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# BOLTED CONNECTION OF COLD-FORMED STEEL SECTION - A REVIEW

Rahima Ummi Kulsum Nadya[1] and Fathoni Usman[2]
[1]College of Graduate Studies, Universiti Tenaga Nasional (UNITEN), Kajang, Malaysia
[2]Institute of Energy Infrastructure, Universiti Tenaga Nasional (UNITEN), Kajang, Malaysia
E-Mail: Rahimaummi19@gmail.com

## ABSTRACT

Cold-formed steel (CFS) sections are lightweight material that are made by rolling or pressing thin gauges of sheet steel into goods in a room temperature environment that are usually manufactured into channel sections, Z-sections, hat sections and some other open sections. CFS usage has increased in the recent years as a structural frame not only for residential buildings but also for multi-storey commercial buildings. Therefore, the understanding of the behaviour of CFS sections especially in its connection are important to be studied. This paper describes the behaviour of CFS sections by reviewing the previous studies specifically on bolted connection of cold-formed steel sections, emphasizing on the design guidelines from Euro code 3: BS EN 1993-1-3 and Euro code 3: BS EN 1993-1-8. The findings from previous studies are discussed in this paper.

**Keywords:** review, cold-formed steel, bolted connection, finite element modelling.

## 1. INTRODUCTION

Nowadays, many construction industries went for a more sustainable development by using lesser materials in constructions. The same steel that enables manufacturers to make lighter, more fuel-efficient vehicles, and taller, safer structures is also the most recycled material in the world. While competing materials focus their sustainability claims on specific phases of product application, superior sustainability performance of steel member minimizes environmental impact when measured through the entire life cycle. Structural steel is generally divided into hot-rolled sections and cold-formed sections [1]. Hot-rolled steel sections are formed at high temperature up to 1400 °C in blast furnace or electric arc furnace, while the cold formed steel sections are manufactured by rolling or pressing thin gauges of sheet steel into goods at room temperature environments [2]. The method of steel manufacturing is very important as it differentiates the properties of hot-rolled and cold-formed steel disparity in strength, structural performance, and failure mode.

In general, steel has become an essential material for manufacturers in the automotive, energy, machinery and equipment, container, appliance and rail industries. Steel is a critical building material for the nation's energy, transportation and water infrastructure and also for commercial and residential construction. New products and technologies including the Industrial Building System (IBS) seem to gain some tractions in the market. According to Malaysian Iron and Steel Industry Federation (MISIF), in 2016, Malaysia imported 9.13 million MT of the steel products, a surge of 15.3 % from year 2015. The major imported steel products are hot-rolled, bars and cold-rolled steel where 24% imported steel are hot-rolled, 20% are bars and 15% are cold-rolled steel. However, export of steel products declined 2.7% from 2.3 million MT in 2015 to 2.2 million MT in 2016, 41% within the ASEAN region. Major export of the steel products are cold-rolled, pipes & tubes and galvanized steel sheet where 25% exports are cold rolled sheets and strips, 23% pipes and tubes and 11% galvanised sheets. Based on this statistics, it still shows that cold-rolled sections or cold-formed steel sections are used widely in Malaysia even though the steel exports are declining.

Cold-formed steel (CFS) sections are lightweight materials suitable for building constructions that are usually manufactured into channel sections, Z-sections, hat sections and some other open sections by cold-rolling or brake-pressing technique [3]. Section thicknesses of typically ranging from 1.0 mm to 3.0 mm, cold- formed members have been fabricated with a common yield stress of 350 MPa for normal steel and recently up to 550 MPa for high strength steel [6]. The increase of the yield strength is due to strain hardening and depends on the type of steel used for cold rolling [7]. On the contrary, the increase of the ultimate strength is related to strain aging, which is accompanied by a decrease of the ductility and depends on the metallurgical properties of the material.

CFS has been produced for more than a century since the first flat sheet of steel was produced by the steel mills. The use of CFS members in building construction began in the 1850s in both the United States and Great Britain. In the early 1920s and 1930s, acceptance of CFS as a construction material was still limited because there was no adequate design standard and limited information on material use in building codes. One of the first documented uses of CFS as a building material is the Virginia Baptist Hospital constructed around 1925 in Lynchburg, Virginia [2]. Since then, the application of cold-formed steel have come a long way.

In the past few decades, there are an increasing number of CFS usage as a structural frame not only for residential buildings but also for multi-storey commercial buildings, e.g., roof systems, wall studs, girts, and steel framed housing [8]. This is due to the inherent features of cold-formed steel that overcome the downsides of conventional products. Their strength, light weight, versatility, non-combustibility, and ease of production

have encouraged architects, engineers, and contractors to use CFS products, which can improve structural function and building performance and provide aesthetic appeal at a lower cost [5]. In building construction, hot-rolled steel sections are commonly used as primary structural frames while CFS sections are used as secondary structural members to support claddings in forming external building envelopes [6].

For typical applications of cold formed steel sections, there are many design code on cold formed steel structures available such as AISI (1996), BS5950 (1998) and Eurocode 3: Part 1-3 (2006). Moreover, a number of design guides and commentaries are also available to assist structural engineers to design cold formed steel structures. Design guidelines, i.e., Eurocode 3 (EN 1993-1-3) and Eurocode 3 (EN 1993-1-8) are used for the connection design of CFS members in the European Countries while American Iron and Steel Institute (AISI) is used by research institutes in United States of America.
Cold formed steel members have unique structural stability issues primarily due to their large width to thickness comparison element ratios, which is not common in hot-rolled steel. Unlike joints for hot-rolled steel structures, design guidelines for cold-formed steel connections are limited to their fundamental behaviour [9]. Detailed design procedures for cold-formed steel joint profiles are uncommon due to their wide variance and specific purpose, and most designs are made based on testing results. Since many researches have been carried out and as the significance of cold-formed steel is concerned, this paper describes the behaviour of CFS sections by reviewing the studies that have been done specifically on bolted connection of cold-formed steel sections. This paper focuses on the design guidelines from Eurocode 3: BS EN 1993-1-3 and Eurocode 3: BS EN 1993-1-8 as Malaysia follows the design standard from United Kingdom. Eurocode 3: BS EN 1993-1-3:2006 can provide supplementary rules for cold-formed members and sheeting while Eurocode 3: BS EN 1993-1-8 is the design guidelines specific to connection.

## 2. CFS SECTIONS AND ITS APPLICATION

Cold-formed members and profiled sheets are steel products made from coated or uncoated hot-rolled or cold-rolled flat strips or coils. Within the permitted range of tolerances, they have constant or variable cross section [10]. Cold-formed structural members can be classified into two major types, i.e., individual structural framing members, panels and decks. The individual structural members (bar members) include single open sections (Figure-1.a); open built-up sections (Figure-1.b) and closed built-up sections (Figure-1.c).

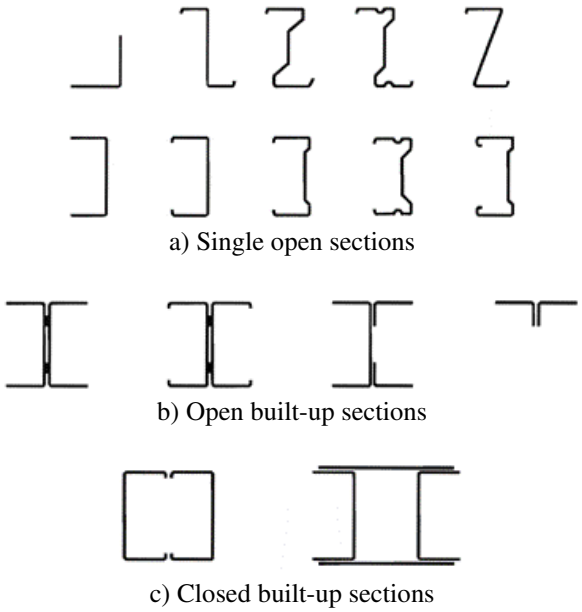



**Figure-1.** Typical forms of sections for cold-formed members [10].

Usually, the depth of cold-formed sections for bar members ranges from 50-70 mm to 350-400 mm, with thickness from about 0.5 mm to 6 mm. Panels and decks are made from profiled sheets and linear trays (cassettes) and the depth of panels usually ranges from 20 to 200 mm, while thickness is from 0.4 to 1.5 mm. In order to increase the stiffness of both cold-formed steel sections and sheeting, edge and intermediate stiffeners are used. These stiffeners can enhance the strength of sections by acting as the out-of-plane supports to the flat plate elements of sections. The stiffeners can improve the efficiency of the CFS material up to 50% supported by the study conducted by [11].

The advantages of using cold-formed steel sections are high strength to weight ratio, flexibility in fabricating different cross-section shapes, easy for construction, versatility and it has a high structural efficiency [12]. CFS's also have a long-term durability together with high yield strength and high buildability [6]. Even though CFS sections have many advantages, there are also some disadvantages in using the cold-formed steel sections. The CFS members are more likely to undergo torsional deformation due to their low torsional rigidity resulting from their thin walls [13]. The plate elements constituting the cold-formed steel sections usually have large width-to-thickness ratio. Due to their typically large flat width-to-thickness ratios, CFS sections are inherently susceptible to local, distortional and global buckling modes, resulting in a complex optimization process, particularly in the fields of structural stability and joints [14].

## 3. BEHAVIOUR AND DESIGN RESISTANCE OF CFS

Steel sections may be subject to one of the four generic types of buckling, namely local, global, and

distortional and shear. The CFS sections to be optimized were evaluated according to the cross-sectional strength and stability provisions in BS EN 1993-1-3 [10] accounting for both local and distortional buckling modes. The individual components of cold-formed steel members are usually so thin with respect to their widths that they buckle at stress levels less than the yield point when subjected to compression, shear, bending or bearing [7]. Local buckling of such elements is therefore one of the major considerations in cold-formed steel design. The following section describes about the behaviour and the resistance of cross-section of CFS sections.

### 3.1 Resistance of cross-section of CFS

The design values of the internal forces and moments at each cross section shall not exceed the design values of the corresponding resistances. The design resistance of a cross section shall be determined either by calculation according to Eurocode 3 Part 1-3 or by design assisted by testing. Design assisted by testing may be used instead of design by calculation for any of these resistances. Design assisted by testing is particularly likely to be beneficial for cross sections with relatively high width to thickness ratios, e.g. in relation to inelastic behaviour, web crippling or shear lag. In members with cross sections that are susceptible to cross sectional distortion, account shall be taken of possible lateral buckling of compression flanges and lateral bending of flanges generally.

### 3.1.1 Bending moment

The design value of the bending moment MEd at each cross section should satisfy:

$$\frac{M_{Ed}}{M_{c,Rd}} \leq 1 \quad (1)$$

The moment resistance of a cross section for bending about a principal axis shall be obtained from the following:

- If the effective section modulus $W_{eff}$ is less than the gross elastic section modulus $W_{el}$:

$$M_{c,Rd} = \frac{W_{eff} \cdot f_{yb}}{\gamma_{Mo}} \quad (2)$$

- If the effective section modulus $W_{eff}$ is equal to the gross elastic section modulus $W_{el}$:

$$M_{c,Rd} = f_{yb}\,(W_{el} + (W_{pl} - W_{el})4\left(1 - \frac{\bar{\lambda}_{e\,max}}{\bar{\lambda}_{e0}}\right))/\gamma_{M0}$$

but not more than $W_{pl} f_{yb}/\gamma_{M0}$

Where,

$f_{yb}$ = basic yield strength;

$\bar{\lambda}_{e\,max}$ = slenderness of the element which correspond to the largest value of $\bar{\lambda}_e/\bar{\lambda}_e 0$

For stiffened elements

$$\bar{\lambda}_e = \bar{\lambda}_p \text{ and } \bar{\lambda}_{e0} = 0.5 + \sqrt{0.25 - 0.055(3 + \psi)}$$

Where $\psi$ is the stress ratio;

For unstiffened elements $\bar{\lambda}_e = \bar{\lambda}_p$ and $\bar{\lambda}_{e0} = 0.673$

### 3.1.2 Shear Force

Based on Eurocode 3: Part 1-3, the design value of the shear force $V_{Ed}$ at each cross section should satisfy:

$$\frac{V_{Ed}}{V_{b,Rd}} \leq 1 \quad (3)$$

Where $V_{b,Rd}$ is the design shear resistance. The design shear resistance $V_{b,\,Rd}$ shall be determined from:

$$V_{b,Rd} = \frac{\frac{h_w}{\sin\phi} . t . f_{bv}}{\gamma_{Mo}} \quad (4)$$

Where

$f_{bv}$ = shear buckling strength considering buckling according to Table 1;

$h_w$ = web height between the midlines of the flanges;

$\phi$ is the slope of the web relative to the flanges.

The shear buckling strength $f_{bv}$ for the appropriate value of the relative web slenderness $\lambda_w$ shall be obtained from Table-1.

**Table-1.** Shear buckling strength $f_{bv}$ [10].

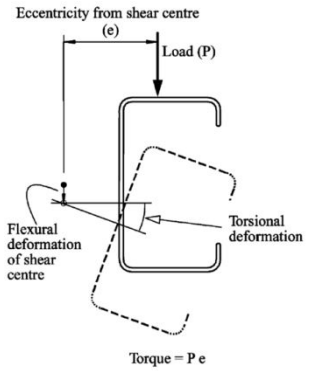
| Relative web slenderness | Web without stiffening at the support | Web with stiffening at the support [1] |
|---|---|---|
| $\lambda_w \leq 0.83$ | $0.58\ f_{yb}$ | $0.58\ f_{yb}$ |
| $0.83 < \lambda_w < 1.40$ | $0.48\ f_{yb} / \lambda_w$ | $0.48\ f_{yb} / \lambda_w$ |
| $\lambda_w \geq 1.40$ | $0.67\ f_{yb} / \lambda^2_w$ | $0.48\ f_{yb} / \lambda_w$ |
| [1] Stiffening at the support, such as cleats, arranged to prevent distortion of the web and designed to resist the support reaction. | | |

The relative web slenderness $\lambda_w$ shall be obtained from the following:

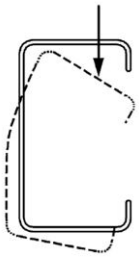
$$\lambda_w = \sqrt{\frac{f_{yb}/\sqrt{3}}{\tau_{cr}}} = \frac{s_w}{t}\sqrt{\frac{12(1-v^2)f_{yb}}{\sqrt{3}.\pi^2.E.k_\tau}} \quad (5)$$

### 3.1.3 Torsional moment

Due to the low torsional rigidity resulting from the slender walls of cold-formed steel section, an open section of CFS are predicted to go through torsional deformation. Further, the sections are often loaded eccentrically from their shear centres and so are subjected to considerable torques as shown in (Figure-2(a)) and sometimes include distortional buckling (Figure-2(b)). Furthermore, the sections are often loaded eccentrically from their shear centres and so are subjected to substantial torques.




(a)



(b)

**Figure-2.** Torsional and distortional deformations of channel sections [13].

## 4. DESIGN CONSIDERATION FOR CFS SECTIONS

As the industry demand for cold-formed steel grows over the past few years, many research studies were done to minimize the safety issue in the uses of cold-formed steel members as primary structural members, for example, roof truss, beam, and column member [9]. The research includes the investigation of buckling properties of cold-formed steel members, failure mode of the members, and strength and stiffness of the cold-formed steel member. The following sections describes the design consideration of CFS section connection according to Eurocode 3 Part 1-8.

### 4.1 Connection

A physical component which mechanically fastens structural elements and concentrated at the location where the fastening action occurs are known as connection [15]. It is important in transferring force and moment from a structural member to the supporting elements. Joint is defined as the connection plus the corresponding zone of interaction between the connected members and the panel zone of the column web. Structural joints can be categorized into several groups depending on its strength and also their stiffness. According to Eurocode 1993 Part 1-8 Clause 5.2.3, the strength of joints can be divided into nominally pinned, partial strength, and full strength classification. For stiffness behaviour, nominally pinned, semi-rigid, and rigid connections are classified according to EC3-1-8 Clause 5.2.2 [16].

Connections are important parts of every structure, not only from the point of view of structural behaviour but also in relation to the method of production [7]. It is always desirable that if any failure happens, the structural member is to fail first rather than the connection. If the connection fails before the failure of structural member, the results will be more catastrophic [17]. The connections for hot-rolled steel structures are divided into bolted connections and welded connections while for cold-formed steel structures, there are nine types of joint that are commonly used in the construction industry [15] namely, bolts, self-tapping screws, blind rivets, powder actuated pins, spot welding, puddle welding, clinching, self-piercing rivets, and nailing. The different types of connection contribute to different applications; for example, self-tapping screws, blind rivets, and powder actuated pins are used to fasten thin sheeting to sections, bolts are used to connect thicker cold-formed section, spot welding is used as factory joining of thin steel, and so forth. The characteristics of steel joint are easy to install, ready in the market, low cost of installation, and less maintenance needed. In Eurocode 3: Part 1-3; the types of joint models are divided into simple, continuous, and semi-continuous models. When a connection is classified as semi-continuous model, the connections flexibility should be estimated and included in the structural analysis in order to analyse the inner forces and displacement.

### 4.1.1 Failure mode of bolted connection

In the conventional analysis and design, connections in bare steel frames are often considered either as pinned or fixed. In the ideal state, pinned connections are assumed that no moment is transferred between the beam and the column. Since rotation is a great concern for such joints, the beam and the column that are connected together by a pin must conform to serviceability limit states [18]. The main failure modes for bolted connections are tear-out, bearing failure of sheet material, tension failure of net section, shear failure of bolts, and combinations of two or more of these failures [19]. These failure modes happen because of shear as shown in Figure-3. Tear-out failure occurs most in connections where the bolt is near the edge of the plate, or the distance between adjacent bolts parallel to the line of force is small. The

plate tears from a bolt hole to the edge of the plate or to another adjacent bolt hole. Furthermore, there is also another type of failure mode which is failure mode in tension. This failure mode is illustrated in Figure-4. To prevent this type of failure, design guidelines specify minimum edge and spacing distances for bolted connections. Hancock *et al.* [19] suggest that when the edge and spacing distances are large enough to avoid tear-out failure, bearing failure of the sheet could occur. Bearing failure often produces stretching of the hole on one side of the bolt, while the sheet material is bunched together on the other side of the bolt.

Rogers and Hancock have proven in an experiment that the use of washers under the bolt head and nut can significantly increase a connection resistance against bearing failure, and proposed that this is due to the commonly low thicknesses of CFS. When the stresses in the net section of the connected sheet are large enough, failure of the sheet can occur across the bolt hole. This type of failure occurs at the connection because it is often the cross-section of the sheet with the lowest net area, and hence it is the weakest section of the sheet. The stresses in the net section depend on the spacing arrangements, and the total number of bolts at a connection. The spacing arrangements usually determine the net section, particularly in solid sheets without perforations additional to the bolt holes. Bolt shear failure occurs when the grade of bolt used does not have shear strength high enough to resist loads beyond the load capacity of the sheet and/ or the load capacity of the connection arrangement as a whole. The bolt can fail in double or single shear depending on the connection arrangement. In this case the failure is brittle and therefore is undesirable.

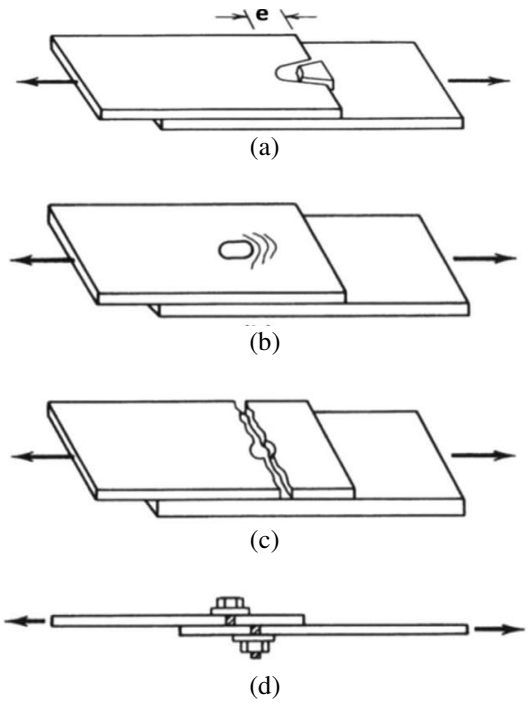



**Figure-3.** Failure modes of bolted connections in shear: a) tear-out failure of sheet, b) bearing failure of sheet, c) tension failure of net section, d) bolt shear failure [20].

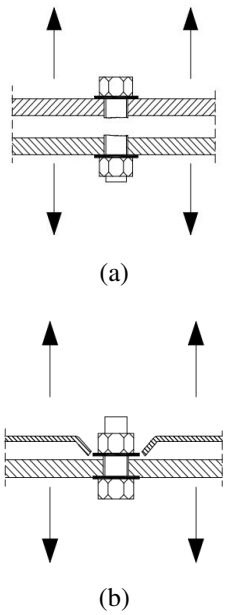



**Figure-4.** Failure modes of bolted connections in tension [7].

### 4.1.2 Design of bolted connection for CFS

Design procedure of connection of cold-formed steel section are according to EN 1993-1-8 [16]. Eurocode 3: Part 1-8, distinguishes between connections and also joints. However, for cold-formed steel sections, there is no significant distinction can be made between connections and joints. For cold-formed steel structures, elastic analysis is recommended for the design of joints. Bolted connections are normally used as shear, tension or moment resistant connections in cold-formed steel framing. Design of bolted connections can be done on the basis of the general formula (1), in which the design resistances given below must be used and the partial factor, γM2 = 1.25.

**Table-2.** Design resistance for individual fasteners subjected to shear and/or tension [16].

| Failure mode | Bolts |
|---|---|
| Shear resistance per shear plane | $F_{v,Rd} = \frac{\alpha_v f_{ub} A}{\gamma_{M2}}$ |
| Bearing resistance | $F_{b,Rd} = \frac{k_1 \alpha_b f_u dt}{\gamma_{M2}}$ |
| Tension resistance | $F_{t,Rd} = \frac{k_2 f_{ub} A_s}{\gamma_{M2}}$ |

The bearing resistance for bolts can be divided into two. For the bearing resistance, $F_{b,Rd}$ for bolts in an oversized holes, it is 0.8 times the bearing resistance for bolts in normal holes. Meanwhile, in a slotted holes, in slotted holes, where the longitudinal axis of the slotted hole is perpendicular to the direction of the force transfer, is 0.6 times the bearing resistance for bolts in round, normal holes.

## 5. PREVIOUS RESEARCH ON CFS BOLTED CONNECTION

Cold-formed sections are normally thin and consequently they have low torsional stiffness. Many researchers have been done for the past few decades to study the behaviour of bolted connection on cold-formed steel sections. Lightweight steel sections are being widely used in the form of thin-walled members with high strength-to-weight ratios to minimise the material cost needed for the construction but they are prone to buckling [21]. Hence, laboratory work and numerical modelling analysis can be used to determine the behaviour of cold form steel section specifically in bolted connection.

### 5.1 Laboratory testing and finite element modelling

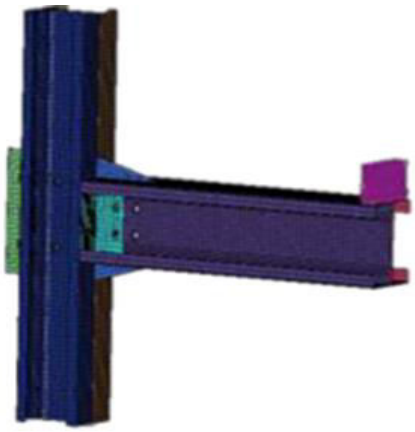
In an experimental study, [22] showed that the two lipped C-sections of the CFS were used as the beam and column members and found that out of 16 tested model, the moment resistance of 4 bolts per member was between 42% and 84% of the moment capacities of the connected CFS members. This indicated that the model configuration of two lipped C CFS sections back-to-back and with 4 bolts per member are sufficient and economical. Chung [23] also investigated the structural performances of the bolted connection of CFS sections in beam-column sub-frames. Eight test with different connection configurations were carried out where double lipped C-sections back to back with hot rolled steel gusset plate of 10mm and 16mm were tested. Both gusset plate have a different shapes and four bolts. Three different failure modes were identified. The measured moment resistance ranged from 36% to 97% making the design are structurally feasible and also economical.

Chung and Lawson [24] presented a different kind of CFS testing where web cleats of CFS strips were used as a connection between beam and column to investigate the structural performance of shear resisting connection of CFS section. Four different connection configurations were applied in a total of 24 testing in this study. It was found that web cleats can be used with bolts as practical shear resisting connection in a building construction. This allows for a simple and effective connection of CFS section to improve buildability in constructions. Chung and Ip [6] investigated finite element analysis of CFS bolted connections. From the analysis compared with codified rules, it was found that the design rules were not applicable for bolted connection with high strength steel due to reduction in ductility. Therefore, a semi empirical design formula for bearing resistance of bolted connection were proposed. It was shown that the proposed design will always give safe bearing resistance for CFS connection. Wong and Chung [25] demonstrated that the moment capacities of 20 column base connection test were 50% to 85%. This was effective in transmitting moment between the connected sections.

In a three-dimensional (3D) elastoplastic finite element study by Ju, Fan and Wu[26]that showed comparison between the Finite Element Modelling (FEM) results and the AISI data, it was found that when the steel reaches non-linear behaviour, the bolt nominal forces obtained from the finite element analyses are almost linearly proportional to the bolt number arranged in the connection. It was agreeable to neglect the deformation and the bolt bending effect when calculating the ultimate load for the bolt shear failure of the connection with an appropriate bolt spacing and end distance. Kartal, Bayraktar and Muvafik [27] explained about the rotational spring stiffness-connection ratio relation where a finite element program were developed for the numerical analysis that can define semi-rigid connection in terms of the rotational spring stiffness and it was found that semi-rigid beam-to-column and column-to-foundation connections were more effective on general structural behaviour than steel brace and truss member connections to joints.

A study by Ali, Saad and Osman [28] showed that the stiffness and performance of the column base connections had significant effect on the structural behaviour of the frames and an effective CFS framing can be constructed through a rational design. Meanwhile, Kaling, Patil and Hosur [17] focused on determining the structural performances of various beam-column with bolted moment connection on CFS section using FEM and experimental testing and the result showed the connection failed by torsional buckling system. Figure-5 describes the experimental set-up and the FEM model.

(a)



(b)

**Figure-5.** (a) Experimental set-up (b) FEM model [17].

In [18], Hassan et al., presents numerical investigation and modelling that studies the usage of cold-formed steel section as energy dissipative elements for moment resistance in buildings. A 2 meter cold-formed steel beams have been modelled in ANSYS to determine the rotational capacity of the cold-formed section. A web bolted to through plate connection has been assigned where the boundary conditions and constraints are shown in Figure-6.

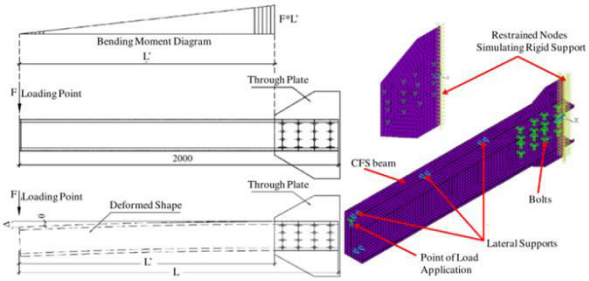



**Figure-6.** The boundary condition of modelled section [18].

The analysis of the numerical modelling shows that the behaviour of the cold-formed steel section will not enhance even though the number of flange bends have been increased. Meanwhile, from the results also, the stiffeners of the cold-formed steel sections can increase the seismic energy dissipation, strength, rotational capacity and the initial stiffness of the beam connection of cold-formed steel section.

A series of laboratory tests and finite element analysis were carried out in [29] to determine the ultimate strength of bolted moment connections between cold-formed steel members. Web buckling was concerned in design to predict the ultimate strength of the connections and cannot be ignored. A method that determines the initial rotational stiffness of shear-bolted moment connections between cold-formed steel members has been presented in the study by [30]. The predicted stiffness can be used in frame design with reasonable safety considerations. From the predicted strength and stiffness, beam idealization method was used to predict the structural behaviour of 2D-portalframe. A closed comparison with experimental results was found and was able to reduce the computational time while using beam idealization. Flexural strength and structural behaviour (moment-rotation relation, the yield, and ultimate moment capacity) were studied experimentally. Both experimental data and numerical data were conducted. Using secant stiffness, semi-rigid connection was applied to the analysis which was estimated from moment-rotation curves from tests.

Another study by Yu *et al*. [8], four column bases with different connection configurations were tested to investigate the moment resistance and also the typical modes of failure that happen to the model. A finite element model was generated using the ABAQUS (Version 6.4, 2004) software package was later established using shell and spring elements to model the sections and bolted fastenings respectively. The experimental test results and the numerical modelling results were compared. Grade 450 cold-formed double channel steel sections of size of 150 mm depth and 64 mm width were utilized as specimens in the test. The section's thickness was 1.6 mm and 2.0 mm respectively. 16 mm diameter of Grade 8.8 bolts were used in all the experimental test and numerical test. Hot-rolled steel plates of 16 mm and 20 mm thickness were formed into T-shape as column bases. Findings from this study showed that the proposed design and analysis method was structurally adequate to predict the failure loads of column-base under shear and bending.

A study were carried out by Tan and Tahir [31] in Universiti Teknologi Malaysia to develop the design procedures of bolted beam to column connections for double channel cold-formed steel sections. The pin and partial strength behaviour of the developed connections based on their strength and stiffness performance were studies and the performance of the proposed connection configurations were also being validated by comparing the analytical calculation to experimental results. A series of experimental investigation involving twenty-four isolated joint tests and twelve sub-assemble frame tests were carried out to analyse the connections' strength and also the stiffness behaviour. From the experimental results, good agreement was defined compared to theoretical predictions. From the experimental and analytical results, some of the connections were classified as pin joints, with the strengths less than 25% of beam capacity; while others were classified as partial strength joints with moment resistance of joints were in the range of 46% to 96% to the moment resistance of the connected beam.

Lim and Nethercot [30] presented a linear finite element model of cold-formed steel portal frame that become comparison to the experimental results. Two methods were implemented in the analysis, a full three dimensional linear shell element using ABAQUS software and linear beam elements with ANSYS program. Both these approaches are able to idealize the column and rafter members and rotational spring elements that are used to represent the rotational flexibility of the joints. Furthermore, the beam idealization took into consideration of the finite connection length of the joints. The expected deflection using the beam idealization was shown to be comparable to deflections obtained from both a linear finite element and laboratory tests. The expected deflection predicted using the beam elements also shown to be close to those predicted using more accurate shell model. As a conclusion from this study, using the beam idealization, engineers can analyse and design cold-formed steel portal frames, including making appropriate allowances for connection effects, without the need to resort to expensive finite element shell analysis.

## 6. CONCLUSIONS

The uses of CFS sections in the construction industries has increased in the recent years because of its strength, light weight, versatility, non-combustibility, and ease of production that made engineers, and contractors to use cold formed steel products, which can improve

structural function and building performance and provide aesthetic appeal at a lower cost.

In designing bolted connection according to Eurocode 3 Part 1-8, some design need to be considered to achieve the suitable design. Due to thin-walled behaviour, cold-formed steel exhibits different failure mode and large deformation as the buckling is the major concern of the connection structural analysis. However, the formulations in Eurocode, initially developed for hot-rolled steel joints, might be inaccurate when applied to the design of bolted cold-formed steel connections. Thus, comprehensive studies should be carried out to improve the reliability of connection design according to codes of practice for CFS sections. Researchers can used numerical modelling as an alternative to test numerous number of analysis in finding the behaviour of bolted connection of cold form steel section without incurring an extra cost as opposed to laboratory analysis which is quite expensive and time consuming. Nevertheless, laboratory analysis is still required to support numerical modelling data.

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