3.5b  Flexural failure of connected cold-formed steel in C150 section.

Figure 3.5c  Flexural failure of connected cold-formed steel in C250 section.
Figure 5.6a  LTBgp: Lateral torsional buckling of connected gusset plate.

Figure 5.6b  Lateral torsional buckling of connected gusset plate in long span test specimen.
Figure 5.7a  Load deflection curve of test specimen G180A1.

Figure 5.7b  Moment-joint rotation curve of test specimen G180A1.
Figure 5.8a  Load deflection curve of test specimen G180A2.

Figure 5.8b  Moment-joint rotation curve of test specimen G180A2.
Figure 5.9a  Load deflection curve of test specimen G180D1.

Figure 5.9b  Moment-joint rotation curve of test specimen G180D1.
Figure 5.10a  Load deflection curve of test specimen G180D4.

Figure 5.10b  Moment-joint rotation curve of test specimen G180D4.
Figure 5.11a  Load deflection curve of test specimen G440A2L.

Figure 5.11b  Moment-joint rotation curve of test specimen G440A2L.
Figure 5.12a  Load deflection curve of test specimen G340D1L.

Figure 5.12b  Moment-joint rotation curve of test specimen G340D1L.
Figure 5.13a  Load deflection curve of test specimen G340D4S.

Figure 5.13b  Moment-joint rotation of test specimen G340D4S.
Figure 5.14a  Load deflection curve of test specimen G340D4L.

Figure 5.14b  Moment-joint rotation curve of test specimen G340D4L.
Figure 5.15a  Load deflection curve of test specimen G440D4S.

Figure 5.15b  Moment-joint rotation curve of test specimen G440D4S.
Figure 5.16a  Load deflection curves of beam-column sub-frame tests.

Figure 5.16b  Moment-joint rotation curves of beam-column sub-frame tests.
Figure 5.17a  Load deflection curves of beam-column sub-frame tests.

Figure 5.17b  Moment-joint rotation curves of beam-column sub-frame tests.
In short span test specimens

Mode of failure  Level arm coefficient
FFCs          \[ \frac{465}{755} = 0.62 \]

Figure 5.18a  Level arm coefficient in gravity loading tests of C150 section.
In short span specimens

Mode of failure
LTBgp

Level arm coefficient
830/955 = 0.87

In long span specimens

Mode of failure
FFcs (bolt pitch = 440)  
LTBgp

Level arm coefficient
1241/1841 = 0.67  
1716/1841 = 0.93

Figure 5.18b  Level arm coefficient in gravity loading tests of C250 section.
Table 5.1 Summary of tensile test results

<table>
<thead>
<tr>
<th>Designation</th>
<th>Test specimens</th>
<th>Dimension of specimens</th>
<th>Maximum applied force (kN)</th>
<th>Elongation (%)</th>
<th>$\sigma_{0.2%}$ (N/mm$^2$)</th>
<th>$0.84 U_s$ (N/mm$^2$)</th>
<th>$U_s$ (N/mm$^2$)</th>
<th>Young's modulus (kN/mm$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C15016DS G300</td>
<td>G180D4</td>
<td>13.39</td>
<td>1.51</td>
<td>7.23</td>
<td>26.0</td>
<td>292.0</td>
<td>300.4</td>
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### Table 5.2 Summary of test data - Test Series III

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<tr>
<th>Test</th>
<th>Section</th>
<th>Maximum applied force (kN)</th>
<th>Failure mode</th>
<th>Maximum moment resistance</th>
<th>Moment resistance at 0.05 rad. (kN)</th>
<th>Initial stiffness (kNmm/rad.)</th>
<th>Member thickness (mm)</th>
<th>Member yield strength (N/mm²)</th>
<th>Gusset plate thickness (mm)</th>
<th>Gusset plate yield strength (N/mm²)</th>
<th>Normalised moment (kNm)</th>
<th>Normalised moment (CFS) (HRS)</th>
<th>( \psi )</th>
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<td>5.96</td>
<td>383</td>
<td>30.8</td>
<td>0.88</td>
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</tbody>
</table>

**Notes:**

- **G** denotes an internal beam-column sub-frame with a 'cross' shaped gusset plate under gravity load.
- **180** denotes a bolt pitch of 180 mm.
- **340** denotes a bolt pitch of 340 mm.
- **440** denotes a bolt pitch of 440 mm.
- **A** denotes 4 bolts per member.
- **D** denotes 4 bolts per member with chamfers in gusset plate.
- **1** denotes the thickness of gusset plate thickness at 10 mm.
- **2** denotes the thickness of gusset plate thickness at 16 mm.
- **4** denotes the thickness of gusset plate thickness at 6 mm.
- **S** denotes a short span beam-column sub-frame.
- **L** denotes a long span beam-column sub-frame.

The measured moment capacities of C15016DS G300, C25020DS G300 and C25025DS G450 are 10.86 kNm, 34.79 kNm and 62.20 kNm respectively.
CHAPTER SIX

NON-LINEAR ANALYSIS OF COLD-FORMED STEEL BEAM-COLUMN SUB-FRAMES WITH SEMI-RIGID CONNECTIONS

6.1 Objectives of investigation

A numerical investigation on cold-formed steel beam-column sub-frames under lateral load was carried out and fully presented in this Chapter. A non-linear analysis with semi-rigid joints was performed to study the structural behaviour of the beam-column sub-frames. The main objectives of the numerical investigation are:

a) To establish the accuracy of the proposed non-linear analysis for cold-formed steel beam-column sub-frames with bolted moment connections.

b) To confirm the use of local rotational characteristic of moment connections measured from tests to predict the overall structural behaviour of beam-column sub-frames.

c) To establish the derivation of rotational characteristic of moment connections from both test data and semi-empirical design rules.
As presented in the previous chapters, the moment resistances of the proposed connection configurations are found to range from 50% to over 95% of the moment capacities of connected sections. Furthermore, it is demonstrated that the most effective connections are those connections with large bolt pitches and thick gusset plates, and thus, the critical mode of failure is the flexural failure in connected sections. Out of the sixteen beam-column sub-frames under lateral load presented in the previous chapters, only six of them are found to fail in this mode of failure. All of them can safely mobilize at least 80% of the moment capacities of the connected sections. For the present investigation, test data of these six test specimens are selected for study. The general test set-up and the connection details of the six tests are presented in Figure 6.1; the test results are summarized in Table 6.1.

For each test, the measured moment-rotation curves of the bolted connections were inputted into a non-linear analysis software GMNA\textsuperscript{[34]} as local connection characteristics of the connections in order to generate the overall lateral deflection history of the beam-column sub-frame. Comparison on the predicted results with measured data is also presented. Moreover, the derivation of the rotational characteristic of the bolted moment connections from measured test data to moment joint-rotation curves is fully described. Finally, a simple design rule for the prediction
of the moment joint-rotation curves of the connections is proposed after calibrating against measured data.

6.2 Evaluation of rotational characteristics of bolted connections

The load deflection curves of the six cold-formed steel beam-column frames with bolted moment connections of high structural efficiency are plotted in Figure 6.2. Owing to the member configurations in internal beam-column sub-frames, four different moment joint-rotation curves, or \( M - \theta \) curves, may be derived, and such four curves for Test S180D1 are presented in Figure 6.3a. Similarly, for external beam-column sub-frames, two different moment joint-rotation curves may be derived, and such two curves for Test E180D1 are plotted in Figure 6.3b. The joint-rotations of the connections are the relative rotations between the column members and the beam members, and the calculation is presented in Figure 6.4.

6.2.1 Errors in rotation measurement

In general, while resistance measurements in tests are always considered to be adequately accurate, errors are always present in deformation measurements. For each beam-column sub-frame considered in the present study, the joint-rotations of the connections are obtained as the relative rotations between the connected beam and column members, each in turn obtained as the relative lateral displacements of section
flanges of the connected sections within the connection zones. While the relative lateral displacements of each member are measured precisely using transducers, it is difficult to define precisely the centres of rotation of each member within the connection zones. Due to the presence of cross-sectional distortion, any measured rotation based on the deflection of the section flange does not necessarily correspond to the rotation of the whole section. In general, this error is considered to be small during the initial deformation stage, but may become significant under high applied loads at large deformation.

Moreover, as the beam-column sub-frames are flexible, significant lateral deformation was measured in the tests, as shown in Figure 6.2. As the transducers are stationary, the reference points for member rotations are moved continuously according to the lateral displacements of the test specimens, as shown in Figure 6.4. Consequently, there is a growing error embedded in the rotation measurements which is directly proportional to the lateral displacement of the test specimens. Consequently, correction to the rotation measurements is necessary.
6.2.2 Correction on measured member rotations

In order to derive the corrected moment joint-rotation curves of bolted moment connections in cold-formed steel beam-column sub-frames in a simple and yet conservative manner, a correction procedure is suggested as follows:

Step 1

An average moment joint-rotation curve of the beam-column connection is first obtained by averaging the measured rotational characteristic curves obtained directly from tests. For internal beam-column sub-frames, all four curves should be used in the averaging whilst only two curves for external beam-column sub-frames.

Step 2

Assuming constant curvatures within the connection zone, a reduction factor $\chi$ is

$$\chi' = \frac{l_{n} - \Delta}{l_{n}}$$

applied to the member rotations of the beam members; the reduction factor $\chi$ is defined as follows:

where $l_{h_{x}}$ = distance between the centre-line of the beam-column connection and the bolt group centre
\[ \Delta = \text{lateral displacement of the centre-line of the beam-column connection measured from tests} \]

The joint-rotation is then computed as the relative rotation between the beam and the column members. Corrected moment joint-rotation curves of Tests S180D1 and E180D1 are plotted in the same graphs of the measured moment-rotation curves for direct comparison, as shown in Figure 6.3. It is shown that the corrected curves follow closely to the measured ones up to two-third of the connection resistances. After that, the tangent stiffness of the corrected curves are reduced by 10 to 20% when compared with those measured ones. Consequently, it is important to use the corrected moment joint-rotation curves in determining the overall structural behaviour of cold-formed steel beam-column sub-frames at large deformation.

6.3 Normalized moment joint-rotation curves

In order to re-present the corrected moment joint-rotation curves of the connections for non-linear analysis with semi-rigid joints, each moment joint-rotation curve is normalized in terms of \( m_r \) and \( \theta_r \), where \( m_r \) is the moment ratio of the connection, and \( \theta_r \) is the normalized connection rotation. The moment ratio, \( m_r \), is equal to the measured moment resistance of the connection, \( M_{con} \), divided by the moment capacity of cold-formed steel sections, \( M_R \). The normalized connection rotation, \( \theta_r \),
is equal to the measured joint-rotation, $\theta$, divided by the rotation parameter $M_R / (EI / L)$, where $EI$ and $L$ are the flexural rigidity and the member length of the connected section. All the normalized moment joint-rotation curves, $m_r - \theta$, curves, are illustrated in Figure 6.5 with the legend of 'Model I'.

It is shown that the curves may be divided into three parts or stages of deformation, namely, the initial stage, the non-linear stage, and the final stage of deformation. While the initial stage represents the linear elastic rotation of the connections, the non-linear stage represents the flexibility of the connections primarily due to local bearing deformation of section web around bolt holes. The final stage is a flat line with zero slope, i.e. zero stiffness, representing flexural failure of the connected sections.

6.4 Non-linear analysis for semi-rigid connections

In order to examine the overall structural behaviour of the cold-formed steel beam-column sub-frames under lateral loads, a non-linear analysis incorporating semi-rigid connections is carried out on the six cold-formed steel beam-column sub-frames. All the beam and the column members are two lipped C sections back-to-back with a section depth of 150 mm and a nominal thickness of 2.0 mm, or C15020DS. The nominal yield strength is 450 N/mm$^2$. 
The finite element model is shown in Figure 6.6. Moreover, the bolted connections are modelled as a rigid link of finite length, $S_r$, which attaches at one end rigidly to the column member, while in a semi-rigid manner to the beam members at the other end. As the moment joint-rotation curves of the connections are derived in such a way that all flexibilities between the beam and the column members have been fully incorporated into the rotational characteristic of the semi-rigid joints, the use of the rigid link is justified.

Based on the dimensions of the test specimens and also the normalized moment joint-rotation curves of the connections, the lateral load resistances of the beam-column sub-frames are predicted with the non-linear analysis; the results of all the six test specimens are summarized in Table 6.2. The measured lateral load resistances of the beam-column sub-frames are also presented in Table 6.2 for comparison.

In order to establish the adequacy of the analysis method, a model factor is established and defined as follows:

$$\psi = \frac{\text{Measured lateral load resistance from test}}{\text{Lateral load resistance obtained from non-linear analysis}}$$
A model factor of unity suggests that the result obtained from the non-linear analysis is conservative, and in general, the value of the model factor is expected to range from 1.0 to 1.2 for structural adequacy and economy. It is shown that the model factors for the six tests range from 0.98 to 1.14 with an average value of 1.04.

Moreover, the predicted lateral load-deflection curves of the six tests are illustrated in Figure 6.7 with the legend of 'Model I'. Moreover, the measured lateral load-deflection curves of the six tests are also plotted on the same graph for direct comparison in Figure 6.7. It is shown that all the predicted lateral load-deflection curves compare very well with the measured ones. Consequently, it is considered that the non-linear analysis is able to predict both the lateral load resistances and the overall deformation characteristic of the cold-formed steel beam-column sub-frames based on the local rotational characteristic of the connections.

6.5 Flexibility in bolted moment connections

In a typical beam-column connection investigated in the present project, the flexibility of the bolted moment connection is considered to arise from:

a) Bearing deformation around bolt holes in connected section webs of both column and beam members.
b) Clearances in bolt holes in both cold-formed steel sections and hot rolled steel gusset plates.

c) Flexural deformation in both cold-formed steel sections and hot-rolled steel gusset plates.

d) Slippage against friction between the washers and the connected parts of both cold-formed steel sections and hot-rolled steel sections, if any.

Both the bearing deformation around bolt holes and the shear deformation of the hot-rolled steel gusset plates under lateral loads are considered to be small, and thus neglected. In general, the contribution of each of the flexibility depends largely on the constructional details of the connections, such as frictional forces in the interfaces between steel sections and washers, and clamping forces developed in bolt shanks. The determination of the connection flexibility is generally very complicated, and semi-empirical design rules are always proposed for simplicity.

6.5.1 Semi-empirical design rule for connection stiffness

For the prediction of the connection flexibility of bolted moment connections between cold-formed steel sections, a semi-empirical design rule is proposed as follows:
\[
\left( \frac{1}{K} \right)_{con} = \left( \frac{1}{K_{con,cfs}} \right) + \left( \frac{1}{K_{con,hear}} \right) + \left( \frac{1}{K_{con,gp}} \right)
\]

where

\[
\left( \frac{1}{K} \right)_{con} = \text{flexibility of the connection}
\]

\[
\left( \frac{1}{K_{con,cfs}} \right) = \text{flexibility due to bending of cold-formed steel sections within the connection zone}
\]

\[
\left( \frac{1}{K_{con,hear}} \right) = \text{flexibility due to bearing of cold-formed steel sections around bolt holes}
\]

\[
\left( \frac{1}{K_{con,gp}} \right) = \text{flexibility due to bending of hot-rolled steel gusset plate within the connection zone}
\]

and

\[
K_{con,cfs} = \frac{3(EI)_{cfs}}{l_{con}}
\]

\[
K_{con,hear} = \left( \frac{M}{\theta_{hear}} \right) \quad \text{and} \quad \theta_{hear} = \frac{\Delta}{r}
\]

\[
K_{con,gp} = \frac{3(EI)_{gp}}{l_{con}}
\]

where

\((EI)_{cfs}, (EI)_{gp}\) are the flexural rigidities of the cold-formed steel section and the gusset plate respectively.
\( l_{con} \) is the length of the connection

\( r \) is the distance from the bolt group centre to the uttermost bolt, and

\( \Delta \) is the bearing deformation of the section web around bolt hole to be specified.

After calibration against test data, it is found the connection stiffness at both the initial and the non-linear stages of deformation may be obtained simply by setting the value of \( \Delta \) to 1 mm and 3 mm respectively. The connection stiffness at the final stage of deformation is taken as zero. Refer to Table 6.3 for details of the calculation.

6.5.2 Calibration against test data

In order to verify the accuracy of the proposed semi-empirical design rule, the connection stiffnesses of the six test specimens at both the initial and the non-linear stages of deformation are evaluated; they are illustrated in Figure 6.5 with the legend 'Model II' for direct comparison with the measured curves with the legend 'Model I'. Furthermore, the overall load deflection curves of the six beam-column frames are predicted using the predicted moment joint-rotation curves, and they are also plotted in the same graphs of the measured load-deflection curves of the test specimens with the legend of 'Model II' in Figure 6.7. It is shown that the predicted load-deflection
curves follow closely to those measured ones during the entire deformation history until failure.

The lateral load resistances of the beam-column sub-frames are also presented in Table 6.2 for comparison with the measured resistances. The ratios of the measured lateral load resistance to the predicted lateral load resistance of the six test specimens are also presented in Table 6.2. The model factors are found to range from 0.97 to 1.08, confirming the accuracy of the proposed design rule.

6.6 Conclusions

The results of a numerical investigation on bolted moment connections between cold-formed steel sections through non-linear analysis with semi-rigid connections is presented. Based on the measured rotational characteristics of the connections in six beam-column sub-frames, the overall structural behaviour of the beam-column sub-frames are predicted satisfactorily through non-linear analysis. Moreover, it is also shown that the predicted lateral load deflection curves of the beam-column sub-frames follow closely the measured load deflection curves along the entire deformation history until failure. Thus, the proposed non-linear analysis is confirmed to be structurally adequate for the analysis and design of cold-formed steel structures.
with bolted moment connections, provided that the local rotational characteristic of
the connections is used in the analysis.

A semi-empirical design rule is also presented for the determination of the moment
joint-rotation curves of the bolted moment connections. Calibration against test data is
also carried out with favourable comparison with the measured values.
a) Internal beam-column sub-frames

- All bolts are M8.8 φ16
- 16 mm thick gusset plate without chamfers
- 10 mm thick gusset plate with chamfers

(b) External beam-column sub-frames

- All bolts are M8.8 φ16
- 16 mm thick gusset plate without chamfers
- 10 mm thick gusset plate with chamfers

Figure 6.1 General test setup of cold-formed steel beam-column sub-frames.
Figure 6.2 Load deflection curves of beam-column sub-frames tests with flexural failure in connected section.
Figure 6.3 Measured and corrected joint rotation curves of beam-column sub-frames.
Figure 6.4a Details of measurement and calculation rotation for internal beam-column sub-frames.
Figure 6.4b  Details of measurement and calculation rotation for external beam-column sub-frames.
\[ \frac{\theta}{\theta_r} = \frac{M_{cen}}{M_R} = \frac{\theta}{[M_R/(EI/L)]]} \]

- \( M_{cen} \) = measured moment resistance of the connection
- \( M_R \) = measured moment capacity of cold-formed steel sections
- \( EI \) and \( L \) = flexural rigidity and member length of the connected section
- \( \theta \) = measured joint rotation obtained directly from test

Model I - flexural stiffness obtained directly from test
Model II - flexural stiffness obtained from semi-empirical design rules

Figure 6.5 Moment rotation curves for non-linear analysis
<table>
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<tr>
<th>Bolt pitch, S</th>
<th>Length of rigid link, S_r</th>
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<tr>
<td>240</td>
<td>380</td>
</tr>
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</table>

e: Length of rigid link S_r is calculated as follow

\[= 75 + 5 + 30 + 180 + 30 = 320 \text{ for } S = 180 \text{ mm}\]

\[= 75 + 5 + 30 + 240 + 30 = 380 \text{ for } S = 240 \text{ mm}\]

Figure 6.6 F.E. model with semi-rigid joints.
Figure 6.7 Load deflection curves of beam-column sub-frames
### Table 6.1 Summary of test program and test data

<table>
<thead>
<tr>
<th>Test</th>
<th>Section</th>
<th>Maximum applied force (kN)</th>
<th>Failure mode</th>
<th>Maximum moment resistance (kNm)</th>
<th>Moment resistance at 0.05 rad (kNm)</th>
<th>Initial stiffness</th>
<th>Member</th>
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</tr>
<tr>
<td>E180C2</td>
<td>C15020DS</td>
<td>11.05</td>
<td>FFcs</td>
<td>20.89</td>
<td>0.012</td>
<td>-</td>
<td>1.94</td>
<td>468</td>
<td>15.89</td>
</tr>
<tr>
<td>S180D1</td>
<td>C15020DS</td>
<td>21.47</td>
<td>FFcs</td>
<td>20.31</td>
<td>0.040</td>
<td>-</td>
<td>2.02</td>
<td>486</td>
<td>9.96</td>
</tr>
<tr>
<td>E180D1</td>
<td>C15020DS</td>
<td>11.35</td>
<td>FFcs</td>
<td>21.47</td>
<td>0.043</td>
<td>-</td>
<td>2.06</td>
<td>476</td>
<td>9.96</td>
</tr>
<tr>
<td>S240D1</td>
<td>C15020DS</td>
<td>22.91</td>
<td>FFcs</td>
<td>20.91</td>
<td>0.036</td>
<td>-</td>
<td>2.01</td>
<td>478</td>
<td>9.96</td>
</tr>
<tr>
<td>E240D1</td>
<td>C15020DS</td>
<td>10.43</td>
<td>FFcs</td>
<td>19.03</td>
<td>0.038</td>
<td>-</td>
<td>2.04</td>
<td>450</td>
<td>9.96</td>
</tr>
</tbody>
</table>

Notes:
- **S** denotes an internal beam-column sub-frame with a 'cross' shaped gusset plate under lateral load
- **E** denotes an external beam-column sub-frame with a 'lee' shaped gusset plate under lateral load
- **180** and **240** denote bolt pitches of 180 mm and 240 mm respectively
- **A** denote 4 bolts per member without chamfers in gusset plate
- **C** denote 4 bolts per member with chamfers in gusset plate
- **I** denotes the thickness of gusset plate at 10 mm
- **2** denotes the thickness of gusset plate at 16 mm
- **4** denotes the thickness of gusset plate at 6 mm

The measured moment capacity of C15020DS G450 is 21.36 kNm after normalised to design thickness and yield strength.

### Table 6.2 Summary of non-linear analysis results

<table>
<thead>
<tr>
<th>Test</th>
<th>Model I</th>
<th>Model II</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>P&lt;sub&gt;test&lt;/sub&gt;</td>
<td>P&lt;sub&gt;design&lt;/sub&gt;</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S180A2</td>
<td>20.94</td>
<td>20.30</td>
</tr>
<tr>
<td>E180C2</td>
<td>11.05</td>
<td>11.31</td>
</tr>
<tr>
<td>S180D1</td>
<td>21.47</td>
<td>20.61</td>
</tr>
<tr>
<td>E180D1</td>
<td>11.35</td>
<td>11.04</td>
</tr>
<tr>
<td>S240D1</td>
<td>22.91</td>
<td>20.16</td>
</tr>
<tr>
<td>E240D1</td>
<td>10.43</td>
<td>10.69</td>
</tr>
</tbody>
</table>

Notes:
- Model I - flexural stiffness obtained directly from test data
- Model II - flexural stiffness obtained from semi-empirical design rules
### Table 6.3a
**Calculation of the connection stiffness at initial stage and nonlinear stage**

**INPUT DATA**
- For: C15020DS G450  
- Test specimen: S180A2 & E180C2
- Maximum moment resistance = 21.36 kNm
- Moment in bolt group center = 19.224 kNm

**Material properties**

<table>
<thead>
<tr>
<th>CFS</th>
<th>Gusset plate</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E$ = 2.00E+08 kN/m²</td>
<td>$E$ = 2.05E+08 kN/m²</td>
</tr>
<tr>
<td>$I$ = 4.35E-06 m⁴</td>
<td>$I$ = 4.47E-06 m⁴</td>
</tr>
<tr>
<td>$L$ = 0.18 m</td>
<td>$L$ = 0.18 m</td>
</tr>
<tr>
<td>$r$ = 100.62 mm</td>
<td>$r$ = 100.62 mm</td>
</tr>
<tr>
<td>Initial stage</td>
<td>Nonlinear stage</td>
</tr>
<tr>
<td>$\Delta$ = 1 mm</td>
<td>$\Delta$ = 3 mm</td>
</tr>
</tbody>
</table>

**OUTPUT RESULTS**

<table>
<thead>
<tr>
<th>Initial stage</th>
<th>Nonlinear stage</th>
</tr>
</thead>
<tbody>
<tr>
<td>$K_{con,cfs}$ = 14500 kNm/rad</td>
<td>$K_{con}$ = 14500 kNm/rad</td>
</tr>
<tr>
<td>$K_{con,bear}$ = 1934 kNm/rad</td>
<td>$K_{con,bear}$ = 645 kNm/rad</td>
</tr>
<tr>
<td>$K_{con, gp}$ = 15273 kNm/rad</td>
<td>$K_{con, gp}$ = 15273 kNm/rad</td>
</tr>
<tr>
<td>$K_{con}$ = 1535 kNm/rad</td>
<td>$K_{con}$ = 593 kNm/rad</td>
</tr>
</tbody>
</table>

By equation

$$\frac{1}{K_{con}} = \frac{1}{K_{con,cfs}} + \frac{1}{K_{con,bear}} + \frac{1}{K_{con, gp}}$$

since

$$K_{con, cfs} = \frac{3EI}{L}$$

$$K_{con, bear} = \frac{M}{\theta} = \frac{M}{\Delta \cdot r}$$

$$K_{con, gp} = \frac{3EI}{L}$$
Table 6.3b

Calculation of the connection stiffness at initial stage and nonlinear stage

**INPUT DATA**

For: C15020DS G450
Maxmum moment resistance = Test specimen: S180D1 & E180D1
Moment in bolt group center = 21.36 kNm

**Material properties**

<table>
<thead>
<tr>
<th></th>
<th>CFS</th>
<th>Gusset plate</th>
</tr>
</thead>
<tbody>
<tr>
<td>(E)</td>
<td>2.00E+08 kN/m²</td>
<td>2.05E+08 kN/m²</td>
</tr>
<tr>
<td>(I)</td>
<td>4.35E-06 m⁴</td>
<td>6.64E-06 m⁴</td>
</tr>
<tr>
<td>(L)</td>
<td>0.18 m</td>
<td>0.18 m</td>
</tr>
<tr>
<td>(r)</td>
<td>100.62 mm</td>
<td></td>
</tr>
<tr>
<td>(\Delta)</td>
<td>1 mm</td>
<td>3 mm</td>
</tr>
</tbody>
</table>

**OUTPUT RESULTS**

<table>
<thead>
<tr>
<th></th>
<th>Initial stage</th>
<th>Nonlinear stage</th>
</tr>
</thead>
<tbody>
<tr>
<td>(K_{con,cfs})</td>
<td>14500 kNm/rad</td>
<td>14500 kNm/rad</td>
</tr>
<tr>
<td>(K_{con,bear})</td>
<td>1934 kNm/rad</td>
<td>645 kNm/rad</td>
</tr>
<tr>
<td>(K_{con, gp})</td>
<td>22687 kNm/rad</td>
<td>22687 kNm/rad</td>
</tr>
<tr>
<td>(K_{con})</td>
<td>1587 kNm/rad</td>
<td>601 kNm/rad</td>
</tr>
</tbody>
</table>

By equation

\[
\frac{1}{K_{con}} = \frac{1}{K_{con,cfs}} + \frac{1}{K_{con,bear}} + \frac{1}{K_{con, gp}}
\]

since

\[
K_{con, cfs} = \frac{3EI}{L}
\]

\[
K_{con, bear} = \frac{M}{\theta} = \frac{M}{\Delta/r}
\]

\[
K_{con, gp} = \frac{3EI}{L}
\]
Table 6.3c
Calculation of the connection stiffness at initial stage and nonlinear stage

**INPUT DATA**

For: *C* 15020DS G450  
Test specimen: S240D1 & E240D1
Maximum moment resistance = 21.36 kNm  
Moment in bolt group center = 19.224 kNm

**Material properties**

<table>
<thead>
<tr>
<th></th>
<th>CFS</th>
<th>Gusset plate</th>
</tr>
</thead>
<tbody>
<tr>
<td>E</td>
<td>2.00E+08 kN/m²</td>
<td>2.05E+08 kN/m²</td>
</tr>
<tr>
<td>I</td>
<td>4.35E-06 m⁴</td>
<td>6.64E-06 m⁴</td>
</tr>
<tr>
<td>L</td>
<td>0.24 m</td>
<td>0.24 m</td>
</tr>
<tr>
<td>r</td>
<td>128.16 mm</td>
<td></td>
</tr>
<tr>
<td>Δ</td>
<td>1 mm</td>
<td>3 mm</td>
</tr>
</tbody>
</table>

**OUTPUT RESULTS**

<table>
<thead>
<tr>
<th></th>
<th>Initial stage</th>
<th>Nonlinear stage</th>
</tr>
</thead>
<tbody>
<tr>
<td>K&lt;sub&gt;con,cfs&lt;/sub&gt;</td>
<td>10875 kNm/rad</td>
<td>10875 kNm/rad</td>
</tr>
<tr>
<td>K&lt;sub&gt;con,bear&lt;/sub&gt;</td>
<td>2464 kNm/rad</td>
<td>821 kNm/rad</td>
</tr>
<tr>
<td>K&lt;sub&gt;con, gp&lt;/sub&gt;</td>
<td>17015 kNm/rad</td>
<td>17015 kNm/rad</td>
</tr>
<tr>
<td>K&lt;sub&gt;con&lt;/sub&gt;</td>
<td>1797 kNm/rad</td>
<td>731 kNm/rad</td>
</tr>
</tbody>
</table>

By equation

\[
\frac{1}{K_{\text{con}}} = \frac{1}{K_{\text{con,cfs}}} + \frac{1}{K_{\text{con,bear}}} + \frac{1}{K_{\text{con, gp}}} 
\]

since

\[
K_{\text{con, cfs}} = \frac{3EI}{L} 
\]

\[
K_{\text{con, bear}} = \frac{M}{\theta} = \frac{M}{\Delta} 
\]

\[
K_{\text{con, gp}} = \frac{3EI}{L} 
\]
CHAPTER SEVEN

OVERALL COMMENTS AND CONCLUSIONS

After the detailed presentation of both the experimental and the numerical investigations on cold-formed steel beam-column sub-frames with bolted moment connections, the overall comments and the conclusions of the research project are presented as follows.

7.1 Structural performance of beam-column sub-frames with bolted moment connections

An extensive experimental investigation on cold-formed steel beam-column sub-frames with bolted moment connections was carried out with two lipped C-sections back-to-back according to the proposed basic configuration. Two test series with a total of sixteen internal and external beam-column sub-frames covering a wide range of connection configurations were examined under lateral loads. Moreover, a total of nine internal beam-column sub-frames with various connection configurations were tested under gravity loads. The results of Test series I, II and III may be found in Table 3.2, 4.2 and 5.2, respectively.
It is interesting to compare the moment resistance ratios of the connections in all the tested beam-column sub-frames, and to group them together according to their critical failure modes; Table 7.1 presents the summary of the comparison. It should be noted that:

a) A total of eleven connections with different configurations are able to develop moment resistances over 75% of the moment capacities of the connected sections with flexural failure in connected cold-formed steel sections; all of them are, thus, regarded as effective moment connections. However, in most cases the lateral deformations of the beam-column sub-frames under lateral load are significantly, and in some cases, excessive. Consequently, the proposed bolted moment connections are semi-rigid connections and high moment resistances may only be mobilized at large rotation.

b) The presence of 50 mm deep chamfers is found to be effective in gusset plates of 10 mm thick as their structural behaviour is similar to those gusset plates of 16 mm thick but without chamfers. In both cases, the moment resistance ratios of the connections are found to be over 90% with flexural failure in connected cold-formed steel sections.
c) There is little difference in the moment resistances of connections with bolt pitches of 180 mm and 240 mm. Thus, a bolt pitch of 180 mm may be considered to be effective for 150 mm deep lipped C-sections.

d) While the member rotations are measured with high accuracy, it should be cautious on the interpretation of the measured rotations. As the test specimens are relatively flexible, deformation of the test specimens are always significant before failure. Thus, the reference points for member rotations are moved continuously according to the lateral displacements of the test specimens since the transducers are stationary. Consequently, the measured member rotations may only be considered as an approximation to the actual member rotations of the test specimens.

Moreover, it is difficult to define precisely the centres of rotation of the members within the connections. Any measured rotations based on deflections of the section flanges may not necessarily correspond to rotations of the whole section in the presence of cross-sectional distortion, especially under high stress levels. In general, the error is considered to be small during the initial
deformation stage, and the maximum discrepancy in joint rotations is envisaged to be within 15% at large lateral displacements.

7.2 Relative performance on connections with different failure modes

Among all the tests, four different modes of failure are identified and the relative performance of bolted moment connections with different failure modes are presented as follows:

a) The bearing failure in section web around bolt hole, or Mode BFcsw, always occurs at low applied load, and thus this failure mode is unfavourable as the moment resistances of the connections are relatively low with large member rotation. Due to progressive yielding of material in section web under confined bearing, this mode of failure may be considered to be ductile.

b) For connections with thin gusset plates, lateral torsional buckling, or Mode LTBgp, may be critical and thus the moment resistances of the connections are also low. In practice, this failure mode depends largely on the unrestrained length of the gusset plates between the beam and the column members, and also the provision of external restraints to the connections. It should be noted that this failure mode always takes place in a sudden manner with quick unloading.
c) For connections with thick and effectively restrained gusset plates, two different failure modes may occur, depending on the relative moment capacities of the gusset plates and the connected cold-formed steel sections.

- For those gusset plates with limited moment capacities, flexural failure of the connected gusset plates, or Mode FFgp, is critical at the location subjected to the largest applied moment, i.e. at the corners of both 'cross' and 'tee' shaped gusset plates close to column members. In general, this failure mode is ductile with significant member rotation after failure. However, due to the presence of defects around the corners of gusset plates in some tests, premature failure with little ductility was observed.

- For those gusset plates with large moment capacities, flexural failure of the connected cold-formed steel sections, or Mode FFcs, is critical at the end of the gusset plates where the beam members are connected. Due to limited post-buckling strength in cold-formed steel sections after yielding, there is no apparent rotation capacity in the connection after failure.
Among all the failure modes observed in the tests, flexural failure of connected cold-formed steel sections is the most desirable one as the moment resistances of the connections are found to be very close to those of the connection sections.

7.3 Numerical modelling

Based on the test results of the cold-formed steel beam-column sub-frames, the measured rotational characteristics of bolted moment connections were adopted to investigate the overall structural behaviour of the sub-frames through non-linear analysis with semi-rigid connections. The measured moment-rotation curves of the bolted connections were inputted into a non-linear analysis software GMNAF as local connection characteristics to generate the overall lateral deflection history of the beam-column sub-frames for comparison with test data.

It was shown that the predicted lateral load deflection curves of the beam-column sub-frames with connections of high structural efficiency were found to follow closely the measured load deflection curves up to failure with a lateral deformation of 250 mm. Thus, it was confirmed that the proposed non-linear analysis was structurally adequate for the analysis and design of cold-formed steel structures with bolted moment
connections, provided that the local moment rotation characteristic of the connections was sufficiently adequate.

7.4 Conclusions

In order to establish moment framing for cold-formed steel sections in building applications, a bolted moment connection configuration for double lipped C sections back-to-back was proposed. A total of twenty five beam-column sub-frame tests were executed and four different modes of failure were identified. The proposed connection configuration with thin hot-rolled steel gusset plates but with chamfers was proven to be efficient in mobilizing high moment resistances. The moment resistances of the proposed connection configurations were found to range from below 50% to over 85% of the moment capacities of connected sections. However, lateral deformations in some beam-column sub-frames are significant and high moment resistance of the connections may only be mobilized at large rotations. Moreover, a non-linear analysis method incorporating semi-rigid joints is shown to be effective in the analysis of design of cold-formed steel beam-column frames with bolted moment connections.
Based on the findings of the experimental investigation, it is demonstrated that bolted moment connections between cold-formed steel sections are readily achieved with the proposed connection configurations. The bolted moment connections are shown to be effective in transmitting moment between the connected sections, enabling effective moment framing in cold-formed steel structures. Engineers are encouraged to build light-weight low to medium rise moment frames with cold-formed steel sections.
<table>
<thead>
<tr>
<th>Failure mode</th>
<th>Moment resistance ratio $\Psi$</th>
<th>Number of tests</th>
<th>Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>BFcsw</td>
<td>$&lt; 0.5$</td>
<td>2</td>
<td>S090A1 / S090A2</td>
</tr>
</tbody>
</table>
| LTBgp        | $0.5 \sim 0.9$                | 8               | S180D4 / E180D4  
               |                               |                 | S240D4 / E240D4  
               |                               |                 | G340D1L / G340D1S  
               |                               |                 | G340D4L / G440D4S  |
| FFgp         | $0.7 \sim 0.8$                | 4               | S180A1 / E180C1  
               |                               |                 | S240A1 / E240C1  |
| FFcs         | $0.77 \sim 0.99$              | 11              | S180A2 / E180C2  
               |                               |                 | S180D1 / E180D1  
               |                               |                 | S240D1 / E240D1  
               |                               |                 | G180A1S / G180A2S  
               |                               |                 | G180D1S / G180D4S  
               |                               |                 | G440A2L   |
LIST OF REFERENCES

Codes and Standards

2. ECCS - Technical Committee 7 - working Group 7.1 ‘Design of cold-formed steel sheetings and sections: European recommendations for the design of light gauge steel members’, first edition, 1987, R.4.2.2.
3. American Iron and Steel Institute, Load and resistance factor design specification for cold-formed steel structural members: LRFD Cold-formed steel design manual: Part 1, Washington DC, 1996.

Design Guides


Technical Papers


Appendix A  Published technical papers in international journals

A1. Structural behaviour of bolted moment connections in beam-column sub-frames
Structural Behaviour of Bolted Moment Connections in Cold-Formed Steel Beam-Column Sub-Frames

M.F. Wong & K.F. Chung

Department of Civil and Structural Engineering, The Hong Kong Polytechnic University, Hung Hom, Kowloon, Hong Kong, China.

Abstract

This paper presents an experimental investigation on bolted moment connections between cold-formed steel sections. A total of twenty column base connection tests and beam-column sub-frame tests with different connection configurations were carried out to assess the strength and stiffness of bolted moment connections between cold-formed steel sections. Among the tests, four different modes of failure were identified:

- Mode BFcs w Bearing failure in section web around bolt hole
- Mode LTBgp Lateral torsional buckling of gusset plate
- Mode FFgp Flexural failure of gusset plate
- Mode FFCs Flexural failure of connected cold-formed steel section

For those connections failed in Mode BFcs w, the moment resistances of the connections were typically found to be below 50% of the moment capacities of the connected sections. For those connections failed in Modes LTBgp and FFgp, the moment resistances of the connections were found to be about 60% and 75% of the moment capacities of the connected sections. Among all, the moment resistances of those connections failed in Mode FFCs were the highest with a minimum of 85% of the moment capacities of the connected sections.

Consequently, it is demonstrated that through rational design and construction, effective moment connections between cold-formed steel sections may be readily achieved. Engineers are encouraged to build light-weight low to medium rise moment frames with cold-formed steel sections.

I. Introduction

Cold-formed steel sections are light-weight materials and suitable for building construction owing to their high structural performance and durability. They are widely used as secondary members, such as purlins in roofs, joists of medium span in floors, studs in wall panels, storage racking in warehouses, and hoarding structures in construction sites. Since 1990, there is a growing trend to use cold-formed steel sections as primary structural members in building construction, such as low to medium rise residential houses and portal frames of modest span.
The most common cold-formed steel sections are lipped C-sections and lipped Z-sections, and the thickness typically ranges from 1.2 mm to 3.2 mm. Common yield strengths are 280 N/mm² and 350 N/mm². Moreover, there are a whole range of variants of these basic shapes, including sections with single and double lips, and sections with internal stiffeners. Due to the thinness of cold-formed steel sections, local buckling is a predominant consideration in assessing their section capacities. Furthermore, as they are very weak in torsion, torsional flexural buckling in columns and lateral torsional buckling in beams may be critical. There are a number of codes of practice [1-4] on the design of cold-formed steel structures together with complementary design guides and worked examples [5-8] to assist practising engineers.

In building construction, cold-formed steel sections are usually bolted to hot rolled steel plates or sections to form simple and moment connections. However, despite their simplicity, simple connections between cold-formed steel sections have received relatively little attention. An experimental experiment [9,10] was reported recently to study the structural performance on simple connections between cold-formed steel sections using web cleats of folded cold-formed steel strips. A complementary set of design rules was also provided in accordance with BS5950: Part 5 and Eurocode 3: Part 1.3.

Much research work has been reported in the literature on the development of moment connections between cold-formed steel purlins in modern roof systems. A number of different connection configurations [11] with sleeves or overlaps were found in various proprietary systems which offer partial continuity along the purlins. Cold-formed steel moment connections in column bases and also in beam-column connections were also tested and the proposed connection configurations were suitable for portal frame construction [12-16]. Besides experimental investigations on bolted connections between cold-formed steel strips [17,18], advanced finite element modelling using three-dimensional solid elements with material, geometrical and boundary non-linearities were also reported in the literature [19-22].

It should be noted that most of the design recommendations on connections among cold-formed steel sections concern the load carrying capacities of individual fasteners such as bolts, screws, rivets and spot welds. Little information on the structural performance of the bolted moment connections among cold-formed steel sections may be found in the literature. It is important to carry out physical tests to establish the use of moment connections between cold-formed steel sections so that efficient moment framing may be designed and built in building construction.

This paper presents the findings of an experimental investigation [23-25] on the structural performance of bolted moment connections between cold-formed steel sections. A preliminary test series on four column base connections was first carried out, and then a total of sixteen beam-column sub-frame tests in two test series with different connection configurations under lateral loads were performed. The investigation aims to assess the strength and the stiffness of connections with different configurations in practical member orientations in order to establish the structural efficiency of bolted moment connections between cold-formed steel sections for general building applications.

2. Basic configuration

The basic configuration of bolted moment connections proposed for cold-formed steel lipped C-sections in general building applications is:
• All structural members such as beams and columns are formed with two lipped C-sections back-to-back with interconnections at regular intervals.
• Moment connections between beams and columns are formed with hot-rolled steel gusset plates. In general, only the column members are continuous over the connections.
• Only the webs of lipped C-sections are bolted onto gusset plates for ease of buildability.
• Four bolts per member are used as a minimum configuration.

The connection details are rationalized after considering ease of fabrication and installation. In general, the proposed moment connections are not able to develop full moment capacity of the connected sections due to discontinuity of load paths along section flanges in the sections. Consequently, it is aimed to develop bolted moment connections with high structural efficiency and to mobilize at least 75% of the moment capacities of the connected sections.

3 Scope of investigation

The research project may be divided into two main parts of investigation:

• **Bolted moment connections in column base tests - Test Series A**
  This was a preliminary test series devised to establish the feasibility of the proposed basic configuration for bolted moment connections between cold-formed steel lipped C-sections. A total of four column base connections were carried out with different bolt pitches in order to compare the effectiveness of the connections. For simplicity, thick gusset plates were used in all tests in order to ensure failure in the connected sections.

• **Bolted moment connections in beam-column sub-frames - Test Series B1 and B2**
  In order to investigate bolted moment connections in typical building applications, test series B1 was devised in which five internal and three external beam-column sub-frames under lateral loads were carried out. Among the eight tests, the connection configurations were similar, but the thicknesses of the gusset plates and also the bolt pitches were varied systemically. It was envisaged that a number of failure modes in the gusset plates and also in the connected sections would be obtained for comparison.

  In order to improve the structural behaviour of the bolted moment connections in beam-column sub-frames, another test series B2 was carried out with engineered gusset plates. Four internal and four external beam-column sub-frames were tested.

  Two different section sizes of lipped C-sections in double sections back-to-back were used in the tests, namely, C15016DS and C15020DS; the nominal yield strength is 450 N/mm². The moment capacities of the sections measured from four point load tests were 16.95 kNm, and 21.36 kNm for C15016DS and C15020DS respectively. All bolts are 16mm in diameter and of Grade 8.8, they are all installed with a torque of 50 Nm.

4 Test program

The general arrangement of the test set-up for column base connections is shown in Figure 1 while that for beam-column sub-frames is shown in Figure 2. The connection configurations of all the tests are also illustrated in Figures 1 and 2 while key parameters of the connections are summarized in Table 1. All these test specimens were constructed according to the basic
configuration with systematic variations in the connection details, i.e. the bolt pitch, and the shape and thickness of gusset plates. Details of the test series are presented as follows:

4.1 Test Series A

A total of four column base connection tests were carried out as follows:

a) Three long columns in which the bolt pitches, S, of the connections are equal to 90 mm, 180 mm and 240 mm, and thus the tests are referred as Tests BC090L, BC180L and BC240L respectively.

b) One short column in which the bolt pitch of the connection is 180 mm, and thus the test is referred as Test BC180S.

The gusset plates are 20 mm thick and the nominal yield strength is 350 N/mm². Refer to Figure 1 for details of the connection configurations.

4.2 Test Series B1

A total of eight tests with three different connection configurations in both internal and external beam-column sub-frames were carried out as follows:

a) Three internal beam-column sub-frame tests in which the bolt pitches, S, of the connections are equal to 90 mm, 180 mm and 240 mm, and thus the tests are referred as Tests S090A1, S180A1 and S240A1 respectively. The gusset plates are 'cross' shaped with 10 mm thick.

b) Two external beam-column sub-frame tests in which the bolt pitches, S, of the connections are equal to 180 mm and 240 mm, and thus the tests are referred as Tests E180C1 and E240C1 respectively. The gusset plates are 'tee' shaped with 10 mm thick.

c) Two internal beam-column sub-frame tests in which the bolt pitches, S, of the connections are equal to 90 mm and 180 mm, and thus the tests are referred as Tests S090A2 and S180A2 respectively. The gusset plates are 'cross' shaped with 16 mm thick.

d) One external beam-column sub-frame test in which the bolt pitch, S, of the connection is equal to 180 mm, thus the test is referred as Test E180C2. The gusset plate is 'tee' shaped with 16 mm thick.

The nominal yield strength of the gusset plates is 350 N/mm². Refer to Figure 2 for details of the connection configurations.

4.3 Test Series B2

In order to enhance the structural performance of the connections, chamfers of 50 mm in depth are provided in all the gusset plates for improved structural efficiency. Furthermore, for gusset plates with different shapes and thicknesses, two bolt pitches of 180 mm and 240 mm are used for comparison. Consequently, another test series with a total of eight beam-column sub-frame tests in four different connection configurations with engineered gusset plates were carried out as follows:
a) Two internal beam-column sub-frame tests with 'cross' shaped gusset plates of 10 mm thick, namely, Tests S180D1 and S240D1 respectively.

b) Two external beam-column sub-frame tests with 'tee' shaped gusset plates of 10 mm thick, namely, Tests E180D1 and E240D1 respectively.

c) Two internal beam-column sub-frame tests with 'cross' shaped gusset plates of 6 mm thick, namely, Tests S180D4 and S240D4 respectively.

d) Two external beam-column sub-frame tests with 'tee' shaped gusset plates of 6 mm thick, namely, Tests E180D4 and E240D4 respectively.

The nominal yield strength of the gusset plates is 275 N/mm². Refer to Figure 2 for details of the connection configurations.

4.4 Test procedures

In all tests, the applied load and the displacements of each member of the test specimens were measured during the entire deformation history. In order to assess end rotations of the members, two transducers were used at each location of interest, and member end rotations were obtained as the relative displacements divided by the separation between the transducers. For beam-column tests, any out-of-plane deformation of the test specimens was prohibited by two sets of restraining rollers.

In general, the tests were terminated when excessive deformation, section failure or member buckling occurred. In most cases, a pre-load of 2 kN was applied to the test specimens before testing in order to ensure that all bolts were in contact with the section webs of connected sections despite all the bolt holes were 'perfect-fitted' to 16 mm diameter bolts.

5 Test results

Table 1 summaries the results of Test Series A, B1 and B2 together with the measured thickness and yield strength of both the cold-formed steel sections and the gusset plates. Furthermore, all the measured moment resistances are evaluated at failure positions and normalized with the ratio of design yield strength and design thickness to measured yield strength and thickness of the test specimens. For test specimens with excessive deformation under testing, the moment resistances of the connections are restricted to be the applied moment at a connection rotation of 0.05 radian.

Among all twenty tests, four different modes of failure were identified as follows:
- Mode BFcsw Bearing failure in section web around bolt hole, as shown in Figure 3
- Mode LTbgp Lateral torsional buckling of gusset plate, as shown in Figure 4
- Mode FFgp Flexural failure of connected gusset plate
- Mode FFcs Flexural failure of connected cold-formed steel section, as shown in Figure 5

Figure 6 illustrates the locations of critical sections for various failure modes in the connections. The measured load-deflection curves of test specimens in Test Series A, B1 and B2
are presented in Figures 7, 8 and 9 respectively, and they are plotted on the same graph for each test series for easy comparison.

In order to allow direct comparison, the applied moments for all of the test specimens are normalized with respect to the measured moment capacities of the connected sections. After evaluation of member rotations, joint rotations are then obtained as the difference of the member end rotations of those connected members. For internal beam-column sub-frames, four different joint rotations may also be evaluated from the connections, depending on the calculation. Similarly, two different joint rotations may be obtained for external beam-column sub-frames. Typical moment-rotation curves of all the test specimens in Test Series A, B1 and B2 are presented in Figures 10, 11 and 12 respectively.

5.1 Moment resistance ratios

In order to assess the effectiveness of the bolted moment connections, a moment resistance ratio, \( \psi \), is established which is defined as follows:

\[
\psi = \frac{\text{Measured moment resistance of a connection}}{\text{Measured moment capacity of connected section}}
\]

For connections with \( \psi \) equal to unity, the connections are shown to be capable in mobilizing the full moment resistances of the connected sections. For connections with \( \psi \) less than 0.50, the connections are regarded to be inefficient in transmitting moment across. All the moment resistances of the connections are evaluated at the failure positions of the connections, and thus they may be used directly in connection design. The results of all the tests are compared among each other and the findings are presented in the following sub-sections.

5.2 Test Series A

a) Tests BC090L, BC180L and BC240L

In test BC090L, significant bearing deformation was observed in the bolt holes of the section webs due to high moment acting at small lever arms. In both tests BC180L and BC240L, flexural failure in connected sections was apparent. The moment resistance ratios, \( \psi \), at the failure positions of the connections were found to be 0.75, 0.83 and 0.92 in tests BC090L, BC180L and BC240L respectively.

b) Tests BC180L and BC180S

In both tests, flexural failure in connected sections was apparent. The moment resistance ratios, \( \psi \), at the failure positions of the connections were found to be 0.83 and 0.71 in tests BC180L and BC180S respectively.

c) Consequently, it is shown that for connections with small bolt pitches, the structural efficiency of the connections is low with bearing failure in section web around bolt holes. For connections with large bolt pitches, flexural failure of connected cold-formed steel sections is critical, and the corresponding moment resistance ratio of the connections, \( \psi \), is at least 0.83. This is regarded as a favourable mode of failure with high structural efficiency.
5.3 Test Series B1

a) Tests S090A1, S180A1 and S240A1
In test S090A1, significant bearing deformation was observed in the bolt holes of the section webs due to high moment acting at small lever arms. In both tests S180A1 and S240A1, gross bending deformation was apparent in the hot-rolled steel gusset plates. The gusset plates failed pre-maturely in flexure due to the presence of defects at the corners of the gusset plates. The moment resistance ratios, Ψ, at the failure positions of the connections were found to be 0.36, 0.69 and 0.74 in tests S090A1, S180A1 and S240A1 respectively.

b) Tests E180C1 and E240C1
In both tests, gross bending deformation was apparent in the hot-rolled steel gusset plates, and the gusset plates failed pre-maturely in flexure due to the presence of defects at the corners of the gusset plates. The moment resistance ratios, Ψ, at the failure positions of the connections were found to be 0.84 and 0.88 in tests E180C1 and E240C1 respectively.

c) Tests S090A2, S180A2 and E180C2
In test S090A2, significant deformation was observed in the bolt holes of section webs due to high moment acting at small lever arms. For both tests S180A2 and S240A2, flexural failure in connected cold-formed steel sections was apparent. The moment resistance ratios, Ψ, at the failure positions of the connections were found to be 0.57, 0.92 and 0.97 in tests S090A2, S180A2 and E180C2 respectively.

d) Comparison among tests S090A1, S180A1 and S240A1 shows that for internal beam-column sub-frames with 10 mm thick gusset plates, the connection resistance ratios are increased from 0.36, 0.69 and 0.74 when the bolt pitch is increased from 90 mm, to 180 mm and then to 240 mm. Furthermore, comparison among tests E180C1 and E240C1 also shows that for external beam-column sub-frames with 10 mm thick gusset plates, the connection resistance ratios are also increased when larger bolt pitch is used.

It is thus demonstrated that connections with large bolt pitch always have higher moment resistances. Moreover, with an increase in the moment resistances, flexural failure in gusset plates becomes more critical than bearing failure in connected section web.

e) By increasing the thickness of gusset plates from 10 mm to 16 mm, the moment resistance ratios of the proposed connection configurations are found to be increased from

- 0.36 to 0.57 as shown in tests S090A1 and S090A2,
- 0.69 to 0.92 as shown in tests S180A1 and S180A2, and
- 0.84 to 0.97 as shown in tests E180C1 and E180C2.

This shows that thicker gusset plates always give higher moment resistances. Moreover, with an increase in the moment resistances of the gusset plates, flexural failure in the connected cold-formed steel sections becomes critical instead of flexural failure of the gusset plates.

The maximum moment resistance of the proposed connection configuration is found to be over 90% of the moment capacities of the connected sections, demonstrating that the proposed connection configuration is effective in transferring moment across the connected
members. As small bolt pitch is found to give connections of low moment resistance and large member rotation, it is thus recommended that the minimum value of bolt pitch for moment connections should not be less than 150 mm and 1.0 times section depth.

5.4 Test Series B2:

a) Tests S180D1 and E180D1
For both tests, flexural failure in connected sections was apparent. The moment resistance ratios, Ψ, at the failure positions of the connections were found to be 0.87 and 0.92 in tests S180D1 and E180D1 respectively.

b) Tests S180D4 and E180D4
In both tests, lateral torsional buckling in connected hot-rolled steel gusset plate was apparent. The moment resistance ratios, Ψ, at the failure positions of the connections were found to be 0.63 and 0.64 in tests S180D4 and E180D4 respectively.

c) Tests S240D1 and E240D1
For both tests, flexural failure in connected cold-formed steel sections was apparent. The moment resistance ratios, Ψ, at the failure positions of the connections were found to be 0.92 and 0.87 in tests S240D1 and E240D1 respectively.

d) Tests S240D4 and E240D4
In both tests, lateral torsional buckling in connected hot-rolled steel gusset plate was apparent. The moment resistance ratios, Ψ, at the failure positions of the connections were found to be 0.67 and 0.62 in tests S240D4 and E240D4 respectively.

e) For connections with thick gusset plates, it is shown that flexural failure of connected cold-formed steel sections is always critical, and the corresponding moment resistance ratio of the connections, Ψ, is at least 0.87. This is regarded as a favourable mode of failure with high structural efficiency.

For connections with thin gusset plates, lateral torsional buckling of hot rolled steel gusset plates is always critical, and the corresponding moment resistance ratio, Ψ, of the connections is about 0.60. This failure mode is regarded to be inefficient with low structural efficiency.

f) For connections with similar configurations but with different bolt pitches, it is shown that there is little difference in the moment resistance ratios. Thus, the bolt pitch of 180 mm may be considered to be effective in forming moment connections for 150 mm deep lipped C-sections.

6 Relative performance on connections with different failure modes

a) The bearing failure in section web around bolt hole, or Mode BFcsw, always occurs at low applied load, and thus this failure mode is unfavourable as the moment resistances of the connections are relatively low with large member rotation. Due to progressive yielding of material in section web under confined bearing, this mode of failure may be considered to be ductile.
b) For connections with thin gusset plates, lateral torsional buckling, or Mode LTBgp, may be critical and thus the moment resistances of the connections are also low. In practice, this failure mode depends largely on the unrestrained length of the gusset plates between the beam and the column members, and also the provision of external restraints to the connections. It should be noted that this failure mode always takes place in a sudden manner with quick unloading.

c) For connections with thick and effectively restrained gusset plates, two different failure modes may occur, depending on the relative moment capacities of the gusset plates and the connected cold-formed steel sections.

   • For those gusset plates with limited moment capacities, flexural failure of the connected gusset plates, or Mode FFgp, is critical at the location subjected to the largest applied moment, i.e. at the corners of both 'cross' and 'tee' shaped gusset plates close to column members. In general, this failure mode is ductile with significant member rotation after failure. However, due to the presence of defects around the corners of gusset plates in some tests, premature failure with little ductility was observed.

   • For those gusset plates with large moment capacities, flexural failure of the connected cold-formed steel sections, or Mode FFcs, is critical at the end of the gusset plates where the beam members are connected. Due to limited post-buckling strength in cold-formed steel sections after yielding, there is no apparent rotation capacity in the connection after failure.

d) Among all the failure modes observed in the tests, flexural failure of connected cold-formed steel sections is the most desirable one as the moment resistances of the connections are found to be very close to those of the connection sections.

7 Overall comments

It is interesting to compare the moment resistance ratios of all the twenty connections and to group them together according to their critical failure modes; Table 2 presents the summary of the comparison. It should be noted that:

a) A total of ten connections with different configurations are able to develop moment resistances over 75% of the moment capacities of the connected sections with flexural failure in connected cold-formed steel sections; all of them are, thus, regarded as effective moment connections.

b) The presence of 50 mm deep chamfers is found to be effective in gusset plates of 10 mm thick as their structural behaviour is similar to those gusset plates of 16 mm thick but without chamfers. In both cases, the moment resistance ratios of the connections are found to be over 90% with flexural failure in connected cold-formed steel sections.

c) There is little difference in the moment resistances of connections with bolt pitches of 180 mm and 240 mm. Thus, a bolt pitch of 180 mm may be considered to be effective for 150 mm deep lipped C-sections.

d) While the member rotations are measured with high accuracy, it should be cautious on the interpretation of the measured rotations. As the test specimens are relatively flexible,
deformation of the test specimens are always significant before failure. Thus, the reference points for member rotations are moved continuously according to the lateral displacements of the test specimens since the transducers are stationary. Consequently, the measured member rotations may only be considered as an approximation to the actual member rotations of the test specimens.

Moreover, it is difficult to define precisely the centres of rotation of the members within the connections. Any measured rotations based on deflections of the section flanges may not necessarily correspond to rotations of the whole section in the presence of cross-sectional distortion, especially under high stress levels. In general, the error is considered to be small during the initial deformation stage, and the maximum discrepancy in joint rotations is envisaged to be within 15% at large lateral displacements.

8 Conclusions

In order to establish moment framing for cold-formed steel sections in building applications, a bolted moment connection configuration for double lipped C sections back-to-back was proposed. A total of twenty column base connections and beam-column sub-frames tests were executed and four different modes of failure were identified. The moment resistances of the proposed connection configurations were found to range from below 50% to over 85% of the moment capacities of connected sections.

Based on the findings of the experimental investigation, it is demonstrated that bolted moment connections between cold-formed steel sections are readily achieved with the proposed connection configurations. The bolted moment connections are shown to be effective in transmitting moment between the connected sections, enabling effective moment framing in cold-formed steel structures. Engineers are encouraged to build light-weight low to medium rise moment frames with cold-formed steel sections.

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<table>
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<tr>
<th>Test</th>
<th>Section</th>
<th>Maximum applied force (kN)</th>
<th>Failure mode</th>
<th>Maximum moment resistance</th>
<th>Moment resistance at 0.05 rad (kN)</th>
<th>Initial stiffness</th>
<th>Member</th>
<th>Gusset plate</th>
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Notes:  
BC denotes a column-base connection with a 'tee' shaped gusset plate under lateral load  
S denotes an internal beam-column sub-frame with a 'cross' shaped gusset plate under lateral load  
F denotes an external beam-column sub-frame with a 'tee' shaped gusset plate under lateral load  
90, 180, 240 denote bolt pitches of 90 mm, 180 mm and 240 mm respectively  
A, C denote 4 bolts per member without chamfers in gusset plate  
D denotes 4 bolts per member with chamfers in gusset plate  
1 denotes the thickness of gusset plate at 10 mm  
2 denotes the thickness of gusset plate at 16 mm  
4 denotes the thickness of gusset plate at 6 mm  
L, S denote a long column of height 1500 mm and a short column of height 750 mm respectively  
* denotes connections with high structural efficiency.  
+ denotes pre-mature flexural failure at sections with defective cracks.  
The measured moment capacities of C15016DS G450 and C15020DS G450 are 16.95 kNm and 21.36 kNm respectively after normalised to respective design thickness and yield strength.
Table 2  Structural performance of bolted moment connections

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<th>Failure mode</th>
<th>Moment resistance ratio $\Psi$</th>
<th>Number of tests</th>
<th>Test</th>
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<td>4</td>
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Column-base connections

Test Series A

- Wooden blocks
- Applied load
- Failure position
- Interconnection plates
- Connection under investigation

S = 090
180
240

Hot rolled steel T section

Anchor bolts

All bolts are M 8.8 φ16

<table>
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<th>Test specimen</th>
<th>Height, H (mm)</th>
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<tr>
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<td>1215</td>
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<tr>
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<tr>
<td>BC240L</td>
<td>1500</td>
<td>1155</td>
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</tbody>
</table>

Figure 1  General test setup of cold-formed steel column-base connections
Internal beam-column sub-frames

Lateral Load
Applied load
Connection under investigation

S = 090 180 240
Test Series B1
Test Series B2

Gusset plate without chamfers
Gusset plate with chamfers

All Bolts are M8.8 φ16

External beam-column sub-frames

Lateral Load
Applied load
Connection under investigation

S = 090 180 240
Test Series B1
Test Series B2

Gusset plate without chamfers
Gusset plate with chamfers

All Bolts are M8.8 φ16

Figure 2 General test setup of cold-formed steel beam-column sub-frames.
Figure 3  Mode BFCsw  Bearing failure in section web around bolt holes.
4a Mode FFcs  Flexural failure of connected section in a column-base connection test.

4b Mode FFcs  Flexural failure of connected cold-formed steel section in an external beam-column sub-frame test.
5a Mode LTBgp  Lateral torsional buckling of connected gusset plate in an internal beam-column sub-frame test.

5b Mode LTBgp  Lateral torsional buckling of connected gusset plate in an external beam-column sub-frame test.
Figure 6 Locations of critical sections in Test Series B1 and B2.
Figure 7  Load deflection curves of column-base connections.
Test Series B1

Figure 8a  Load deflection curves of internal beam-column sub-frames.

Test Series B1

Figure 8b  Load deflection curves of external beam-column sub-frames.
Test Series B2

Figure 9a  Load deflection curves of beam-column sub-frames tests with 10 mm thick gusset plates.

Test Series B2

Figure 9b  Load deflection curves of beam-column sub-frames tests with 6 mm thick gusset plates.
Test Series A

![Graph showing moment rotation curves of column-base connections.](image)

Figure 10 Typical moment rotation curves of column-base connections.
Figure 11a  Typical moment rotation curves of internal beam-column sub-frames.

Figure 11b  Typical moment rotation curves of external beam-column sub-frames.
Test Series B2

![Graph](image1)

Figure 12a Typical moment rotation curves of beam-column sub-frames tests with 10 mm thick gusset plates.

Test Series B2

![Graph](image2)

Figure 12b Typical moment rotation curves of beam-column sub-frames tests with 6 mm thick gusset plates.
Appendix A  Published technical papers in international journals

A2. Non-linear analysis with semi-rigid connections in cold-formed steel beam-column sub-frames
Non-linear Analysis with Semi-Rigid Connections in Cold-Formed Steel Beam-Column Sub-Frames

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Abstract

This paper presents a numerical investigation on bolted moment connections between cold-formed steel sections through non-linear analysis with semi-rigid connections. An extensive experimental investigation on bolted moment connections between cold-formed steel sections was carried out, and a total of sixteen internal and external beam-column sub-frames with various connection configurations were tested under lateral loads. It is found that for connections with large bolt pitches and thick gusset plates, flexural failure of the connected sections is always critical, and the moment resistances attain at least 85% of the moment capacities of the connected sections.

Based on the measured rotation characteristics of the connections in six beam-column sub-frames, the overall structural behaviour of the beam-column sub-frames were predicted through non-linear analysis; the effect of semi-rigid joints is fully incorporated. It is shown that the predicted lateral load deflection curves of the beam-column sub-frames are found to follow closely the measured load deflection curves. Thus, it is confirmed that the proposed non-linear analysis is structurally adequate for the analysis and design of cold-formed steel structures with bolted moment connections, provided that the local moment rotation characteristic of the connections is sufficiently adequate.

Consequently, it is demonstrated that through rational design and construction, effective moment connections between cold-formed steel sections may be readily achieved. Engineers are encouraged to build light-weight low to medium rise moment frames with cold-formed steel sections.

Keywords: Cold-formed steel, bolted moment connections, non-linear analysis, semi-rigid joints, connection flexibility

1. Introduction

Cold-formed steel sections are light-weight materials and suitable for building construction owing to their high structural performance. Conventionally, they are used as purlins and side rails in the building envelopes of industrial buildings. The most common sections are lipped C-sections and lipped Z-sections, and the thickness typically ranges from 1.2 mm to 3.2 mm. Common yield strengths are 280 N/mm² and 350 N/mm². For general applications of cold-formed steel sections, there are a number of codes of practice available in the literature together with complementary design guides and worked examples.

Since 1990, there is a growing trend to use cold-formed steel sections as primary structural members in building construction, such as low to medium rise residential houses and portal frames of modest span. In order to extend the usage of cold-formed steel sections in building construction, and it is highly desirable to develop efficient moment connections in cold-formed steel sections for efficiency structural framing. Cold-formed steel moment connections in both column bases and beam-column connections were tested; the proposed
connection configurations were suitable for portal frame construction \cite{12-16}. Besides experimental investigations on bolted connections between cold-formed steel sections \cite{17-18}, advanced finite element modelling using three-dimensional solid elements with material, geometrical and boundary non-linearities were also reported in the literature \cite{19-22}.

Traditional analysis and design of hot-rolled steel structures usually idealize the connections to two extreme cases, namely, a) fully rigid connections, and b) pinned connections. While the assumption allows simple design of connections in hot-rolled steel sections, in reality, the actual moment-rotation characteristics in practical connections fall between these two extreme cases, i.e. semi-rigid connections. A large amount of research effort has been dedicated to the effect of semi-rigid connections on the overall behaviour of hot-rolled steel structures for the last thirty years. However, there is little design guidance on the use of semi-rigid connections in cold-formed steel structures. As cold-formed steel structures tend to be very slender when compared with typical hot rolled steel structures, it is expected that deformations in cold-formed steel structures are generally significant. Thus, non-linear effect in cold-formed steel structures is important, and any secondary moment induced by both global and local deformation of structures should be considered in assessing the overall behaviour of cold-formed steel structures.

Consequently, it is important to extend the investigation of semi-rigid connections from hot-rolled steel structures into cold-formed steel structures. The effect of the local rotational characteristics of bolted moment connections on the overall behaviour of cold-formed steel structures will be examined in details.

2. Scope of work

This paper presents a numerical investigation on cold-formed steel beam-column sub-frames under lateral load, and the structural behaviour of the beam-column sub-frames is investigated through the use of an established non-linear analysis, namely, . The effect of semi-rigid connections in the beam-column sub-frames is fully incorporated. For the present investigation, test data of six test specimens are selected from an extensive test programme on cold-formed steel beam-column sub-frames conducted by the first and the second authors; all the six tests failed in the flexural failure of connected sections. For details of the non-linear analysis, refer to the literature published by the third author.

The main objectives of the present investigation are:

a) To establish the accuracy of the proposed non-linear analysis for cold-formed steel beam-column sub-frames with bolted moment connections.

b) To confirm the use of local rotational characteristic of moment connections measured from tests to predict the overall structural behaviour of beam-column sub-frames.

c) To establish the derivation of rotational characteristic of moment connections from both test data and semi-empirical design rules.

For each test, the measured moment joint-rotation curve of the connections is inputted into the non-linear analysis software as the local deformation characteristic of the connections in order to generate the overall lateral deflection history of the beam-column sub-frame. Comparison on the predicted results with measured data is also presented. Moreover, the derivation of the rotational characteristic of the bolted moment connections from measured test data to moment
joint-rotation curves was fully described. Finally, a simple design rule for the prediction of the moment joint-rotation curves of the connections is proposed after calibrating against measured data.  

3. Bolted moment connections with high efficiency

In order to enable moment framing for cold-formed steel sections in building construction, a bolted moment connection configuration for two lipped C sections back-to-back is proposed as follows:

- All structural members such as beams and columns are formed with two lipped C-sections back-to-back with interconnections at regular intervals.
- Moment connections between beams and columns are formed with hot-rolled steel gusset plates. In general, only the column members are continuous over the connections.
- Only the webs of the lipped C-sections are bolted onto the gusset plates.
- Four bolts per member are used as a minimum configuration.

The connection details are rationalized after considering ease of fabrication and installation. In general, the proposed moment connections are not able to develop full moment capacity of the connected sections due to discontinuity of load paths along section flanges in the sections.

An extensive experimental investigation on bolted moment connections between cold-formed steel sections was carried out with two lipped C-sections back-to-back according to the proposed basic configuration. A total of sixteen internal and external beam-column sub-frames with a wide range of connection configurations were examined under lateral loads, and four different modes of failure were identified. The general layout of those tests is illustrated in Figure 1 together with details of the connection configurations. Details of the rotation measurements for both internal and the external beam-column sub-frames are illustrated in Figure 2 and 3 respectively.

In general, the moment resistances of the proposed connection configurations were found to range from 50% to over 95% of the moment capacities of connected sections. Furthermore, it is demonstrated that the most effective connections are those connections with large bolt pitches and thick gusset plates, and thus, the critical mode of failure is the flexural failure in connected sections. Out of the sixteen tests, six of them are found to fail in this mode of failure with high structural efficiency, and all of them can safely mobilize at least 80% of the moment capacities of the connected sections. The connection details of the six tests are also presented in Figure 1 while their test results are summarized in Table 1. Consequently, it is established that effective bolted moment connections between cold-formed steel sections are readily achieved through the use of the proposed connection configurations.

4. Evaluation of rotational characteristics of bolted connections

The load deflection curves of the six cold-formed steel beam-column frames with bolted moment connections of high structural efficiency are plotted in Figure 4. Owing to the member configurations in internal beam-column sub-frames, four different moment joint-rotation curves, or M - θ curves, may be derived, and such four curves for Test S180D1 are plotted in Figure 5a). The joint-rotations of the connection are the relative rotations between the column members and the beam members, and the calculation is also presented in Figure 2. Similarly, for external beam-column sub-frames, two different moment joint-rotation curves may be derived, and such two curves for Test E180D1 are plotted in Figure 5b).
In general, while resistance measurements in tests are always considered to be adequately accurate, errors are always present in deformation measurements. For each beam-column sub-frame considered in the present study, the joint-rotations of the connections were obtained as the relative rotations between the connected beam and column members, each in turn obtained as the relative lateral displacements of section flanges of the connected sections within the connection zones. While the relative lateral displacements of each member were measured precisely using transducers, it was difficult to define precisely the centres of rotation of each member within the connection zones. Due to the presence of cross-sectional distortion, any measured rotation based on the deflection of the section flange did not necessarily correspond to the rotation of the whole section. In general, this error was considered to be small during the initial deformation stage, but might become significant under high applied loads at large deformation.

Moreover, as the beam-column sub-frames were flexible, significant lateral deformation was observed in the tests, as shown in Figure 4. As the transducers were stationary, the reference points for member rotations were moved continuously according to the lateral displacements of the test specimens, as shown in both Figures 2 and 3. Consequently, there was a growing error embedded in the rotation measurements which was directly proportional to the lateral displacement of the test specimens. Consequently, correction to the rotation measurements is necessary.

In order to derive the corrected moment joint-rotation curves of bolted moment connections in cold-formed steel beam-column sub-frames in a simple and yet conservative manner, a correction procedure is suggested as follows:

**Step 1**
An average moment joint-rotation curve of the beam-column connection is first obtained by averaging the measured rotational characteristic curves obtained directly from tests. For internal beam-column sub-frames, all four curves should be used in the averaging whilst only two curves for external beam-column sub-frames.

**Step 2**
Assuming constant curvatures within the connection zone, a reduction factor $\chi$ is applied to the member rotations of the beam members; the reduction factor $\chi$ is defined as follows:

$$\chi = \frac{l_{bc} - \Delta}{l_{bc}}$$

where $l_{bc}$ = distance between the centre-line of the beam-column connection and the bolt group centre

$\Delta$ = lateral displacement of the centre-line of the beam-column connection measured from tests

The corrected moment joint-rotation curves of Tests S180D1 and E180D1 are plotted in the same graphs of the measured moment-rotation curves for direct comparison, as shown in Figure 5. It is shown that the corrected curves follow closely to the measured curves up to two-third of the connection resistances. After that, the slopes of the corrected curves are reduced by 10 to 20% when compared with those measured curves. Consequently, it is important to use the corrected moment joint-rotation curves in determining the overall structural behaviour of cold-formed steel beam-column sub-frames at large deformation.
In order to re-present the corrected moment joint-rotation curves of the connections for nonlinear analysis with semi-rigid joints, each moment joint-rotation curve is normalized in terms of $m_r$ and $\theta_r$, where $m_r$ is the moment ratio of the connection, and $\theta_r$ is the normalized connection rotation. The moment ratio, $m_r$, is equal to the measured moment resistance of the connection, $M_{con}$, divided by the moment capacity of cold-formed steel sections, $M_R$. The normalized connection rotation, $\theta_r$, is equal to the measured joint-rotation, $\theta$, divided by the rotation parameter $M_R / (EI / L)$, where $EI$ and $L$ are the flexural rigidity and the member length of the connected section. All the normalized moment joint-rotation curves, $m_r - \theta_r$ curves, are illustrated in Figure 6.

It is shown that the curves may be divided into three parts or stages of deformation, namely, the initial stage, the non-linear stage, and the final stage of deformation. While the initial stage represents the linear elastic rotation of the connections, the non-linear stage represents the flexibility of the connections primarily due to local bearing deformation of section web around bolt holes. The final stage is a flat line with zero slope, i.e. zero stiffness, representing flexural failure of the connected sections.

5 Non-linear analysis for semi-rigid connections

In order to examine the overall structural behaviour of the cold-formed steel beam-column sub-frames under lateral loads, non-linear analyses incorporating semi-rigid connections were carried out on the six cold-formed steel beam-column sub-frames. The finite element model is shown in Figure 7. All the beam and the column members are two lipped C sections back-to-back with a section depth of 150 mm and a nominal thickness of 2.0 mm, or C15020DS. The nominal yield strength is 450 N/mm². Moreover, the bolted connections are modelled as a rigid link of finite length, $S_r$, which attaches at one end rigidly to the column member, while in a semi-rigid manner to the beam members at the other end. As the moment joint-rotation curves of the connections are derived in such a way that all flexibilities between the beam and the column members have been fully incorporated into the rotational characteristic of the semi-rigid joints, the use of the rigid link is justified. The rotational characteristics of the semi-rigid connections are described by the normalized moment joint-rotation curves, $m_r - \theta_r$ curves, as shown in Figure 6. Based on the dimensions of the test specimens and also the normalized moment joint-rotation curves of the connections, the lateral load resistances of the beam-column sub-frames were predicted with the non-linear analysis; the results of all the six test specimens are summarized in Table 2. The measured lateral load resistances of the beam-column sub-frames are also presented in Table 2 for comparison.

In order to establish the adequacy of the analysis method, a model factor is established and defined as follows:

$$\psi = \frac{\text{Measured lateral load resistance from test}}{\text{Lateral load resistance obtained from non-linear analysis}}$$

A model factor of unity suggests that the result obtained from the non-linear analysis is conservative, and in general, the value of the model factor is expected to range from 1.1 to 1.5 for structural adequacy and economy. It is shown that the model factors for the six tests range from 0.98 to 1.14 with an average value of 1.04.

Moreover, the predicted lateral load-deflection curves of the six tests are illustrated in Figure 8 together with the measured lateral load-deflection curves for comparison. It is shown that
all the predicted lateral load-deflection curves compare very well with the measured ones. Consequently, it is considered that the non-linear analysis is able to predict both the lateral load resistances and the global deformation characteristic of the cold-formed steel beam-column sub-frames based on the local rotational characteristic of the connections.

6 Flexibility in bolted moment connections

In a typical beam-column connection investigated in the present project, the flexibility of the bolted moment connection is considered to arise from:

a) Bearing deformation around bolt holes in connected section webs of both column and beam members.
b) Clearances in bolt holes in both cold-formed steel sections and hot rolled steel gusset plates.
c) Flexural deformation in both cold-formed steel sections and hot-rolled steel gusset plates.
d) Slippage against friction between the washers and the connected parts of both cold-formed steel sections and hot-rolled steel sections, if any.

Both the bearing deformation around bolt holes and the shear deformation of the hot-rolled steel gusset plates under lateral loads are considered to be small, and thus neglected. In general, the contribution of each of the flexibility depends largely on the constructional details of the connections, such as frictional forces in the interfaces between steel sections and washers, and clamping forces developed in bolt shanks. The determination of the connection flexibility is generally very complicated, and a semi-empirical design rule for the prediction of the connection flexibility of bolted moment connections between cold-formed steel sections is proposed as follows:

\[
\left( \frac{1}{K} \right)_{con} = \left( \frac{1}{K_{con,cfs}} \right) + \left( \frac{1}{K_{con,bear}} \right) + \left( \frac{1}{K_{con,gp}} \right)
\]

where

\[
\left( \frac{1}{K} \right)_{con} = \text{flexibility of the connection}
\]

\[
\left( \frac{1}{K_{con,cfs}} \right) = \text{flexibility due to bending of cold-formed steel sections within the connection zone}
\]

\[
\left( \frac{1}{K_{con,bear}} \right) = \text{flexibility due to bearing of cold-formed steel sections around bolt holes}
\]

\[
\left( \frac{1}{K_{con,gp}} \right) = \text{flexibility due to bending of hot-rolled steel gusset plate within the connection zone}
\]

and

\[
K_{con,cfs} = \frac{3 (EI)_{lw}}{l_{lw}}
\]

\[
K_{con,bear} = \left( \frac{M}{\theta_{lw}} \right) \quad \text{and} \quad \theta_{bear} = \frac{\Delta}{r}
\]
\[ K_{con,gp} = \frac{3 (EI)_{gp}}{I_{con} r \Delta} \]

where

- \((EI)_{cfs}, (EI)_{gp}\) are the flexural rigidities of the cold-formed steel section and the gusset plate respectively.
- \(I_{con}\) is the length of the connection.
- \(r\) is the distance from the bolt group centre to the uttermost bolt, and
- \(\Delta\) is the bearing deformation of the section web around bolt hole to be specified.

After calibration against test data, it is found the connection stiffness at both the initial and the non-linear stages of deformation may be obtained simply by setting the value of \(\Delta\) to 1 mm and 3 mm respectively. The connection stiffness at the final stage of deformation is taken as zero.

In order to verify the accuracy of the proposed semi-empirical design rule, the connection stiffnesses of the six test specimens at both initial and non-linear stages of deformation are evaluated and summarized in Table 2. Moreover, the predicted moment joint-rotation curves of the six test specimens are also illustrated in Figure 6 for direct comparison with the measured curves. Furthermore, the overall load deflection curves of the six beam-column frames are predicted using the predicted moment joint-rotation curves. They are also plotted in the same graphs of the measured load-deflection curves of the test specimens, as shown in Figure 8. It is shown that the predicted load-deflection curves follow closely to the measured load-deflection curves. The lateral load resistances of the beam-column sub-frames are also summarized in Table 2. The ratios of the measured lateral load resistance to the predicted lateral load resistance of the six test specimens are also presented in Table 2. The model factors are found to range from 0.97 to 1.08, confirming the accuracy of the proposed method.

7. Conclusions

Examination on the structural performance in cold-formed steel portal frames with bolted moment connections shows that it is possible to design the structures against each of the failure mode independently. The lateral load resistances of the cold-formed steel beam-column sub-frames may be evaluated with adequate accuracy using nonlinear analysis with semi-rigid connections. Both the failure loads and the lateral deformation characteristics of the beam-column sub-frames are predicted satisfactory.

Engineers are encouraged to build short to medium span cold-formed steel portal frames with bolted moment connections for improved buildability when compared with timber or reinforced concrete frames.

Acknowledgements

The research project leading to the publication of this paper is supported by the Research Grants Council of the Hong Kong Government of the Special Administrative Region (Project No. PolyU5031/98E), and also by the Research Committee of the Hong Kong Polytechnic University Research (Project No. G-V750).
References


Lateral Load

Applied load

Connection under investigation

1106

1106

2000

2000

(S180A2)

S=180 mm

16 mm thick gusset plate without chamfers

10 mm thick gusset plate with chamfers

(S180D1/S240D1)

S=180, 240 mm

All bolts are M8.8 φ16

a) Internal beam-column sub-frames

Lateral Load

Applied load

Connection under investigation

1106

1106

2000

2000

(E180C2)

S=180 mm

16 mm thick gusset plate without chamfers

10 mm thick gusset plate with chamfers

(E180D1/E240D1)

S=180, 240 mm

All bolts are M8.8 φ16

b) External beam-column sub-frames

Figure 1 General test setup of cold-formed steel beam-column sub-frames.
Figure 2  Details of rotation measurement and calculation for internal beam-column sub-frames.
Figure 3  Details of rotation measurement and calculation for external beam-column sub-frames.
Figure 4  Load deflection curves of beam-column sub-frames tests with flexural failure in connected section.
Figure 5 Measured and corrected moment joint-rotation curves of beam-column sub-frames
Notation:
- $m_r = \frac{M_{\text{con}}}{M_R}$
- $\theta_r = \frac{\theta}{[M_R/(E/I/L)]}$
- $M_{\text{con}}$ = measured moment resistance of the connection
- $M_R$ = measured moment capacity of cold-formed steel sections
- $E$, $I$, $L$ = flexural rigidity and member length of the connected section
- $\theta$ = measured joint rotation obtained directly from test

Model I - flexural stiffness obtained directly from test
Model II - flexural stiffness obtained from semi-empirical design rules

Figure 6 Moment joint-rotation curves for non-linear analysis
Note: The length of the rigid link is the distance from the centre line of the column to the end of the connection:

\[ = 75 + 5 + 30 + 180 + 30 = 320 \text{ mm for } S=180 \text{ mm} \]

\[ = 75 + 5 + 30 + 240 + 30 = 380 \text{ mm for } S=240 \text{ mm} \]

Figure 7 Finite element model with semi-rigid joints.
Figure 8 Load deflection curves of beam-column sub-frames
<table>
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<tr>
<th>Column</th>
<th>Moment Resistance (KNm²)</th>
<th>Area (mm²)</th>
<th>Section Modulus (KNmm)</th>
<th>Yield Strength (KN/mm²)</th>
<th>Tension Plate Thickness (mm)</th>
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<tr>
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Table 1: Summary of test program and test data
### Table 2: Summary of non-linear analysis results

<table>
<thead>
<tr>
<th>Test</th>
<th>Model I</th>
<th>Model II</th>
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<tr>
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<td>$P_{\text{test}}$</td>
<td>$P_{\text{design}}$</td>
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</table>

**Notes:**
- **Model I** - flexural stiffness obtained directly from test data
- **Model II** - flexural stiffness obtained from semi-empirical design rules
Appendix B  Published technical papers in international conferences

B1.  Structural behaviour of cold-formed steel portal frames with lipped C sections
STRUCTURAL BEHAVIOUR OF COLD-FORMED STEEL PORTAL FRAMES WITH LIPPED C SECTIONS

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ABSTRACT
In order to understand the structural performance of bolted moment connections among cold formed steel members, a total of 9 component tests and 7 portal frame tests were carried out. A number of connection configurations with gusset plates of both hot rolled steel plates and cold-formed steel strips were examined. Among all the tests, the maximum moment resistance of the beam column connections was found to be 85% of the moment capacity of the members. The test results were fully presented in Reference 1. Thus, it is demonstrated that bolted moment connections between cold-formed steel members are structurally feasible and economical. Furthermore, cold-formed steel members with double lipped C sections back-to-back may be used to build short to medium span portal frames with bolted moment connections through rational design.

Relevant design rules on bolted moment connections specifically developed for cold-formed steel portal frames with bolted moment connections may be found in References 2 and 3. This paper presents some basic considerations on the structural performance of this particular form of cold-formed steel construction. Preliminary design rules in accordance with BS5950: Part 5 \(^{[4]}\) in evaluating the lateral load resistance of cold-formed steel portal frames based on plastic hinge analysis and non-linear analysis with semi-rigid connections are also presented.

Keywords: Cold-formed steel, bolted moment connections, portal frames, plastic design

1. Introduction

Galvanized cold-formed steel strips are commonly used in building construction, such as sections for secondary steel frames and purlins, and sheetings for roof cladding and floor decking. Cold-formed steel sections and sheetings are effective construction materials due to their high strength to weight ratio, high buildability during construction and also long-term durability against environmental attack. The most widely used cold-formed steel members are the C sections and Zed sections, and the thickness of these sections ranges typically from 1.2 mm to 3.0 mm. Both steel with yield strength of
280 N/mm² and 350 N/mm² are commonly used. A number of design recommendations [4-6] on the design of cold-formed steel structures together with worked examples may be found in the literature.

In building construction, cold-formed steel sections are usually bolted to hot rolled steel plates or members to form simple and moment connections. A number of cold-formed steel framing systems [7] have been developed by laboratory tests while finite element analysis on connections between cold-formed steel members have also been carried out [8-10]. In order to extend the application of cold-formed steel members to portal frame construction, it is proposed to use double-flanged C sections back-to-back as primary members. The beam column moment connections are formed by attaching the section webs of the cold-formed steel members onto gusset plates with bolts; the section flanges are not connected for ease of installation.

It should be noted that most of the design recommendations on connections among cold formed steel members concern the load carrying capacities of individual fasteners such as bolts, screws, rivets and spot welds. Little information on the structural performance of the bolted moment connections among cold-formed steel members may be found in the literature. It is desirable to develop design rules to assess the strength and the stiffness of bolted moment connections for efficient cold-formed steel framing in building construction.

2. Recent research on bolted moment connection

In order to examine the structural performance of cold-formed steel members with bolted moment connections, three column base connection component tests and six beam column connection component tests were carried out with different connection configurations using both hot rolled steel and cold-formed steel gusset plates. Furthermore, seven system tests with rectangular, L-shaped and haunched gusset plates of both hot rolled steel and cold-formed steel were also tested as portal frame structures. Figure 1 illustrates the general layout of the component and the system tests. Full details of the test program and the test results are presented in Reference 1.

2.1 Modes of failure

Among all the tests, four failure modes are identified as follows:

- **BFcs** bearing failure in section web around bolt hole,
- **LTBgp** lateral torsional buckling of gusset plate,
- **FFcs** flexural failure of connected member, and
- **CBcol** combined compression and bending of column member.

It should be noted that most of the design recommendations provide design rules to evaluate the load carrying capacity of individual fasteners, which may be used to design connections against the failure mode **BFcs**. For the failure mode **LTBgp**, a set of design rules is formulated in accordance to BS5950 and presented in Reference 2. As it is useful to visualize the elastic stress distribution in the connected parts of cold-formed steel sections and also the load path across the connections, a finite element analysis was carried out using the general finite element package SAP97. Based on the results of the finite element modeling, a simple design expression is proposed in Reference 3 for failure mode **FFcs** to allow for partial effectiveness of the connections due to incomplete load path in section flanges.

3. Structural performance of cold-formed steel portal frames

This paper presents some basic considerations on the structural performance of cold-formed steel portal frames. In order to facilitate the discussion, some of the test results from the sixteen component and system tests are presented in Table 1. The moment resistances of the connections are given at their peak values of their respective moment rotation curves or the applied moment at 0.05 radian rotation; all the values are normalized according to design yield strength and design thickness of cold-formed steel. As the bolted moment connections with four bolts per member at large bolt spacing are regarded as efficient, only the connection configurations of **P8** and **L8** are considered. Refer to Figure 2 for details of the connection configurations.
3.1 Component Tests v.s. System Tests

In order to understand the structural performance of new structural forms such as cold-formed steel portal frames, it is necessary to carry out experimental investigation on full-scale portal frames. However, full-scale tests on portal frames are expensive and also time consuming. It is thus desirable to be able to use the test results of component tests, such as beam column connection tests, as representative data in assessing the overall performance of the portal frames.

It is shown that the moment resistances obtained from the component tests and the system tests are fairly close to each other, and the ratios of those two moment resistances, or the model ratios, are:

- 1.03 for connections with gusset plates of hot rolled steel plates, and
- 0.93 for connections with gusset plates of cold-formed steel strips.

The discrepancy in the moment resistances measured from both types of tests are primarily due to the different level of restraints provided in the tests, i.e. the external lateral restraints in component tests as compared with the secondary bracing members attached onto the frames in system tests. Consequently, it is demonstrated that during the development process of cold-formed steel portal frames, component tests may be carried out to gauge the structural behaviour of a number of proposed connection configurations. Full-scale system tests may then be carried out on portal frames with connection configurations of high structural performance obtained from component tests. This development procedure will enable a wide range of connection configurations to be examined with minimum efforts.

3.2 Plastic hinge analysis on cold-formed steel portal frames

For portal frames with hinged column bases, i.e. HS01-P8 and HS03-P8, Table 1 shows that it is possible to evaluate the lateral load resistance, P, of the portal frames based on plastic hinge analysis. The accuracy of the analysis depends primarily on the level of lateral restraints provided in component tests as compared with those in system tests.

It is interesting to compare the test results between test HS07-L8, a portal frame with fixed column bases, and test HS05-L8, a portal frame with hinged column bases. Figure 3a shows that the moment resistance of the beam column connection, \( M_{b1} \), may be estimated as 13.56 kNm based on plastic hinge analysis on the maximum applied force, P. By considering the formation of four plastic hinges at both the beam column connections and the column base connections as shown in Figure 3b, the lateral load resistance of the portal frame is estimated as 27.96 kN. (The moment resistance of the column bases, \( M_{c2} \), is found to be 10.28 kNm from the column base component tests, as shown in Table 1.) As the measured lateral load resistance is 30.0 kN, the plastic hinge analysis is found to be satisfactory with a discrepancy of merely 7%. In HS07-L8, the presence of fixed column bases will reduce the slenderness of the portal frames, and thus, the moment resistance of the beam column connections in test HS07-L8 is expected to be higher than that of test HS05-L8.

3.3 Deformation ductility of failure modes

The application of plastic hinge analysis to cold-formed steel portal frames is highly desirable to engineers due to its simplicity. It is well established in hot rolled steel portal frame construction that the method is valid only for 'plastic' sections, i.e. for sections which may undergo large rotation without strength reduction. In general, it is considered to be not applicable to cold-formed steel sections in which local buckling is always critical and causes severe reduction in strength and stiffness at large deformation. Lateral torsional buckling of gusset plates and flexural failure of cold-formed steel members are also examples of failure with rapid unloading.

However, in this particular form of cold-formed steel construction, the maximum moment always occurs at the member connections where local buckling in the cold-formed steel sections is prohibited in the vicinity of the connections due to the presence of gusset plates. For those connection configurations examined in the tests, bearing failure of connected parts of the cold-formed steel sections always occur which provides sustained resistance at large deformation, i.e. ductility, as illustrated in the measured moment rotation curves of the connections. The portal frames will collapse when any other failure mode creeps in at a higher load level, i.e. either lateral torsional buckling of
gusset plates or the flexural failure of cold-formed steel sections. At collapse, once the moment resistances at the beam column connections are established, the lateral load resistances of the portal frames may be determined using plastic hinge analysis.

4. Non-linear analysis with semi-rigid connections

In order to examine the lateral load-deflection characteristic of cold-formed steel portal frames, non-linear analyses\[1]\, incorporating semi-rigid joints\[13\] were carried out for both portal frames HS05-L8 and HS07-L8. Figure 4a illustrates the moment-connection rotation curves, or \(m-\theta\) curves, of both the column base connection and the beam column connection as obtained from tests\[1\]. The curves are presented in a normalized format in terms of \(m\) which is the moment ratio of the connection resistance to the moment capacity of the cold-formed steel sections. Two \(m-\theta\) curves are presented for the beam column connections to provide the maximum and the minimum curves for analysis as there are two frames with four beam-column connections in every test specimen of the system tests.

The predicted lateral load-deflection curves of the portal frames, \(P - \Delta\), are presented in Figure 4b together with the test data for direct comparison. It is shown that based on the measured \(m-\theta\) curves for both beam column connections and column base connections, the overall lateral behaviour of the portal frames may be predicted satisfactorily along the entire deformation history, i.e. from local \(m-\theta\) of connections to overall \(P - \Delta\) of portal frames. The non-linear analysis will be useful to allow for the \(P-\Delta\) effect in portal frames under combined vertical and lateral loads.

5. Conclusions

Examination on the structural performance in cold-formed steel portal frames with bolted moment connections shows that it is possible to design the structures against each of the failure mode independently. The lateral load resistances of the portal frames may be evaluated with adequate accuracy using plastic hinge analysis once the moment resistances of the connections are established. Advanced non-linear analysis with semi-rigid joints may be used to predict the load carrying capacity and also the deformation of cold-formed steel portal frames under combined lateral and vertical loads.

Engineers are encouraged to build short to medium span cold-formed steel portal frames with bolted moment connections for improved buildability when compared with timber or reinforced concrete frames. It is necessary to extend the present study further to cover other structural forms, such as box sections or compound C sections for wide application.

Acknowledgement

The publication of the paper is supported by the Hong Kong Research Grant Council (RGC Ref. No. PolyU 5085/97E).

References


<table>
<thead>
<tr>
<th>Component Test</th>
<th>Normalised moment resistance, $M_c$ (kNm)</th>
<th>System Test</th>
<th>Normalised lateral load (kN)</th>
<th>Normalised moment resistance $M_s$ (kNm)</th>
<th>Test ratio $M_s/M_c$</th>
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<tr>
<td>CB04*</td>
<td>10.28</td>
<td>-</td>
<td>-</td>
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<td>HS07-L8</td>
<td>30.00</td>
<td>-</td>
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</tr>
</tbody>
</table>

Note: * denotes a column base component test with four bolts per member.

**Figure 1** General layout of testing

**a) Component Tests**

**b) System Tests**

<table>
<thead>
<tr>
<th>Member Mark</th>
<th>Section Size</th>
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<tr>
<td>C</td>
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</tr>
<tr>
<td>B / WB</td>
<td>2 No. C10015 Back to Back G450</td>
</tr>
<tr>
<td>BB</td>
<td>1 No. C20016 G450</td>
</tr>
<tr>
<td>SB</td>
<td>1 No. C10015 G450</td>
</tr>
</tbody>
</table>
### Figure 2 Details of connection configurations

<table>
<thead>
<tr>
<th>Connection type</th>
<th>Connection details</th>
<th>Sections</th>
<th>Connectors</th>
</tr>
</thead>
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<tr>
<td><strong>HRS-P8</strong></td>
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<tr>
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<td><img src="image8" alt="Diagram" /></td>
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<tr>
<td><strong>CFS-L8</strong></td>
<td><img src="image10" alt="Diagram" /></td>
<td><img src="image11" alt="Diagram" /></td>
<td><img src="image12" alt="Connector" /></td>
</tr>
</tbody>
</table>
a) HS05-L8 with hinged column bases

\[ M_{pl1} = \frac{14.69 \text{ kN}}{2} \times 1.846 \text{ m} = 13.56 \text{ kNm} \]

b) HS07-L8 with fixed column bases

\[ M_{p2} = 10.28 \text{ kNm} \]

\[ P = \frac{13.56 + 10.28}{1.705} \times 2 = 27.96 \text{ kN} \]

Figure 3 Cold formed steel portal frames
Figure 4a Normalized moment-connection rotation curves

<table>
<thead>
<tr>
<th>Non-linear analysis</th>
<th>m-( \theta_c )</th>
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<tr>
<td>A</td>
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</tr>
<tr>
<td>B</td>
<td>S + C</td>
</tr>
</tbody>
</table>

Figure 4b Lateral Load Deflection Curves for Tests

*HS05 and HS07*
Appendix B  Published technical papers in international conferences

B2. Research and development on building construction using cold-formed steel sections
Research and development on building construction using cold-formed steel sections

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\textsuperscript{3}Department of Civil Engineering, Tsinghua University, China.

Abstract  Cold-formed steel sections are light-weight materials and suitable for building construction owing to their high structural performance and durability. In order to improve the buildability of cold-formed steel sections for wide application in building construction, a number of research and development projects were executed by the authors under the support of various government funding bodies, steel suppliers and fabricators in the U.K. and Hong Kong. The objectives and the key findings of selected projects are summarized in the paper.

Key words  Cold-formed steel sections, web openings, modern roof systems, simple and moment connections, experimental investigations, finite element modelling, design development, computer software, and professional design guides.

1 Introduction

Cold-formed steel sections\textsuperscript{1-9} are light-weight materials and suitable for building construction owing to their high structural performance and durability. They are widely used as purlins in roofs, joists of medium span in floors, studs in wall panels, storage racking in warehouses, and hoarding structures in construction sites. Since 1990, there is a growing trend to use cold-formed steel sections as primary structural members in building construction, such as low to medium rise residential houses and portal frames of modest span. Other common cold-formed steel building products include cold-formed steel roof and wall claddings with colour coatings in building envelope construction, and also cold-formed steel profiled decking in composite slab construction.

The most common sections are lipped C sections and lipped Z sections, and the thickness typically ranges from 1.2 mm to 3.2 mm. Common yield strengths are 280 N/mm$^2$ and 350 N/mm$^2$. Moreover, there are a whole range of variants of these basic shapes, including sections with single and double lips, and sections with internal stiffeners. Due to the thinness of cold-formed steel sections, local buckling is a predominant consideration in assessing their section capacities. Furthermore, as they are very weak in torsion, torsional flexural buckling in columns and lateral torsional buckling in beams may be critical. Both bolts and self-drilling self-tapping screws are common fasteners in cold-formed steel construction while welding is seldom used due to the thinness of cold-formed steel sections and also the presence of galvanized coatings.

In order to improve the buildability of cold-formed steel sections for wide application in building construction, a number of research and development projects were executed by the authors under the support of government funding bodies, universities, research institutes, steel suppliers and fabricators in the U.K. and Hong Kong. The objectives and the key findings of selected projects are summarized in the paper.

2 Cold-formed steel sections with single and multiple web openings

In building construction, the buildability of cold-formed steel sections may be greatly improved by the provision of web openings in the sections, which allows easy integration of building services within the section depth.

In order to assess the structural implication of web openings to cold-formed steel sections, the reduction in the web crippling and the load resistances of perforated sections were investigated. The effects of different opening sizes and shapes, and of different forms of stiffening around the openings were also studied. A total of six test series with 57 web crippling tests and 41 beam tests were carried out\textsuperscript{10} on cold-formed steel sections with single and multiple web openings as shown in Fig. 1.

It is found\textsuperscript{11,12} that for perforated sections with practical opening sizes and shapes, the reduction to the web crippling and the load resistances is often not significant. In typical floor beams, the presence of single web openings in the sections may reduce the load resistances of the sections by only 5% to 10%, provided that the web openings are located at specific positions. For sections with multiple web openings, the reduction to the load resistance of the sections is unlikely to exceed 15%.
Furthermore, a set of design rules is formulated in accordance with both BS5950: Part 5 and Eurocode 3: Part 1.3. The resistances of the perforated cross-sections are based on the effective sections of the perforated cross-sections and thus local buckling of the sections is incorporated. For perforated sections susceptible to 'Vierendeel' mechanism, a design rule based on structural design principles is formulated to allow for the effect of coexisting axial and shear forces on the local moment resistances of the perforated sections. Moreover, semi-empirical formulæ are also established to evaluate the reduction factors to the web crippling resistances of cold-formed steel sections to allow for the presence of web openings.

3 Structural performance of modern roofs with thick over-purlin insulation

In conventional roof and purlin systems, it is generally assumed that the roof cladding restraints the purlins from lateral buckling. However, in the 1990's, the requirement of increased insulation thickness for energy saving has led to a proliferation of roof systems and fixing methods for industrial-type buildings in the U.K. and Europe. The modern roofs are likely to behave differently in their interaction with the purlins compared to the conventional trapezoidal sheeting with relatively thin insulation. At worst, the restraint provided to the purlins by the modern roofs may not be sufficient, and the purlins may fall under heavy snow load or wind suction.

In order to investigate the structural performance of modern roof systems with thick over-purlin insulation, three full scale test series with a total of 12 single and 5 double span roofs supported by purlins of zed and sigma sections were executed under both gravity load and wind uplift, as shown in Fig.2.

Among the five modern roof systems examined, two of them are identified to be unable to provide full restraint to the attached purlins under gravity load with failure in clip fixings. Furthermore, all of the roof systems can only provide partial restraints to the attached purlins under wind uplift. Potential danger is also identified in extending current practice to accommodate thicker insulation in the modern roof systems, and positive metal fixing should be used in order to provide full restraint to the purlins. Comparison on the structural performance among the modern roof systems is presented in detail and the implication of thick over-purlin insulation is also fully discussed.

4 Design and construction of connections between cold-formed steel sections

Most of the codified design rules are only applicable in assessing the load carrying capacities of individual fasteners such as bolts and screws rather than the structural performance of connections between cold-formed steel sections. While it is important to assess the load carrying capacity of each fastener, it is also necessary to examine the structural behaviour of the connectors such as web cleats and gusset plates, and also of the connected parts of cold-formed steel sections under highly localized forces and bending moments. In general, there is a lack of design information of simple connections and moment connections between cold-formed steel sections.

4.1 Simple connections between cold-formed steel sections

In order to improve the buildability of cold-formed steel sections, it is highly desirable to use folded cold-formed steel strips as web cleats rather than hot rolled angles to form shear resisting connections. This will allow greater compatibility in materials and connection methods, and both the fabrication and the installation processes will also be simplified. Furthermore, the use of self-drilling self-tapping screws does not require pre-drilled holes and thus, the problem of construction tolerance on site may be reduced significantly.

A total of 24 tests on simple connections between cold-formed steel sections in four connection configurations as beam to beam connections and beam to column connections were carried out as shown in Fig. 3.

It is demonstrated that typical shear resistances of the proposed connections with cold-formed steel web cleats range from 9 kN to 20 kN while the end deflection of the cold-formed steel web cleat is always less than 5 mm, which may be considered to be acceptable in building construction. Moreover, a set of design rules on the load carrying capacities of various connection configurations is formulated in accordance with both BS5950: Part 5 and Eurocode 3: Part 1.3. They have been calibrated against test data and thus verified as safe and structurally economical for general use.

4.2 Moment connections between cold-formed steel sections

Bolted moment connections with high strength and stiffness are essential in safe and economical design and construction of purlin systems, and thus there are many research work reported in the literature on the development of purlin-rafter connections in modern roof systems. They are basically beam-to-beam connections with different degrees
of continuity to reduce mid-span moment and deflection. However, most of the codified design rules do not consider connections between cold-formed steel sections to be moment resisting, and thus many new cold-formed steel products are developed from experimental testing rather than from design methods due to the lack of design guidance.

In general, it is envisaged that bolted moment connections between cold-formed steel sections are not able to develop full moment capacity of the connected sections due to discontinuity of load paths along section flanges. After considering ease of fabrication and installation, the basic connection configurations of bolted moment connections between cold-formed steel sections is established\(^{27,18,19,20,21,22}\) as follows:

- All structural members such as beams and columns are formed with two lipped C-sections back-to-back with interconnections.
- Moment connections between beams and columns are formed with hot-rolled steel gusset plates.
- Only the webs of lipped C-sections are bolted onto gusset plates.
- Four bolts per member are used.

In order to investigate the structural performance of bolted moment connections between cold-formed steel sections, a total of 28 tests on both external and internal beam-column sub-frame tests under lateral load and gravity load were executed\(^{30,21,22}\) as shown in Fig. 4. Among the tests, four different modes of failure were identified, and the moment resistances at the actual failure positions of the proposed connection configurations were found to range from 0.77 to 0.97 of the moment capacities of connected sections.

Consequently, the proposed moment connections are demonstrated to be effective in transmitting moment between the connected sections, and thus enabling effective moment framings in cold-formed steel structures. The moment capacity ratios of the connections are typically 0.75, and they may be increased to 0.90 through rational design\(^{17,18,19}\) in the connection configuration.

5 Structural behaviour of high strength cold-formed steel

In recent years, due to advances in steel technology, cold-formed steel strips with high yield strengths up to 450 N/mm\(^2\) and 550 N/mm\(^2\) become available for sections and sheetings respectively. However, the ductility of high strength steels is found to be reduced significantly with an elongation limit typically less than 10%, as compared with the elongation limits at 15% and 25% in low strength cold-formed steels and hot rolled steels respectively. For high strength cold-formed steel sections, local buckling in plate elements of sections and overall instability in beams and columns become more critical than those in low strength cold-formed steel sections. Furthermore, with reduced ductility, there is concern about the structural adequacy of high strength cold-formed steel sections in term of deformation capacity, especially at connections where highly localized deformations are expected. All the existing design rules may not be adequate for high strength cold-formed steel sections as they are originally developed from test data with low strength cold-formed steel sections. Those design rules are unlikely to provide sufficient safety margin in assessing the section capacities, the member resistances, and also the connection resistances of high strength cold-formed steel sections.

5.1 Numerical investigation on bolted connections

In order to investigate the structural performance of cold-formed steel bolted connections under shear, a finite element model with three-dimensional solid elements is established\(^{22,27}\), and three distinctive failure modes as observed in lap shear tests are successfully modelled\(^{24,35}\), namely, (a) bearing failure, (b) shear-out failure, and (c) net-section failure, as shown in Fig. 5. A finite element model for bolted moment connection is also established as shown in Fig. 6.

It is demonstrated that the predicted load-extension curves of bolted connections in lap shear tests followed closely to the measured load-extension curves provided that measured steel strengths and geometrical dimensions are used in the analysis. Furthermore, it is shown that stress-strain curves, contact stiffnesses and frictional coefficients between element interfaces, and clamping forces developed in bolt shanks are important parameters for accurate prediction of the deformation characteristics of bolted connections. A similar finite element model for double bolted connections is also established\(^{25}\) to investigate the effect of bolt pitch on the load carrying capacities of connections with multiple bolts.

A total of 72 lap shear tests on single and double bolted connections with four steel materials (two thickness and two steel grades), two bolt diameters, four edge distances and four bolt pitches are also carried out for calibration of the finite element models.
A parametric study\textsuperscript{26,27} on bolted connections with different configurations is performed to provide bearing resistances for practical design. The results of the finite element modelling are also compared with a number of codified design rules. It is found that the design rules are not applicable for bolted connections with high strength steels due to reduced ductility. Consequently, a semi-empirical design formula for bearing resistance of bolted connections is proposed after calibrating against finite element results. The proposed design rule relates the bearing resistances with the design yield and tensile strengths of steel strips through a strength coefficient. The design rule is demonstrated to be applicable for bolted connections of both low strength and high strength steels with different ductility limits.

5.2 Numerical investigations on section capacities

In order to formulate a general procedure to evaluate the effective section properties of cold-formed steel sections, a finite element model with both geometrical and material non-linearity using the commercial software ABAQUS is established. The section capacities of a lipped C section and a sigma section under compression, major axis bending, minor axis bending with both sagging and hogging moment may thus be readily obtained for general design as shown in Figs. 7 and 8. In the finite element model, the true stress-strain curve obtained from coupon test results and modified for excessive elongation is used. Both the residual stress and the strength enhancement due to cold-forming around corners are neglected while the lowest eigenmodes of the sections with specified magnitudes of out-of-flatness are adopted as the initial imperfections. However, numerical instability is sometimes encountered even the full Newton-Raphson procedure and the arc length method are used during iterations for solution.

At present, calibration of the finite element model with test data and codified design rules is underway. It is proposed to formalize the finite element modelling method in assessing the effective section properties of cold-formed steel sections as design data. This will be helpful to generate section capacities of sections with complicated profiles during product development of cold-formed steel building products including sections, cladding, and decking.

6 Professional design guides and construction projects

A number of professional design guides\textsuperscript{28,29,30} on building construction using cold-formed steel sections are compiled in accordance with both BS5950: Part 5 and Eurocode 3: Part 1.3 and basic design principles on local buckling in plate elements of cold-formed steel sections under compression, bending and shear are presented. Furthermore, instability in both beams and columns are fully described and simplified design rules are also provided to assess the member resistances of the sections in practical conditions. Both gross and effective section properties of selected cold-formed steel sections are tabulated, and safe load tables of generic sections are also provided for scheming design. Computer software for section capacities and member resistances of cold-formed steel sections\textsuperscript{31,32} is also available. A set of worked examples on typical applications of cold-formed steel sections as roof trusses, beams, and columns is also available, together with connection designs with bolts and screws. Design information and data for cold-formed stainless steel sections\textsuperscript{33} is also available.

A number of cold-formed steel construction projects are recently completed in Hong Kong:

- Temporary accommodations for 2000 workers during the construction of the New Hong Kong Chek Lap Kok International Airport.
- A demountable exhibition hall of the Hong Kong Housing Authority in Ho Man Tin, Kowloon.
- A residential re-development project of the Hong Kong Housing Authority in Yuen Long where a number of 4 storeys high and 6 storeys high residential flats were built on farm lands with low bearing capacities.
- A two storey high portal frame building as new lecture rooms of the Hong Kong University built upon existing structures.

7 Conclusions

Cold-formed steel sections are light-weight materials and suitable for building construction owing to their high structural performance and durability. Engineers are encouraged to take full advantages offered by cold-formed steel construction technology to build strong and stiff buildings of high buildability and structural economy.

8 Acknowledgements

The publication of the paper is supported by the Hong Kong Research Grant Council (RGC Ref. Nos. PolyU5031/98E and PolyU5040/99E), and also the Hong Kong Polytechnic University Research Committee (Research Project No. G-V750).
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33. Chung, K.F., Badshoo N.R and Burgan B.A: Section Property and Member Capacity Tables for Cold Formed Stainless Steel, the Steel Construction Institute, SCI-P-152, 1995.
Figure 1
Investigation of cold-formed steel sections with single and multiple web openings

Figure 2
Structural performance of modern roofs with over-purlin insulation
Figure 3
Simple connections between cold-formed steel sections

(a) Beam-column connections with cold-formed steel web cleats in practical member orientation

(b) Lateral torsional buckling of cold-formed steel web cleat

Figure 4
Moment connections between cold-formed steel sections

(a) Cold-formed steel portal frames with hot rolled steel gusset plates

(b) Internal beam-column sub-frames under lateral load

(c) Internal beam-column sub-frames under gravity load

(d) External beam-column sub-frames under lateral load
Figure 5
Finite element investigation on tension connection
Lap shear test

(a) bearing failure

(b) shear-out failure

(c) net-section failure

(d) bearing failure in double bolted connections

Figure 6
Finite element investigation on moment connection
Cantilever bending test

(a) Geometry

(b) First yield

(c) Yielding pattern at failure
Figure 7  Finite element investigation on a lipped C section

(a) Geometry  Yield strength = 280 N/mm²

(b) Compression

(c) Major axis bending (sagging)

(d) Minor axis bending (sagging)

(e) Minor axis bending (hogging)
Figure 8  Finite element investigation on a sigma section
Appendix B  Published technical papers in international conferences

B3. Experimental investigation on bolted moment connections in beam-column sub-frames – Pilot study
EXPERIMENTAL INVESTIGATION OF COLD-FORMED STEEL BEAM-COLUMN SUB-FRAMES: PILOT STUDY

M F Wong¹ and K F Chung²

SUMMARY
This paper presents the findings of an experimental investigation on the structural performance of bolted moment connections in cold-formed steel beam-column sub-frames. A total of eight tests with three different connection configurations in both internal and external columns were carried out. Double lipped C-sections back-to-back with hot rolled steel gusset plates of 10 mm and of 16 mm in two different shapes were tested; four bolts per member were used in the connections.

Among the tests, three different modes of failure were identified and the measured moment resistances at the connections were found to vary from 36% to 97% of the measured moment capacities of the cold-formed steel sections, demonstrating that bolted moment connections between cold-formed steel members are structurally feasible and economical. Furthermore, structural members with double lipped C sections back-to-back are shown to be practical in constructing short to medium span portal frames with bolted moment connections through rational design.

INTRODUCTION
Cold-formed steel sections are light-weight materials and suitable for low-rise building construction owing to its high buildability¹. The most common section is lipped C section, and the thickness typically ranges from 1.2 mm to 3.2 mm. The yield strengths normally are 280 N/mm² to 450 N/mm². Cold-formed steel sections are widely used as purlin members in modern roof systems, floor joists of medium span, studs in wall panels, storage racking, and hoarding structures.

Both bolts and self-drilling self tapping screws are common fasteners in cold-formed steel construction. While many modern codes and standards²,³,⁴,⁵ for cold-formed steel structures present design expressions on the load carrying capacities of bolts and screws, there is little design recommendations on the strength and the stiffness of connections. The minimum configuration of bolted connections with two bolts per member is commonly regarded as simple (or shear) connections.

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Bolted moment connections with high strength and stiffness are essential in safe and economical design and construction of purlin systems\(^6,7\), and thus there are many research works reported in the literature on the development of purlin-rafter connections in modern roof systems. They are basically beam-to-beam connections with different degrees of continuity to reduce mid-span moment and deflection. However, most of the modern codes and standards do not consider connections between cold-formed steel sections to be moment resisting, and thus many new cold-formed steel products are developed from experimental testing rather than from design methods due to the lack of relevant design recommendations.

In order to extend the effective use of cold-formed steel in building applications, it is highly desirable to build safe and economical moment frames, and thus design information on bolted moment connections in beam-column sub-frames with practical connection configurations should be provided. Several investigations on cold-formed steel members with bolted moment connections have been reported in the literature\(^8,9,10\).

This paper presents the findings of a test program on the investigation of the structural performance of bolted moment connections in cold-formed steel beam-column sub-frames. A total of eight tests with three different connection configurations in both internal and external beam-column sub-frames were carried out. It is intended to demonstrate the high structural performance of bolted moment connections between cold-formed steel members, and to establish the high structural efficiency of bolted moment connections in practical framing.

**TEST PROGRAM**

The basic configuration of bolted moment connections proposed in beam-column sub-frames for building applications is:

- All structural members such as beams and columns are formed with two lipped C sections back-to-back with interconnections.
- Moment connections between beams and columns are formed with hot-rolled steel gusset plates.
- Only the webs of lipped C sections are bolted onto gusset plates.
- Four bolts per member are used.

The connection details are rationalized after considering ease of fabrication and installation. In general, the proposed moment connections are not able to develop full moment capacity of the connected members due to discontinuity of load paths along section flanges in the sections. In the eight beam-column sub-frame tests, two lipped C sections with different thickness are used:

- C15016 DS denotes a double section of lipped C sections of 150 mm section depth, 64 mm flange width with a thickness of 1.6 mm.
- C15020 DS denotes a double section of lipped C sections of 150 mm section depth, 60 mm flange width with a thickness of 2.0 mm.

The design yield strength of all the sections is 450 N/mm\(^2\) which is designated as G450. The moment capacities of the sections C15016DS G450 and C15020DS G450 measured from four point load tests are 16.95 kNm and 21.36 kNm respectively. All bolts are 16 mm in diameter and of Grade 8.8.
All the test specimens are constructed according to the basic configurations with systematic variations in the connection details, i.e. bolt arrangement (or bolt pitch), and shape and thickness of gusset plates. The test designation 'S090A1' refers to an internal beam-column sub-frame with 'cross' shaped gusset plate of 10 mm thick and 4 bolts per member at a bolt pitch of 90 mm. The test program is summarized in Table 1.

In order to examine the structural performance of bolted moment connections against bolt pitches, three internal beam-column sub-frame tests with different pitches of 90 mm, 180 mm and 240 mm were carried out, i.e. S090A1, S180A1 and S240A1; the thickness of the gusset plates is 10 mm. Two external beam-column sub-frame tests with similar connection configurations were also carried out for comparison, i.e. E180C1 and E240C1.

Furthermore, in order to examine the structural performance of bolted moment connections against gusset plate thickness, two internal beam-column sub-frame tests with gusset plates of 10 mm and 16 mm thick were carried out, i.e. S090A2 and S180A2. One additional external beam-column sub-frame with a gusset plate of 16 mm thick was also carried out for comparison, i.e. E180C2.

TEST INSTRUMENTATION
The general arrangement of the test set-up together with the proposed connection configurations for both internal and external beam-column sub-frames are shown in Figure 1. Both the applied load and the displacements of each member of the test specimens were measured during the entire deformation history. The tests were terminated when either section failure or member buckling occurred, or deformation of the test specimens became excessive, i.e. over 250 mm. In most cases, a pre-load of 2 kN was applied before tests to ensure that all bolts were in contact with the section webs of connected sections despite all the bolt holes were 'perfect-fitted' to 16 mm diameter bolts.

TEST RESULTS
Three different modes of failure were identified among the eight tests:

- BFcsW Bearing failure in section web around bolt hole, as shown in Figure 2.
- FFgp Flexural failure of hot rolled steel gusset plate.
- FFcs Flexural failure of connected cold-formed steel section, as shown in Figure 3.

Figure 4 presents the typical lateral load - lateral deflection curves of the test specimens and the typical moment - rotation curves of the test specimens are presented in Figure 5.

The maximum moment resistances of the test specimens may be evaluated at two locations, namely, at the centreline and at the failure position of the connections for different purposes. The centreline evaluation enables easy comparison of the moment capacities of the connections against applied moment obtained directly from conventional structural analysis while the failure position evaluation is required for connection design.

In general, the moment resistances are first evaluated at the centreline of the connections, and the level arm coefficients defined in Figure 6 are then applied according to the associated failure modes to give the moment resistances of the connections at the failure positions.
The results of all the eight tests are summarized in Table 1 together with the measured material properties and dimensions of the cold-formed steel sections and the gusset plates.

In order to assess the effectiveness of the bolted moment connections, a moment resistance ratio, $\psi$, is established which is defined as follows:

$$\psi = \frac{\text{Measured moment resistance of a connection}}{\text{Measured moment capacity of connected section}}$$

All the measured moment resistances are normalized with the ratio of design yield strength and design thickness to measured yield strength and measured thickness of the test specimens. For test specimens with excessive deformation under testing, the moment resistance of the connection is restricted to be the applied moment at a connection rotation of 0.05 radian.

COMPARISON AND DISCUSSION

After data analysis, the moment rotation curves of the internal and the external beam-column sub-frames are presented in Figures 7 and 8 for easy of comparison. The results of all the tests are compared among each other and the findings are presented as follows:

a) Tests S090A1, S180A1 and S240A1

In these three tests, large deflections and rotations of the test specimens were observed during load application. The measured moment resistances of tests S090A1, S180A1 and S240A1 at the centreline of the beam-column connections were 6.72 kNm, 16.74 kNm and 15.01 kNm respectively. There was no distinctive out-of-plane deformation of the test specimens during testing. After the tests, all the members of the sub-frames were disassembled from the connections for inspection. In test S090A1, significant bearing deformation was observed in the bolt holes of the beam members due to high moment acting at small lever arms. For both tests S180A1 and S240A1, gross bending deformation was apparent in the hot-rolled steel gusset plates.

The moment resistance ratios of the connections in tests S090A1, S180A1 and S240A1 were found to be 0.36, 0.69 and 0.74 respectively; the sections in test S180A1 is C15020 DS while the sections in the other two are C15016 DS.

b) Tests E180C1 and E240C1

In these two tests, large deflections and rotations of the test specimens were observed during load application. The measured moment resistances of tests E180C1 and E240C1 at the centreline of the beam-column connections were 20.12 kNm and 20.97 kNm respectively. There was no distinctive out-of-plane deformation of the test specimens during testing. After the tests, all the members of the sub-frames were disassembled from the connections for inspection. Gross bending deformation was apparent in the hot-rolled steel gusset plates of both tests.

The moment resistance ratios of the connections in tests E180C1 and E240C1 were found to be 0.84 and 0.88 respectively.
c) Tests S090A2, S180A2 and E180C2  
In these three tests, large deflections and rotations of the test specimens were observed during load application. The measured moment resistances of tests S090A2, S180A2 and E180C2 at the centreline of the beam-column connections were 18.59 kNm, 23.16 kNm and 24.44 kNm respectively. There was no distinctive out-of-plane deformation of the test specimens during testing. After the tests, all the members of the sub-frames were disassembled from the connections for inspection. In test S090A2, significant deformation was observed in the bolt holes of the beam members due to high moment acting at small lever arms. For both tests S180A2 and S240A2, flexural failure in connected cold-formed steel sections was apparent.

The moment resistance ratios of the connections in tests S090A2, S180A2 and E180C2 were found to be 0.57, 0.92 and 0.97 respectively.

d) For internal beam-column sub-frames with 10 mm thick gusset plates, the increase in the bolt pitch from 90 mm to 180 mm and then to 240 mm is shown to increase the moment resistance ratio of the proposed connection configuration from 0.36, to 0.69 and then to 0.74, as shown in tests S090A1, S180A1 and S240A1. Moreover, for external beam-column sub-frames with 10 mm thick gusset plates, the increase in the bolt pitch from 180 mm to 240 mm is also shown to increase the moment resistance ratio of the proposed connection configuration from 0.84 to 0.88 as shown in tests E180C1 and E240C1.

This shows that connections with large bolt pitch will always give high moment resistances. Moreover, with an increase in the moment resistances, flexural failure in the gusset plates rather than bearing failure in the connected section web becomes critical in the connections.

e) By increasing the thickness of gusset plate from 10 mm to 16 mm, the moment resistance ratio of the proposed connection configuration is found to be increased from

- 0.36 to 0.57 as shown in tests S090A1 and S090A2,
- 0.69 to 0.92 as shown in tests S180A1 and S180A2, and
- 0.84 to 0.97 as shown in tests E180C1 and E180C2.

This shows that thicker gusset plates will always give high moment resistances. Moreover, with an increase in the moment resistances of the gusset plates, flexural failure in the connected cold-formed steel sections rather than flexural failure of the gusset plates becomes critical in the connections. The maximum moment resistance of the proposed connection configuration is found to be over 90% of the moment capacities of the sections, demonstrating that the proposed connection configuration is effective in transferring moment across the connected members.

f) As a bolt pitch of 90 mm is found to give low moment resistance with large connection rotation and member deformation, it is thus not recommended to be used in moment connections.
CONCLUSIONS
In order to enable moment framings for cold-formed steel sections in building applications, a bolted moment connection configuration for double lipped C sections back-to-back was proposed. A total of eight internal and external beam-column sub-frame tests were executed and three different modes of failure were identified in the tests. The moment resistances of the proposed connection configuration were found to range from 36% to 97% of the moment capacities of the connected sections. Consequently, it is demonstrated that bolted moment connections are structurally feasible and economical. Furthermore, structural members with double lipped C sections back-to-back are shown to be practical in building short to medium span portal frames with bolted moment connections through rational design.

ACKNOWLEDGEMENT
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REFERENCES
2. AISI Specification for the design of cold-formed steel structural members, 1996 edition, American Iron and Steel Institute, Washington DC.
Table 1 Summary of test program and test data

<table>
<thead>
<tr>
<th>Test</th>
<th>Section</th>
<th>Maximum applied force (kN)</th>
<th>Failure mode</th>
<th>Maximum moment resistance Moment (kNm)</th>
<th>Rotation (rad.)</th>
<th>Member Thickness (mm)</th>
<th>Yield strength (N/mm²)</th>
<th>Gusset plate Thickness (mm)</th>
<th>Yield strength (N/mm²)</th>
<th>Centreline of connection Normalised moment (kNm)</th>
<th>Failure position of connection Normalised moment (CFS) (HRS)</th>
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<tr>
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<td>388</td>
<td>19.50</td>
<td>0.91</td>
<td>18.77</td>
<td>0.88</td>
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Notes:

- S denotes an internal beam-column sub-frames with a 'cross' shaped gusset plate under lateral load
- E denotes an external beam-column sub-frames with a 'tee' shaped gusset plate under lateral load
- 90 denotes a bolt pitch of 90 mm
- 180 denotes a bolt pitch of 180 mm
- 240 denotes a bolt pitch of 240 mm
- A denotes 4 bolts per member
- C denotes 4 bolts per member (same as A)
- 1 denotes the thickness of gusset plate thickness at 10 mm
- 2 denotes the thickness of gusset plate thickness at 16 mm

The measured moment capacities of C15016DS G450 and C15020DS G450 are 16.95 kNm and 21.36 kNm respectively.
a) Internal beam-column sub-frames.

b) External beam-column sub-frames.

Figure 1    General arrangement of beam-column sub-frames.
Figure 2  BFcsw  Bearing failure in section web around bolt hole

Figure 3  FFcs  Flexural failure of connected cold-formed steel section
Figure 4  Load deflection curves of tests S090A1, S180A1 and S240A1.

Figure 5  Moment rotation curves of test S180A1.
Figure 6 Level arm coefficient in lateral loading tests.
Figure 7  Moment rotation curves of internal beam-column sub-frame tests.

Figure 8  Moment rotation curves of external beam-column sub-frame tests.
Appendix B  Published technical papers in international conferences

B4. Experimental investigation on bolted moment connections in beam-column sub-frames - Comparative study
EXPERIMENTAL INVESTIGATION OF COLD-FORMED STEEL BEAM-COLUMN SUB-FRAMES: COMPARATIVE STUDY

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Abstract

In order to extend the effective use of cold-formed steel in building applications, a pilot study of eight tests with three different connection configurations in both internal and external beam-column sub-frames were carried out by Wong and Chung (2000) and the basic configuration of bolted moment connections between double tipped C-sections was established.

The present paper reports an extension of the experimental investigation of cold-formed steel internal and external beam-column sub-frames under both gravity and lateral loads. It aims to establish the high structural performance of bolted moment connections between cold-formed steel members in practical member orientations. A total of eight beam-column sub-frames with different connection configurations were carried out. It was found that the maximum moment resistances of the proposed bolted moment connections was over 90% of the moment capacities of the connected members, demonstrating high structural efficiency of bolted moment connections between cold-formed steel sections.

Introduction

Cold-formed steel sections are light-weight materials and suitable for low-rise building construction owing to its high buildability as reported by Yu (1991). The most common applications of cold-formed steel sections are purlins and side rails in industrial buildings which support both roof and wall claddings as building envelopes. To date, many design recommendations such as AISI (1996), AS/NZ 4600 (1996), Eurocode 3: Part 1.3 (1996), and BS5950: Part 5 (1998) for cold-formed steel structures present primarily design expressions on the load carrying capacities of fasteners and fixings. There is little design guidance on the strength and the stiffness of bolted moment connections between cold-formed steel sections.

In order to extend the effective use of cold-formed steel in building applications, it is highly desirable to build safe and economical moment frames, and thus design guidance on bolted moment connections in beam-column sub-frames with practical connection configurations should be provided. Several investigations on cold-formed steel members with bolted moment connections have been reported in the literature by Zhao and Hancock (1991), Chung and Lau (1999) and Wheeler, Clarke and Hancock (1999).
A pilot study of eight tests with three different connection configurations in both internal and external beam-column sub-frames under lateral load were carried out by Wong and Chung (2000) and the basic configuration of bolted moment connections between double lipped C-sections was thus established. The moment resistances of the connections were found to be typically 0.70 of the moment capacities of the connected sections.

The present paper reports an extension of the experimental investigation of cold-formed steel beam-column sub-frames under both gravity and lateral loads. A total of eight tests with different connection configurations in both internal and external beam-column sub-frames were carried out. It aims to demonstrate the high structural performance of bolted moment connections between cold-formed steel members, and also to establish the structural efficiency of bolted moment connections in practical member orientations.

Test program

The basic configuration of bolted moment connections proposed in beam-column sub-frames for building applications is:

- All structural members such as beams and columns are formed with two lipped C-sections back-to-back with interconnections.
- Moment connections between beams and columns are formed with hot-rolled steel gusset plates.
- Only the webs of lipped C-sections are bolted onto gusset plates.
- Four bolts per member are used.

The connection configuration is rationalized after considering ease of fabrication and installation. In general, the proposed moment connections are not able to develop full moment capacity of the connected sections due to discontinuity of load paths along section flanges in the sections.

A total of eight test specimens were executed with four test specimens under gravity load and the other four under lateral load. All the test specimens were constructed according to the basic configuration with systemic variation in the connection details, i.e. thickness and shape of gusset plates. Two different sections with different steel grades are used in the test series:

- Gravity load series
  C15016 DS G300 denotes a double section of lipped C-sections of 150 mm section depth and 64 mm flange width with a thickness of 1.6 mm. The design yield strength is 300 N/mm² and the moment capacity measured from four point load tests is 10.86 kNm.

- Lateral load series
  C15020 DS G450 denotes a double section of lipped C-sections of 150 mm section depth and 60 mm flange width with a thickness of 2.0 mm. The design yield strength is 450 N/mm² and the moment capacity measured from four point load tests is 21.36 kNm.
All bolts are 16mm in diameter and of Grade 8.8 and the bolt pitch is fixed at 180 mm in all tests which is the optimal value established in the pilot study in relation to the section depth of 150 mm. Details of the test program is summarized in Table 1.

Test procedures

The general arrangements of the test set-up together with the proposed connection configurations for all the beam-column sub-frames are shown in Figure 1. Both the applied load and the displacements of each member of the test specimens were measured during the entire deformation history. Any out-of-plane deformation was prohibited by two sets of restraining rollers which were installed in front of and behind the test specimens. The tests were terminated when either section failure or member buckling occurred.

Test results

Three different modes of failure were identified among the eight tests:

- $BFsw$ Bearing failure in section web around bolt hole.
- $FFgp$ Flexural failure of hot-rolled steel gusset plate.
- $FFcs$ Flexural failure of cold-formed steel section, as shown in Figure 2.

The maximum moment resistances of the test specimens may be evaluated at two locations, namely, at the centreline and at the failure position of the connections for different purposes. The centreline evaluation enables easy comparison of the moment resistances of the connections against applied moments obtained directly from conventional structural analysis while the failure position evaluation is required for connection design. In general, the moment resistances are first evaluated at the centreline of the connections, and level arm coefficients are then applied according to the associated failure modes to give the moment resistances of the connections at the failure positions. Typical moment-rotation curves of the test specimens are illustrated in Figure 3, and details of the level arm coefficients may be found in Figure 4.

For test specimens in the gravity load series, combined bending and shear interaction is expected to be significant due to short shear span. However, for test specimens in the lateral load series, combined bending and shear interaction is not significant due to low shear force in the connections, and thus neglected.

The results of all the eight tests are also summarized in Table 1 together with the measured material properties and dimensions of both the cold-formed steel sections and the gusset plates. All the measured moment resistances are normalized with the ratio of design yield strength and design thickness to measured yield strength and measured thickness of the test specimens. For test specimens with excessive deformation under testing, the moment resistance of the connection is restricted to be the applied moment at a connection rotation of 0.05 radian.
Comparisons and discussions

In order to assess the effectiveness of the bolted moment connections, a moment resistance ratio, \( \psi \), is established which is defined as follows:

\[
\psi = \frac{\text{Measured moment resistance of a connection}}{\text{Measured moment capacity of connected section}}
\]

After data analysis, the moment-rotation curves of the beam-column sub-frame tests under gravity and lateral loads are presented in Figure 5 for easy of comparison. The results of all the test specimens are compared among each other and the findings are presented as follows:

a) Tests G180A1 and G180A2
In both tests, flexural failure in connected cold-formed steel sections was apparent. The moment resistance ratios, \( \psi \), of the connections in tests G180A1 and G180A2 were found to be 0.81 and 0.77 respectively. No increase in the moment resistance ratio was found with an increase in thickness in the gusset plates from 10 mm to 16 mm as the critical mode of failure in both tests was the flexural failure of the connected cold-formed steel sections under combined bending and shear.

b) Tests G180D1 and G180D4
In both tests, flexural failure in connected cold-formed steel sections was apparent. The moment resistance ratios, \( \psi \), of the connections in tests G180D1 and G180D4 were found to be 0.78 and 0.83 respectively. No reduction in the moment resistance ratio was found with an reduction in thickness in the gusset plates from 10 mm to 6mm as the critical mode of failure in both tests was the flexural failure of the connected cold-formed steel sections under combined bending and shear. It should be noted that chambers of 50 mm in size were provided in the gusset plates.

c) Tests S180A1 and E180C1
In both tests, gross bending deformation with cracking in the corners of the hot rolled steel gusset plates was apparent. The moment resistance ratios, \( \psi \), of the connections in tests S180A1 and E180C1 were found to be 0.78 and 0.93 respectively when compared with the estimated moment capacities of the hot rolled steel gusset plates.

d) Tests S180A2 and E180C2
In both tests, flexural failure in connected cold-formed steel sections was apparent. The moment resistance ratios, \( \psi \), of the connections in tests S180A2 and E180C2 were found to be 0.92 and 0.97 respectively. Consequently, it was shown that by the use of thick gusset plates, flexural failure of gusset plates may be eliminated. The minimum moment resistance ratio of the connections is then increased from 0.69 to 0.92, with flexural failure in connected sections being critical.

Wong and Chung
By comparing the results of all the test specimens under gravity load, it was found that flexural failure under combined bending and shear in cold-formed steel sections was always critical due to their low yield strength, when compared with the hot rolled steel gusset plates. The minimum moment resistance ratio of the connections under combined bending and shear is 0.77.

It should also be noted that while a 'cross' shaped gusset plate of 10 mm thick is sufficient to form an efficient connection, the increase in thickness of the gusset plate from 10 mm to 16 mm does not increase the moment resistance ratio of the connection, as shown in tests G180A1 and G180A2. Furthermore, the effectiveness of the 10 mm thick 'cross' shaped gusset plate may equally be provided by a 'cross' shaped gusset plate of 6 mm thick with 50 mm chambers, i.e. a gusset plate of reduced thickness but with an engineered shape, as shown in test G180D4.

By comparing the results of all the test specimens under lateral load, it was found that flexural failure in cold-formed steel sections in connections with 16 mm thick gusset plates was always critical while flexural failure of gusset plates was critical in connections with 10 mm thick gusset plates.

The maximum moment resistance ratio in connections with flexural failure in connected cold-formed steel sections is 0.97, demonstrating high structural efficiency of bolted moment connections between cold-formed steel sections.

Conclusions

In order to enable moment frames for cold-formed steel sections in building applications, a bolted moment connection configuration for double lipped C-sections back-to-back was proposed. A total of eight internal and external beam-column sub-frame tests were executed under both gravity and lateral loads. Three different modes of failure were identified in the tests, and the moment resistances at the actual failure positions of the proposed connection configurations were found to range from about 0.77 to 0.97 of the moment capacities of connected sections.

The basic configuration for bolted moment connections is thus established for general application and it is demonstrated to be structurally efficient in both external and internal beam-column sub-frames and under both gravity and lateral loads. While the structural performance of the bolted moment connections depends largely on the steel grade and thickness of both the cold-formed steel sections and the hot rolled steel gusset plates, a moment resistance ratio of 0.75 is always achieved. The moment resistance ratio may readily be increased to 0.90 through rational design in the connection configuration.
Acknowledgements

The research project leading to the publication of this paper is supported by the Research Grants Council of the Hong Kong Government of the Special Administrative Region (Project No. PolyU5031/98E), and also by the Research Committee of the Hong Kong Polytechnic University Research (Project No. G-V750). The tests were carried out at the Heavy Structure Laboratory of the Department of Civil and Structural Engineering, the Hong Kong Polytechnic University. The authors would like to express their gratitude to Mr W.K. Yu and Mr K.W. Chow and also the technicians of the Heavy Structure Laboratory for the execution of the tests. The test specimens were supplied and fabricated by the P & L's Engineering Co. Ltd.

References


Load and resistance factor design specification for cold-formed steel structural members, LRFD Cold-formed Steel Design Manual, Part 1, American Iron and Steel Institute, Washington DC, 1996.


Wong and Chung
Table 1 Summary of test program and test data

<table>
<thead>
<tr>
<th>Test</th>
<th>Section</th>
<th>Maximum applied force (kN)</th>
<th>Failure mode</th>
<th>Maximum moment resistance</th>
<th>Member</th>
<th>Gusset plate</th>
<th>Centreline of connection</th>
<th>Failure position of connection</th>
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Notes:
- **G** denotes an internal beam-column sub-frames with a 'cross' shaped gusset plate under gravity load
- **S** denotes an internal beam-column sub-frames with a 'cross' shaped gusset plate under lateral load
- **E** denotes an external beam-column sub-frames with a 'tee' shaped gusset plate under lateral load
- **180** denotes a bolt pitch of 180 mm
- **A** denotes 4 bolts per member
- **C** denotes 4 bolts per member
- **D** denotes 4 bolts per member with chamfers in gusset plate
- **1** denotes the thickness of gusset plate thickness is 10 mm
- **2** denotes the thickness of gusset plate thickness is 16 mm
- **4** denotes the thickness of gusset plate thickness is 6 mm

The measured moment capacities of C15016DS G300 and C1020DS G450 are 10.86 kNm and 21.36 kNm respectively.
Figure 1 Test set-up and instrumentation of beam-column sub-frames.
Figure 4  Level arm coefficient in gravity and lateral load tests.

Figure 5  Moment rotation curves of beam-column sub-frame tests under gravity and lateral loads.
Appendix B Published technical papers in international conferences

B5. Experimental investigation on bolted moment connections in beam-column sub-frames - Enchanced performance
EXPERIMENTAL INVESTIGATION OF COLD-FORMED STEEL BEAM-COLUMN SUB-FRAMES: ENHANCED PERFORMANCE

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Department of Civil and Structural Engineering, the Hong Kong Polytechnic University, Hong Kong, China.

ABSTRACT

This paper presents the findings of an experimental investigation on cold-formed steel beam-column sub-frames with bolted connections engineered for high structural efficiency and buildability. Based on two previous investigations on a total of twelve beam-column sub-frame tests, the basic connection configurations of bolted moment connections with specific ranges of sizes of cold-formed steel sections and hot rolled steel gusset plates was established. In order to increase the structural performance of the proposed connections, another eight beam-column sub-frames with hot rolled steel gusset plates of engineered shape were executed. It was found from the tests that for connections with thick gusset plates, flexural failure of cold-formed steel sections was critical and the moment resistance of the connections was found to be over 85% of the moment capacity of the cold-formed steel sections. It was thus demonstrated that through rational design and construction, effective moment connections between cold-formed steel sections may be readily achieved for practical building applications.

KEYWORDS

Cold-formed steel moment connections, beam-column sub-frames, experimental investigation, C-sections back-to-back

INTRODUCTION

In order to extend the effective use of cold-formed steel in building application, a research project on the structural performance of bolted moment connections in beam-column sub-frames was undertaken. Two previous experimental investigations on a total of twelve beam-column sub-frame tests were executed by Wong and Chung (2000a and 2000b), and the basic connection configurations of bolted moment connections with specific ranges of sizes of cold-formed steel sections and hot rolled steel gusset plates was established as follows:

- All structural members such as beams and columns are formed with two lipped C-sections back-to-back with interconnections.
- Moment connections between beams and columns are formed with hot-rolled steel gusset plates.
- Only the webs of lipped C-sections are bolted onto gusset plates.
- Four bolts per member are used.

The connection details are rationalised after considering ease of fabrication and installation. In general, the proposed moment connections are not able to develop full moment capacity of the connected members due to discontinuity of load paths along section flanges in the sections. It is aimed to develop bolted moment connections with effective use of materials to mobilise at least 75% of the moment capacity of the connected sections. Experimental investigations into connections between cold-formed steel sections are also reported by Bryan (1993), Zhao and Hancock (1991), Chung and Lau (1999), and also Wheeler, Clarke and Hancock (1999).

Based on the previous two experimental investigations on bolted moment connections on both internal and external beam-column sub-frames under both lateral and gravity loads, the maximum moment resistance of the proposed connections was found to range from 75% to 85% of the moment capacities of the connected sections. This paper presents the findings of another experimental investigation on the structural performance of engineered bolted moment connections. A total of eight internal and external beam-column sub-frames with hot rolled steel gusset plates of 50 mm chambers were executed. Comparisons among the test results together with recommendations on bolted connections between cold-formed steel sections were also presented.

TEST PROGRAM AND TEST PROCEDURES

In order to examine the structural performance of bolted moment connections against bolt pitches, two internal beam-column sub-frame tests with different pitches of 180 mm and 240 mm were carried out, i.e. S180D1 and S240D1; the thickness of the gusset plates is 10 mm. Two external beam-column sub-frame tests with similar connection configurations were also carried out for comparison, i.e. E180D1 and E240D1. Furthermore, in order to examine the structural performance of bolted moment connections against gusset plate thickness, two internal beam-column sub-frame tests with gusset plates of 6 mm thick were carried out, i.e. S180D4 and S240D4. Two additional external beam-column sub-frames with a gusset plate of 6 mm thick were also carried out for comparison, i.e. S180D4 and E240D4.

In the present investigation, all the test specimens are constructed according to the proposed basic configuration. All bolts are 16mm in diameter and of Grade 8.8. A double section of two lipped C-sections back-to-back with a section depth of 150 mm, a flange width of 60 mm and a thickness of 2.0 mm is used in all tests; the sections are designated as C15020 DS. The yield strength of the sections is 450 N/mm² which is designated as G450. The moment capacity of the double section obtained from four point load tests is 21.36 kNm. Details of the test series are summarised in Table 1. Figure 1 illustrates the general arrangements of the test set-up together with the proposed connection configurations for both the internal and the external beam-column sub-frames. Both the applied load and the displacements of each member of the test specimens were measured during the entire deformation history. The tests were terminated when either section failure or member-buckling occurred, or deformation of the test specimens became excessive, i.e. over 250 mm.

TEST RESULTS

The results of all the eight tests are summarised in Table 1 together with the measured material properties and dimensions of both the cold-formed steel sections and the hot rolled steel gusset plates. Among the eight tests, two different modes of failure were identified:
• *FFcs* Flexural failure of cold-formed steel section, as shown in Figure 2
• *LTBgp* Lateral torsional buckling of hot-rolled steel gusset plate, as shown in Figure 3

The maximum moment resistances of the test specimens may be evaluated at two locations, namely, at the centreline and at the failure position of the connections for different purposes. The centreline evaluation enables easy comparison of the moment resistances of the connections against applied moments obtained directly from conventional structural analysis while the failure position evaluation is required for connection design. In the data analysis, the moment resistances are first evaluated at the centreline of the connections, and level arm coefficients are then applied according to the associated failure modes to give the moment resistances of the connections at the failure positions. For test specimens with excessive deformation under testing, the moment resistance of the connection is restricted to be the applied moment at a connection rotation of 0.05 radian.

**COMPARISONS AND DISCUSSIONS**

After data analysis, the moment rotation curves of the internal and the external beam-column subframes are presented in Figure 4. In order to assess the effectiveness of the bolted moment connections, a moment resistance ratio, \( \Psi \), is established which is defined as follows:

\[
\Psi = \frac{\text{Measured moment resistance of a connection}}{\text{Measured moment capacity of connected section}}
\]

The results of all the test specimens are compared among each other and the findings are presented as follows:

a) Tests with a bolt pitch of 180 mm

**Tests S180D1 and E180D1 - 10 mm thick gusset plates**
In both tests, flexural failure in the connected cold-formed steel sections was apparent. The moment resistance ratios, \( \Psi \), at the failure position of the connections in tests *S180D1* and *E180D1* were found to be 0.87 and 0.92 respectively.

**Tests S180D4 and E180D4 - 6 mm thick gusset plates**
In both tests, lateral torsional buckling of the hot rolled steel gusset plates was critical. The moment resistance ratios, \( \Psi \), at the failure position of the connections in tests *S180D4* and *E180D4* were found to be 0.65 and 0.64 respectively.

b) Tests with a bolt pitch of 240 mm

**Tests S240D1 and E240D1 - 10 mm thick gusset plates**
In both tests, flexural failure in the connected cold-formed steel sections was apparent. The moment resistance ratios, \( \Psi \), at the failure position of the connections in tests *S240D1* and *E240D1* were found to be 0.92 and 0.87 respectively.

**Tests S240D4 and E240D4 - 6 mm thick gusset plates**
In both tests, lateral torsional buckling of the hot rolled steel gusset plates was critical. The moment resistance ratios, \( \Psi \), at the failure position of the connections in tests *S240D4* and *E240D4* were found to be 0.67 and 0.62 respectively.
c) For connections with thick gusset plates, it is shown that flexural failure of the connected cold-formed steel sections is always critical, and the corresponding moment resistance ratio of the connections, $\psi$, is 0.85. This is regarded as a favourable mode of failure with high structural efficiency. For connections with thin gusset plates, lateral torsional buckling of the hot rolled steel gusset plates is always critical, and the corresponding moment resistance ratio, $\psi$, of the connections is about 0.60. It is regarded to be inefficient with low structural efficiency.

d) For connections with similar configurations but with different bolt pitches, it is shown that there is little difference in the moment resistance ratios. Thus, the bolt pitch of 180 mm may be considered as an optimal value for effective moment connections between sections C15020 DS.

CONCLUSIONS

Based on the findings of the experimental investigation, it is concluded that bolted moment connections between cold-formed steel sections are readily achieved using the proposed connection configurations. The bolted moment connections are demonstrated to be effective in transmitting moment between the connected sections, and thus enabling effective moment framings in cold-formed steel structures. Engineers are encouraged to build light-weight low to medium rise moment frames with cold-formed steel sections.

ACKNOWLEDGEMENTS

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REFERENCES

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<tr>
<th>Test</th>
<th>Section</th>
<th>Maximum applied force * (kN)</th>
<th>Failure mode</th>
<th>Maximum moment resistance</th>
<th>Moment resistance at 0.05 rad.</th>
<th>Member</th>
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Notes:

- S denotes an internal beam-column sub-frame with a 'cross' shaped gusset plate under lateral load
- E denotes an external beam-column sub-frame with a 'tee' shaped gusset plate under lateral load
- 180 denotes a bolt pitch of 180 mm
- 240 denotes a bolt pitch of 240 mm
- D denotes 4 bolts per member with 50 mm chamfers in gusset plate
- I denotes a 10 mm thick gusset plate
- 4 denotes a 6 mm thick gusset plate

The measured moment capacity of C15020DS G450 is 21.36 kNm
Internal beam-column sub-frame tests with 50 mm chamfers in hot-rolled steel 'cross' shaped gusset plates.

External beam-column sub-frame tests with 50 mm chamfers in hot-rolled steel 'tee' shaped gusset plates.

Figure 1  General set-up of beam-column sub-frame tests.
Figure 2  Flexural failure of connected cold-formed steel sections.

Figure 3  Lateral torsional buckling of hot-rolled steel gusset plate.
Figure 4a  Moment rotation curves of beam-column sub-frames tests with 10 mm thick gusset plate.

Figure 4b  Moment rotation curves of beam-column sub-frames tests with 6 mm thick gusset plate.