ISSN: 2454-9940

INTERNATIONAL JOURNAL OF APPLIED SCIENCE ENGINEERING AND MANAGEMENT

E-Mail :
editor.ijasem@gmail.com
editor@ijasem.org

www.ijasem.org

INTERNATIONAL JOURNAL OF APPLIED SCIENCE ENGINEERING AND MANAGEMENT

ISSN2454-9940

www.ijasem.org

Vol 12, Issue 3, 2018

Investigation of Parameters Affecting Rotational Behavior of Cold-Formed Steel Connection

Rajini Umapathi, Basavaraj R , Gangadhar Hugar

Asst. Professor, Asst. Professor, Asst. Professor

rajaniumapathi9441@gmail.com , basavaraja.banavikallu@gmail.com , ganguhugar@gmail.com

Department of Civil, Proudhadevaraya Institute of Technology, Abheraj Baldota Rd, Indiranagar, Hosapete, Karnataka-583225

Abstract

This study focusses on understanding the behaviour of connections, which is crucial for successful designs of cold-formed steel structures. Examining the features that affect the cold-formed steel connection's rotational stiffness and behaviour is our primary goal. The connection is made up of bolted and gusset-plate-fastened single-lipped channel pieces. For the numerical investigation, we relied on component-based finite element analysis. The criteria that were examined were bolt sizes, group size, number of bolts, thickness of the cold-formed steel cross-section, and thickness of the connecting plate. It is possible to evaluate and comprehend the components' effects via comparison. According to the results, the connection's rotational behaviour is dictated by the assembly details. With respect to the specified base connection, the effect of the bolt spacing parameter on rotation stiffness and moment capacity is minimal, while that of the number of bolts and bolt diameters is maximal. The bolt group size, however, is the single most critical component of the member connection. Both types of connections maintain their stiffness and strength regardless of changes in the thickness of the gusset plate or the cold-formed steel section. Bolt failure and plate failure caused by excessive bolt hole deformation are two examples of how connection assembly factors might classify failure types.

Keywords: Cold-Formed Steel; Bolt Connection; Rotational Stiffness; Parametric Study; Component-Based Finite Element Method.

1. Introduction

One of the interesting developments in steel structure work in recent years is the use of cold-formed steel sections either as structural elements or as supporting elements in the structure. Cold-formed steel contains many advantages that facilitate modern construction. First of all, the cold-formed steel section has high load resistance when compared to the weight of the part itself. The thinness and lightweight of the section ease its installation and assembly into a structure, making it possible to reduce construction labor [1]. Material preparation processes such as cutting, bending, and drilling holes for mounting bolts can be handled in the manufacturing plant, making the construction details precise and leading to easy quality control. Since both time and construction costs can be saved, the cold-formed steel very well meets the needs of modern construction, which focuses on construction speed.

One of the key factors in the construction of cold-formed steel structures is how the members are connected. Connection between cold-formed steel members and between a member and a support point can be done via many techniques, including welding, bolting, and riveting [2, 3]. Based on the bolting technique, bolt holes can be pre-fabricated in the plant, allowing easy, precise, and quick installation at the construction site. The bolt-gusset plate connection system allows the force or bending moment to be transferred through bolts, also causing bearing on holes. Since cold-formed steel sections are generally thin, local deformation at the bolt holes in the joint may appear [4–6]. This results in semi-rigid rather than rigid behavior of the connection [7–9].

The flexibility of the connection has an influence on force distribution and the overall deformation of the frame. Some full-scale experiments were set up in order to study the effect of connection flexibility on the overall performance of the cold-formed steel portal frame [10–12]. The results from the laboratory tests showed that the deflection at the apex joint could be closely predicted when considering the connection flexibility due to the bolt hole elongation into the model [10]. The most realistic global response could be obtained via a simple structural model with consideration of the connection stiffness [11]. Similar results were reported in Wrzesien et al. (2015) [12] study, which increase in rotational stiffness of the joints had a significant effect on the frame deformation; the apex deflection was reduced to almost half by doubling the stiffness in their experiments. From the experimental tests in [13], it was also observed that the stiffness of the column base bolted connections directly correlated with the load-bearing capacity of the cold-formed steel frames. From these results, it is essential to understand the behavior of the connection, as well as the behavior of the members, in order to understand the behavior of the overall structure.

The transmission of the bending moment through the connection can be expressed through the relationship between the bending moment and the rotational angle that occurs at the connection. The slope of such a graph is referred to as the rotational stiffness of the connection, which is an important parameter in designing cold-formed steel structures [6, 10, 14]. Although the so-called component method can be used for the determination of the strength, stiffness, and deformation capacity of individual components, which are then combined to obtain the overall joint behavior, current standards such as AISC 360-16 [15] or EN1993-1-8 [16] stipulate that the best method to determine the moment and rotational angle characteristics of joints is to set up laboratory testing. There have been some laboratory and numerical experiments to obtain the connected parts based on different steel sections and connection configurations. For the bolt connection system, there were experiments with the connection of single or built-up cold-formed steel sections with the help of gusset plates [17–19] or connected directly without gusset plates [20–21]. Based on experimental and numerical results, there have also been many attempts to propose empirical models and numerical models used to assess joint rotational behavior [22].

The rotational stiffness and rotational behavior depend on many variables and differ according to connection styles. According to prior research, there have been some studies on the variables that influence the strength, stiffness, and ductility of cold-formed steel bolted connections. Bolt-group size, bolt arrangement, number of bolts, size of cold- formed steel members, size of gusset plates, and addition of distinctive components were all investigated criteria. To date, some research has been conducted utilizing either numerical analysis or experimental experiments to understand the associated factors. The experiment in Hazlan et al. (2010) [20] indicated that adding more bolts and places of connection between members increased the moment resistance and stiffness of the connection. Similar results from [23] showed that an increase in the number of bolts, bolt spacing, and the thickness of the connecting members reduced deflection of the connecting members. For a bolt group, how bolts are arranged could also affect the stiffness of the joint. The optimum choice of bolt arrangement would be a square pattern or, preferably, a circular pattern. Using a proper bolt arrangement could delay bolt bearing failure and therefore increase the ultimate moment capacity and the ductility of the connection [24–26]. In Nagy et al.'s (2019) [27] work, a numerical study was performed using the component method [28], and some observations were noted. The rotational stiffness and the moment resistance increased as the member thickness and steel grade increased and as the diameter and steel grade of the bolts increased. Moreover, the size of the bolts showed some effects on the failure mode of the connection; applying sufficiently large bolts tended to cause failure of the steel profile, while failure of the bolts could be expected when using small bolts.

According to the finite element calculations, the joint strength increased as the thickness of the member or gusset plate increased. However, it did not guarantee that joint stiffness would increase [29]. Adjusting other variables, such as bolt spacing, was a better alternative for increasing moment capacity than increasing member thickness [30]. The experimental results of beam-to-column cold-formed steel connections revealed that the beam depth was another factor that affected the connection's ability to withstand the bending moment; as the depth of the beam increased, so did the connection's ability to withstand moments [20, 31]. However, the results were affected by the placement of the bolt attachment. The use of gusset plates with stiffeners or larger plates also increased the moment resistance capacity. From the experimental tests [32–34], the higher strength and rotational stiffness of the member connections could be obtained by applying bolts on the flange in addition to bolts on the web of the steel section. Adding a sleeve element was also a way to increase the moment resistance and deformation of the connection. However, different types of elements brought different levels of load resistance and ductility. It was found that the gusset connection provided the lowest moment resistance and ductility when compared with the lipped connection and the link connection [35].

In the preceding literature, many variables have been investigated and found to be influential on the behavior of bolted connections. However, there is still a shortage of knowledge on how to alter the flexibility of the connection in an efficient manner. The aim of this work was to bridge the gap by understanding the behavior of the bolted connections of cold-formed steel structures and comparing the influence of the relevant parameters on the connection behavior. The primary focus is on the connections that are simple yet ubiquitous in portal frame systems constructed of single-lipped

ISSN2454-9940
www.ijasem.org
Vol 12, Issue 3, 2018

channel sections. The frames are typical of Thailand's small to medium-cold-formed steel structures. A set of sample connections is modeled in IDEA StatiCa [36], which is component-based finite element modeling software. By simplifying the connection model, the effect of variables on the connection behavior can be more clearly seen. In this paper, five variables that affect the flexibility of the bolt-gusset plate system connections are selected for the parametric study, including bolt size, bolt spacing, number of bolts, thickness of the gusset plate, and thickness of the connected members. The effects of the variables on the behavior of the connection are discussed and compared. With this knowledge, the parameters involved can be modified more efficiently to achieve the desired behavior of the connection as well as the overall structure. The parametric study consists of four parts of work. The methodology is explained in Figure 1.

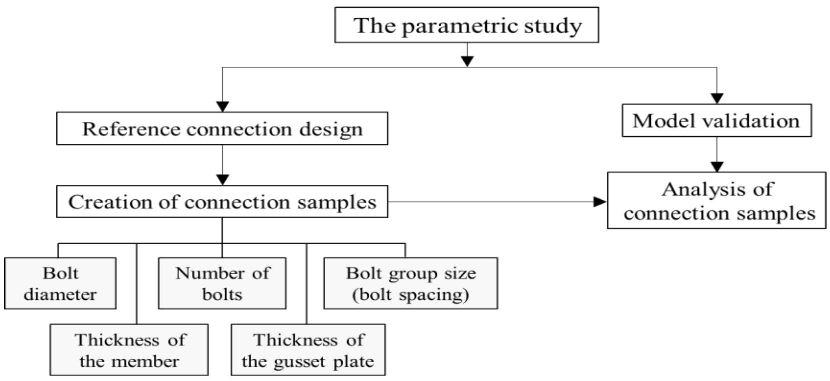


Figure 1. The methodology

2. Samples

2.1. The Portal Frame

The building frame chosen in this study was a portal frame structure that resembled a small to medium-sized factory or warehouse. A series of transverse portal frames consisted of columns and rafters. Each frame member was made of the C30030 cold-formed steel-lipped channel section, for which its geometrical properties were as given in the catalog of LYSAGHT products [37]. The frame members were connected by a bolt-gusset plate system. The cold-formed steel section had an elastic modulus and a shear modulus similar to regular steel, which were equal to 210 GPa and 78 GPa, respectively. The yield strengths of the steel and the bolts were 450 MPa and 640 MPa, respectively. Each plane frame comprised two column base connections, two eaves' connections, and an apex connection. The columns and rafters were made of the same single-lipped channel section. Geometrical information about the portal frame and its member section is defined in Figure 2. The frame members and joints used in this study were designed in accordance with the North American Specification for the Design of Cold-Formed Steel Structural Members AISI S100-2016 [38] to resist the design loads, including gravitational load and wind load, specified in DPT1311-50 Thailand wind code [39].

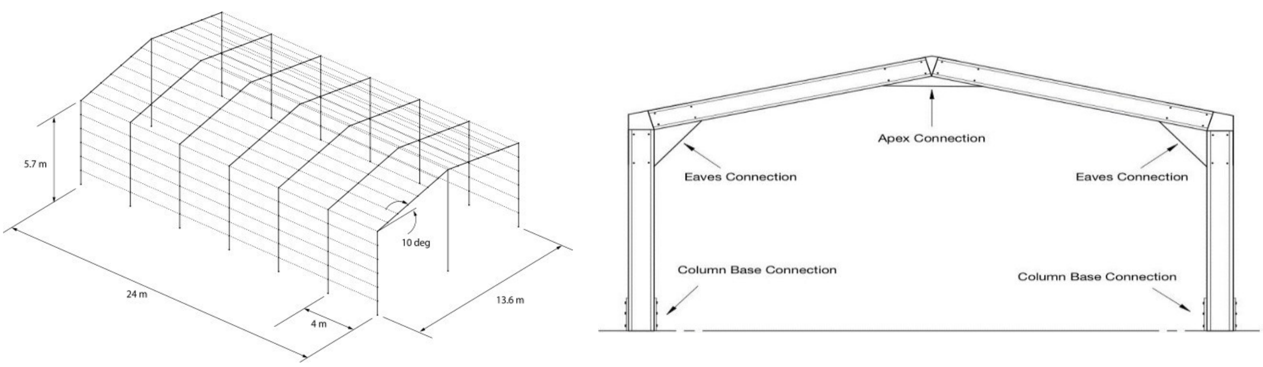


Figure 2. The portal frames

2.2. The Connections

In the portal frame made of single cold-formed steel channel sections, there are mainly two types of connections, namely base connections and member connections. At each column base, the base connection connects the column base to the concrete ground. In this study, the base connection using a U-shaped connector was selected. The U-shaped steel strap was partially placed in the concrete, letting its two legs stick out of the concrete to allow connection with the flanges of the cold-formed steel member via a set of bolts. A row of bolts was aligned vertically, connecting both flanges of the cold-formed steel section to both sides of the strap (see Figure 3-a) [40].

(a) The base connection (b) The member connection

Figure 3. Type of the connections

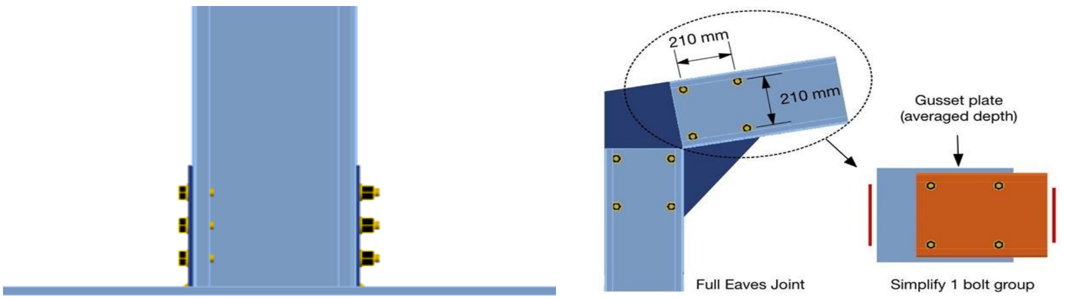
Connections between cold-formed steel members are considered another connection type. Depending on their locations, an eaves connection connects a rafter to a column, and an apex connection connects two rafters together at the top position of the frame. In this study, members were connected via a simple bolt-gusset plate system. The bolts were distributed uniformly in rectangular patterns, as shown in Figure 3-b [41].

3. Numerical Modelling

In this study, the rotational capacity of the cold-formed steel connections was numerically assessed using IDEA StatiCa [36], which is software based on the component-based finite element method (CBFEM) [42]. Based on the original concept of the component method described in Eurocode EN1993-1-8 [16], the CBFEM takes advantage of both the component method [43, 44] and finite element analysis. The cold-formed steel members were modeled using quadrilateral shell elements of four nodes, each composed of three translational degrees of freedom and another three rotational degrees of freedom, adding up to a total of six degrees of freedom. The bolts were modeled as special finite element components. The cold-formed steel was modeled as an elastic-plastic material based on the Von Mises yield criterion. The behavior was assumed to be elastic before reaching the yield plateau, according to EN1993-1-5 [45].

3.1. The Base Connection

The base connection was modeled by assuming the bottom of the U-shaped steel strap to be completely fixed in solid concrete. The two steel plates protruding from the concrete slab were used for connecting the concrete slab to the channel section column, representing the two ends of the U-shaped steel strap protruding from the concrete ground. The concrete ground was assumed to be very rigid; therefore, the steel strap was fixed in place and held firmly. Twisting of the member section was disallowed. The connecting points were on its flanges, where bolts were distributed vertically. The geometrical model of the base connection is shown in Figure 4-a.



(a) The base connection (b) The eaves connection

Figure 4. The connections

4. Validation of the Numerical Model

The component-based finite element model [42], used in IDEA StatiCa Connection, has been well developed and verified by academic scholars so that it is reliable for use in engineering applications. The quality of the analysis results,

INTERNATIONAL JOURNAL OF APPLIED SCIENCE ENGINEERING AND MANAGEMENT

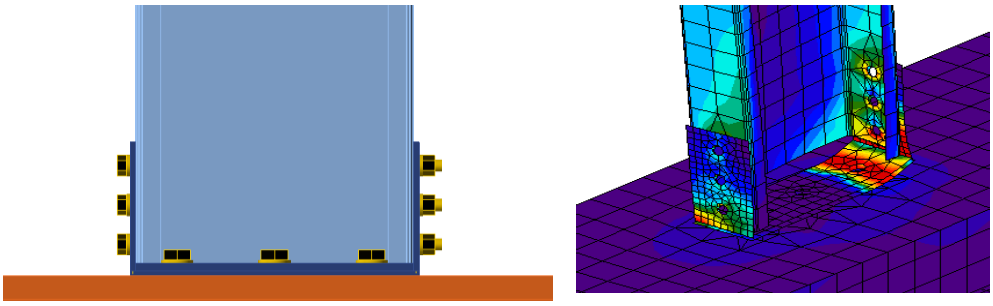
ISSN2454-9940

www.ijasem.org

Vol 12, Issue 3, 2018

however, depends on the understanding and skills of the analyst. In order to prevent any modeling mistakes, the numerical model was validated to ensure well-accepted results of the proposed method of analysis before conducting the parametric study of the cold-formed steel connection. In this work, the laboratory experiments from the research works of Rinchen and Rasmussen (2019) [19] and Ali et al. (2010) [21] were used.

In Rinchen & Rasmussen (2019) [19] study, each column base joint sample was created by connecting a single lipped channel cold-formed steel section to a U-shaped steel strap that was attached to the base plate (cf. Figure 5-a). All the attachments were set up using a bolting system. The samples were subjected to bending moments either on the major axis or the minor axis. To model the sample connection, material properties and geometrical information of the cold-formed steel connection were applied as input in IDEA StatiCa Connection. The cold-formed steel section was modeled using the Von Mises criterion. Using the component-based finite element method, the bolt connection system was represented by a set of springs of different degrees of stiffness. The model geometry and the analysis result are illustrated in Figure 5-b. The outputs as initial rotational stiffness is collected in Table 1, showing that the numerical model could closely predict the experimental results.



(a) Geometry (b) Analysis result

Figure 5. The bolt connection sample for model validation using IDEA StatiCa

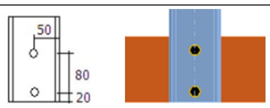
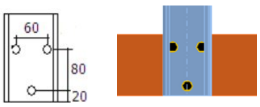
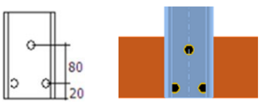
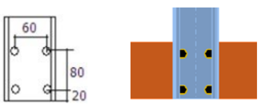
Table 1. Results from the numerical test as compared to the experimental results

Test sample	Bending moment direction	Initial stiffness (kN-m/rad)	
		Experiment*	IDEA StatiCa
BC-1-1	Major axis	1200	1212
BC-1-2	Major axis	1311	1212
BC-2-1	Minor axis	143	94
BC-2-2	Minor axis	92	94

Remarks: *Results from the laboratory experiment by Rinchen and Rasmussen (2019) [19]

The test specimens in [21] were created by attaching a single-lipped channel cold-formed steel member to a plate using different patterns of bolts. The results in Table 2 show that the numerical models could closely predict the experimental results in terms of the initial stiffness and the measured moments.

Table 2. Results from the numerical test as compared to the experimental results

Specimen description	Measured moment resistance @0.05rad (kN·m)		Initial rotational stiffness (kN·m/rad)		Analysis/Experiment	
	Experiment	Analysis	Experiment	Analysis	Moment	Stiffness
CB02 	1.6	1.45	37	32	0.91	0.86
CB03-S1 	2.5	2.50	51	59	1.00	1.16
CB03-S2 	2.5	2.46	56	57	0.98	1.02
CB04 	3.4	3.40	90	92	1.00	1.02

Remarks: * Results from the laboratory experiment by Ali et al. (2010) [21].

INTERNATIONAL JOURNAL OF APPLIED SCIENCE ENGINEERING AND MANAGEMENT

ISSN2454-9940

www.ijasem.org

Vol 12, Issue 3, 2018

5. Setting of the Parametric Study

To study the influence of different variables on flexibility of the cold-formed steel connections, a parametric study was carried out numerically for the base connection and the member connection, which had different characteristics. The objective of the parametric study was to understand how the selected variables affected flexibility and rotational capacity of the connection.

Three values for each variable were selected for each type of connection. For analysis of the base connection, the variables include bolt diameter (D), number of bolts in a group (N), bolt spacing (s), thickness of the U-shaped steel strap (t_p), and thickness of the cold-formed steel section (t_c). The bolt spacing is defined in Figure 6. Table 3 shows a summary of the variables employed in the parametric analysis for the column-base connection. There is a reference sample in each sample group. These reference samples possess the same set of parameters, including CB -D16, CB-N3, CB-S105, CB-TP6, and CB-TC3. They are named differently to ease the comparison of the results for each parameter.

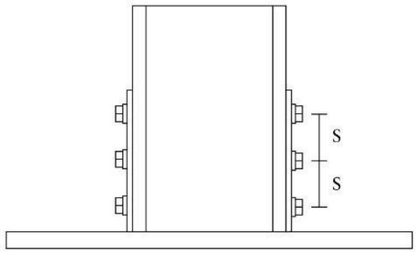


Figure 6. Definition of bolt spacing in the column base connection

Table 3. The selected parameters for the column base connection

Parameter	Sample	Connection details				
		D (mm)	N (mm)	s (mm)	t_p (mm)	t_c (mm)
Bolt diameter	CB-D12	12	3	105	6	3
	CB-D16	16	3	105	6	3
	CB-D24	24	3	105	6	3
Number of bolts	CB-N2	16	2	105	6	3
	CB-N3	16	3	105	6	3
	CB-N4	16	4	105	6	3
Bolt spacing	CB-S65	16	3	65	6	3
	CB-S105	16	3	105	6	3
	CB-S130	16	3	130	6	3
Thickness of U-shape strap	CB-TP3	16	3	105	3	3
	CB-TP6	16	3	105	6	3
	CB-TP9	16	3	105	9	3
Thickness of member section	CB-TC2	16	3	105	6	2
	CB-TC2.4	16	3	105	6	2.4
	CB-TC3	16	3	105	6	3

Similar variables were also used for analysis of the member connection, except the thickness of the U-shaped steel strap which was replaced by the thickness of the gusset plate (t_g), and the bolt spacing which was replaced by bolt group size (S). For each variable, the chosen values must not violate general design standards for cold-formed steel connection design and the chosen size of the parts must be available in practice. The geometrical details of each bolt group of the member connections are illustrated in Figure 7. Table 4 shows a summary of variables employed in the parametric analysis for the member connections. It should be noted that in the parametric study, the reference samples including D16, N4, S210, TG6 and TC3 are the same.

INTERNATIONAL JOURNAL OF APPLIED SCIENCE ENGINEERING AND MANAGEMENT

ISSN2454-9940
www.ijasem.org
Vol 12, Issue 3, 2018

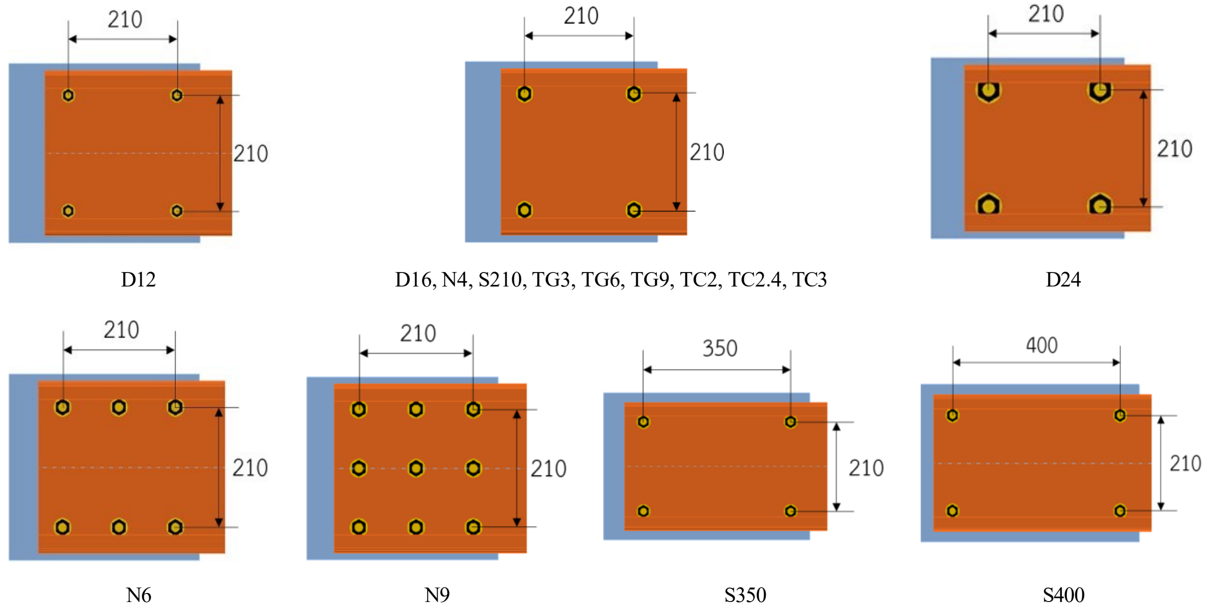


Figure 7. Geometrical details of one bolt group for the numerical samples-[Unit: mm]

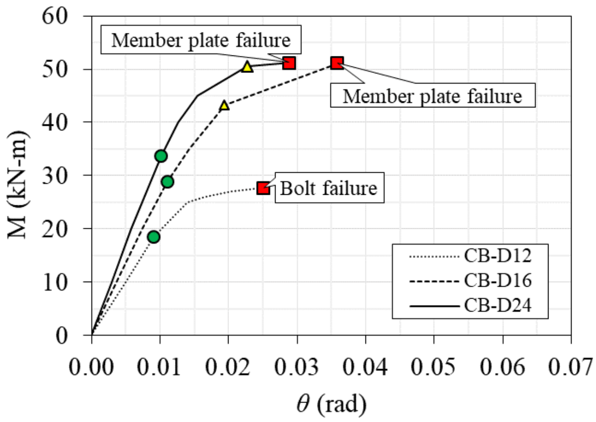
Table 4. The selected parameters for the member connection

Parameter	Sample	Connection details				
		D (mm)	N (mm)	S (mm)	t_g (mm)	t_c (mm)
Bolt diameter	D12	12	4	210	6	3
	D16	16	4	210	6	3
	D24	24	4	210	6	3
Number of bolts	N4	16	4	210	6	3
	N6	16	6	210	6	3
	N9	16	9	210	6	3
Size of bolt group	S210	16	4	210	6	3
	S350	16	4	350	6	3
	S400	16	4	400	6	3
Thickness of gusset plate	TG3	16	4	210	3	3
	TG6	16	4	210	6	3
	TG9	16	4	210	9	3
Thickness of member section	TC2	16	4	210	6	2
	TC2.4	16	4	210	6	2.4
	TC3	16	4	210	6	3

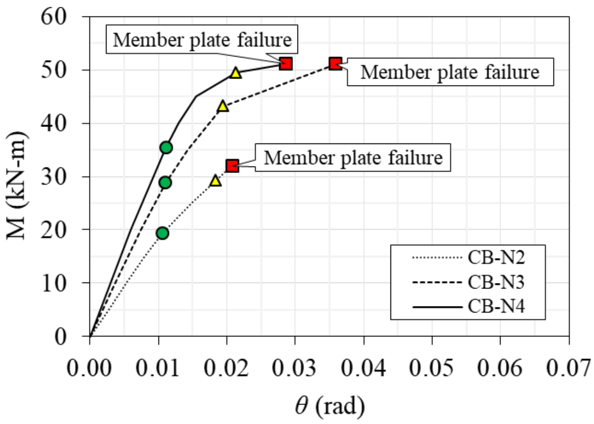
6. Analysis of the Results

The numerical results for the parametric study are shown and discussed in this section for the column-base connection and the member connection. The rotational behaviors of the connections are illustrated via plots of moment- rotation relationships, which are presented in a series of multi-segment lines to simplify the nonlinear curves. Three stages of the curves can be described. The first stage is from the origin to the point on the curve at two-thirds of the limit value of the moment capacity for 5% plastic strain. The initial rotational stiffness of the connection can be determined by the slope of the connecting line. The second stage presents the time when the graph begins to change slope. The lines are extended to the point at which the deformation of any bolt hole, either on the member or on the gusset plate, reaches 5% plastic strain. According to EN1993-1-5 [45], the value of the 5% plastic strain is set as the limit value in the ultimate limit state design criterion. The final stage on the curve starts with another change of the slope to the point where the joint cannot bear any additional moment. The analysis continues until the force reaches the resistance of any bolts or any plates. If the shear and tension existing in a bolt are higher than the shear resistance, tension resistance, or tension- shear interaction resistance of the bolt, the failure is considered a bolt failure. Otherwise, it is considered plate failure, which can be either on the member or the gusset when the plastic strain reaches 15%.

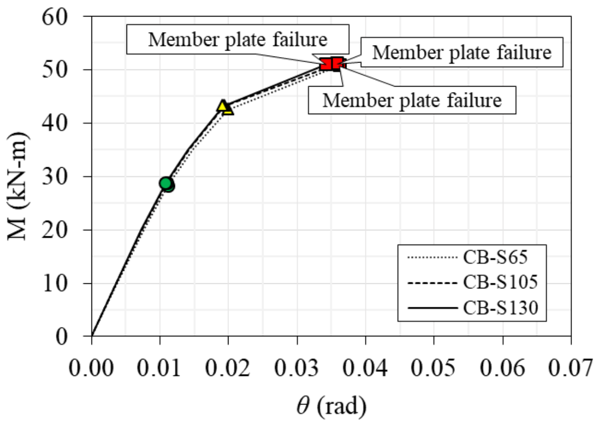
The rotational behaviors of the sample joints obtained from the analyses are presented in Figures 8 and 9 for the column base connection samples and the member connection samples, respectively. The ends of each segment are marked with different symbols. The points that define the initial rotational stiffness are marked with green circles. The yellow triangles are when the plastic strain at any point in the cold-formed steel profile or the connecting plate first reaches 5%. The red squares mark the points where any of the bolts or any of the plates resist the force at their maximum capacities. In some specimens, there may exist the red square but not a yellow triangle. This is because the failure occurs when the plastic strain development is below 5%. All the failure types are labeled.



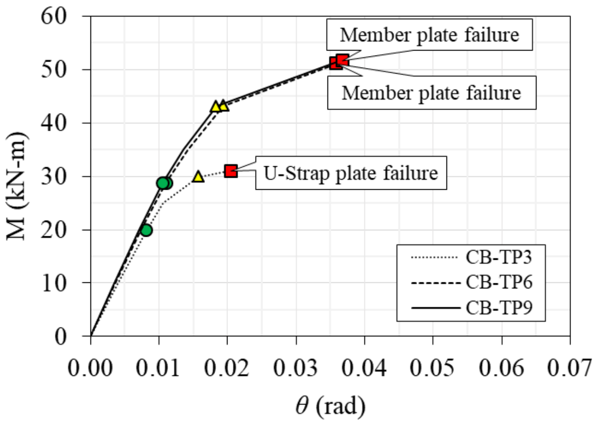
(a) Different bolt sizes



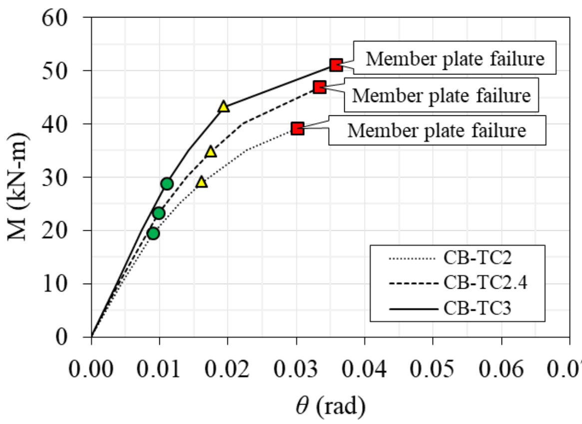
(b) Different numbers of bolts



(c) Different bolt spacings



(d) Different U-shaped plate thicknesses

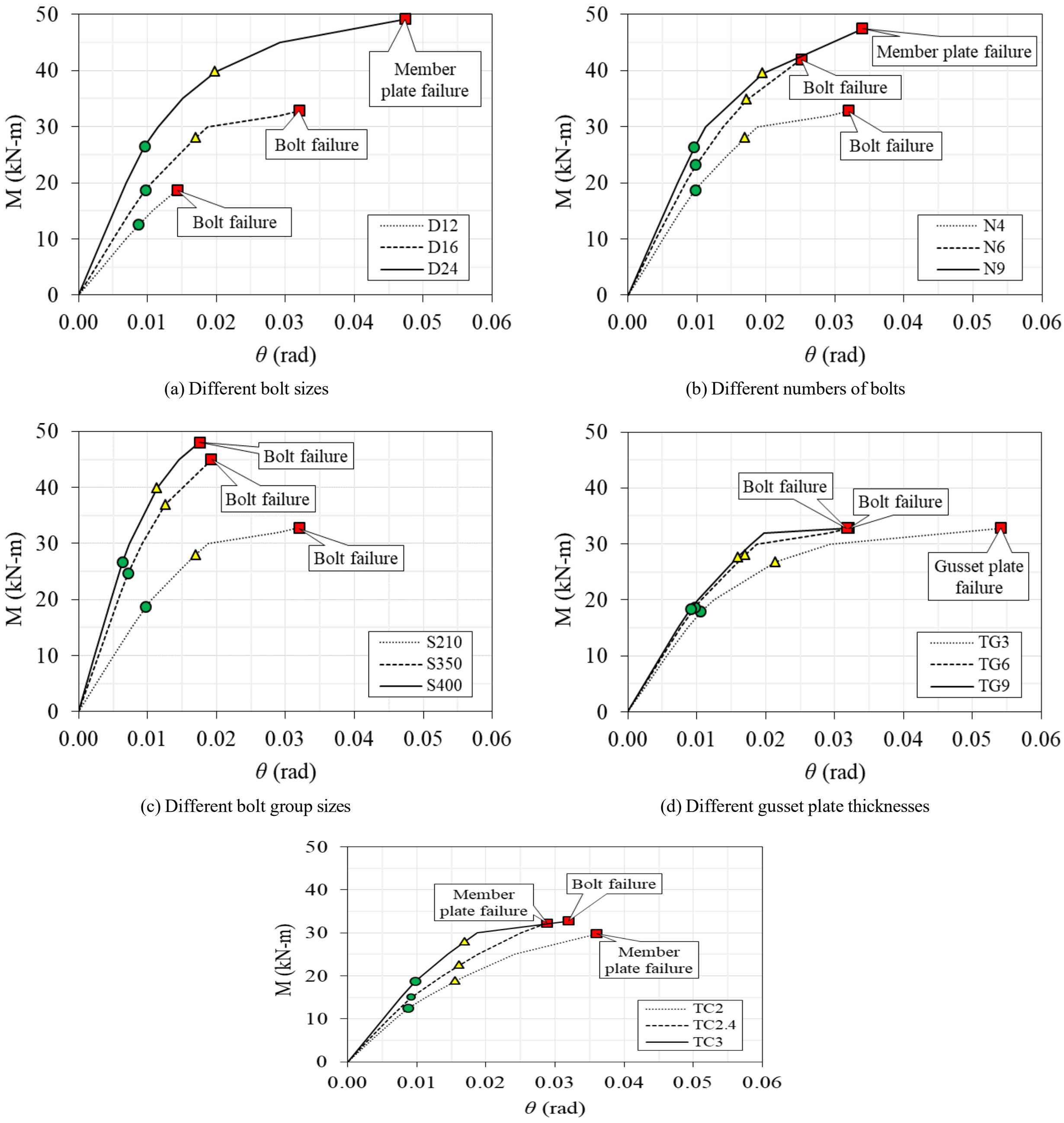


(e) Different cold-formed steel thicknesses

Figure 8. Moment-rotation relationships for the column base connection using different variables

INTERNATIONAL JOURNAL OF APPLIED SCIENCE ENGINEERING AND MANAGEMENT

ISSN2454-9940
www.ijasem.org
Vol 12, Issue 3, 2018



(a) Different bolt sizes

(b) Different numbers of bolts

(c) Different bolt group sizes

(d) Different gusset plate thicknesses

(e) Different cold-formed steel thicknesses

Figure 9. Moment-rotation relationships for the member connections using different variables

The analysis results are presented in the following subsections.

6.1. Effect on Stiffness, Strength and Failure

6.1.1. The Base Connection

Figure 10 shows how the five factors affected the rotational stiffness of the base connection at different levels based on the parametric research. The number of bolts was discovered to be the variable that had the greatest influence on rotational stiffness. Doubling the number of bolts could increase the rotational stiffness to 1.8 times and could increase moment capacity. The second most influential variable was the bolt diameter. Doubling the bolt size could increase the rotational stiffness of the connection by 1.5 times. Larger bolts in the connection could withstand higher moments, leading to the emergence of bolt hole elongation. As a result, the plate failed. In comparison, the variables that were less influential on changes in the rotational stiffness were the thickness of the cold-formed steel section, the thickness of the U-shaped strap, and the bolt spacing, in this order. Doubling the thickness of the cold-formed steel section and the straps

ISSN2454-9940
www.ijasem.org
Vol 12, Issue 3, 2018

increased the rotational stiffness to 1.2 and 1.1 times, respectively. The change in bolt spacing in the vertical direction, however, did not show much improvement.

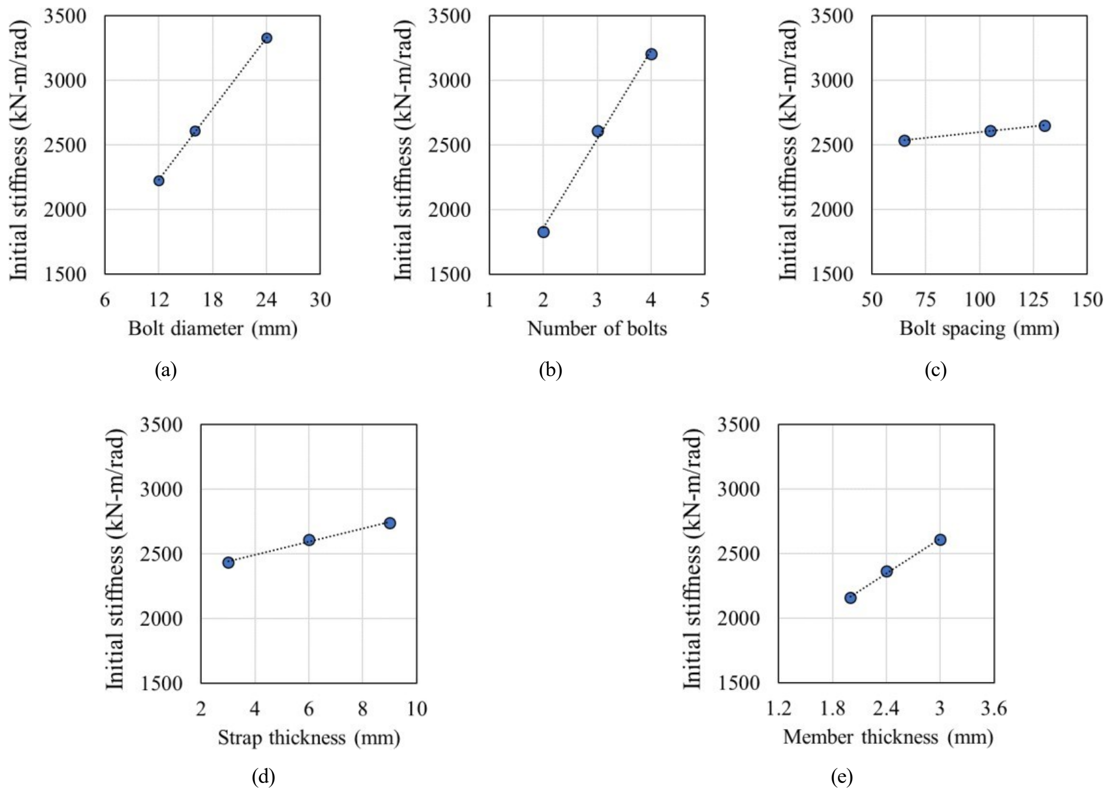


Figure 10. The initial rotational stiffness of the base connection when varying different parameters

In terms of the moment capacity, adjusting any of the five parameters could upgrade the connection to resist the maximum moment of 51 kN-m (cf. Figure 8). All of the sample connections that could attain this maximum moment were found to fail due to bolt hole deformation on a cold-formed steel section flange. Using fewer bolts, smaller bolt sizes, or a thinner U-shaped strap may result in a substantially reduced moment capacity and different failure mechanisms. It should be noted that bearing between a bolt and the corresponding holes depended on the thickness of the connected member and the thickness of the U-shaped strap. For a much greater strap thickness, in comparison to thickness of the member section, the bending moment resistance and the angle of rotation were controlled by bolt hole elongation that appeared on the member section. Again, the parameter that had the least impact on moment capacity of the connection was the bolt spacing. By applying 1.5 times of the original bolt spacing, the rotational behavior still did not show obvious difference from the original case. As the transmission of the bending moment through the bolt of such a connection was in the form of vertical shear (cf. Figure 11), increasing the vertical distance had a relatively small effect on the rotation stiffness and the moment capacity of the connection.

It should be noted that all of the samples could be denoted as the semi-rigid connections (partially restrained connections), in accordance with AISC 360-16 [15] and EN1993-1-8 [16].

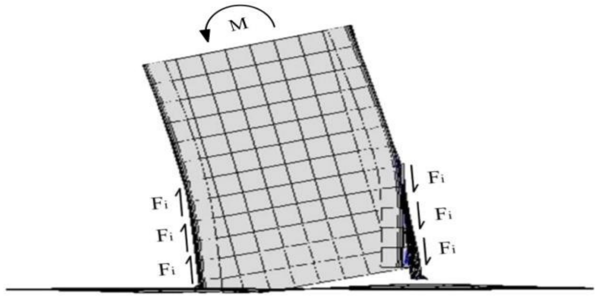


Figure 11. Force transmission in the base connection

6.1.2. The Member Connection

In this study, the member connection represents either the eave connection or the apex connection in the cold-formed steel portal frame. A bolt group was applied to attach the cold-formed steel member to the gusset plate on each side of the connection. The results of the parametric study on moment resistance and rotational behavior of the member connection are shown in Figures 9 and 12, which present the moment-rotation relationships of the member connection and the initial rotational stiffness, respectively. All the samples were classified as the semi-rigid connections (partially restrained connections), in accordance with AISC 360-16 [15] and EN1993-1-8 [16].

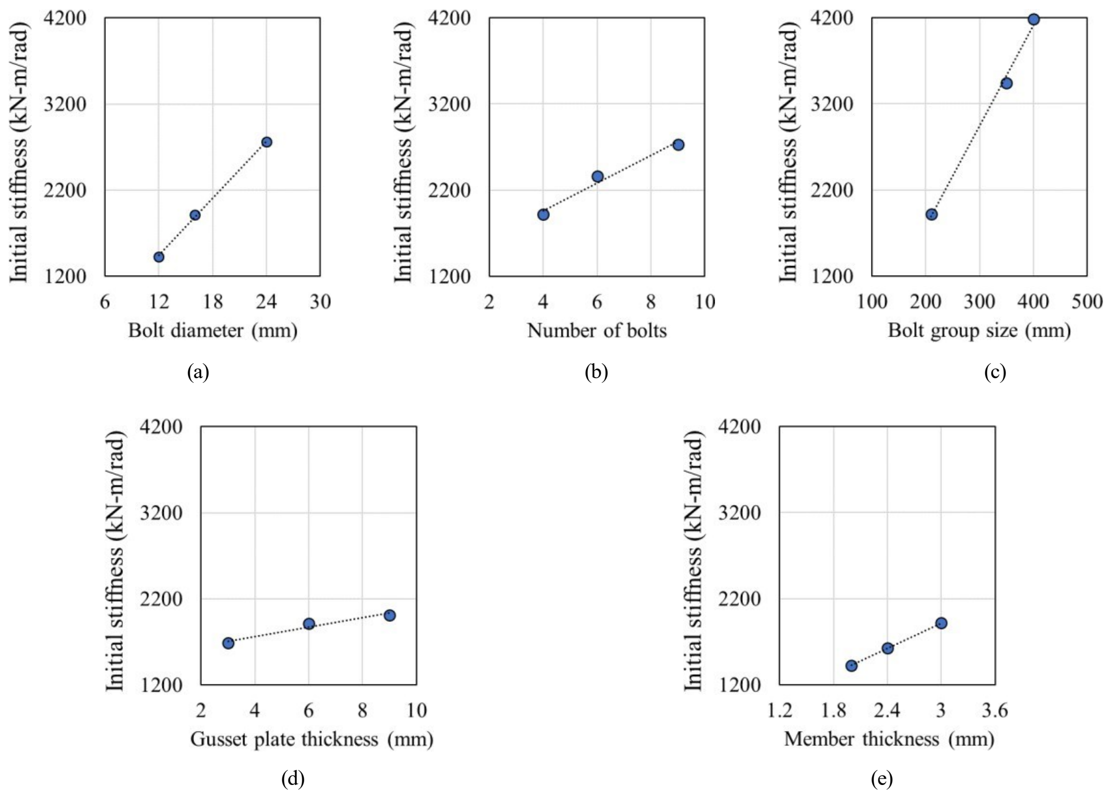


Figure 12. The initial rotational stiffnesses of the member connection when varying different parameters

Among the five variables, the variable that most affected the rotational stiffness of the connection was the bolt group size. Doubling the bolt group size could increase the initial rotation stiffness to 2.2 times. As the distance from the center of rotation to each bolt was increased, the ability to withstand bending moment was higher. The second most influential variable was the bolt size. Doubling the bolt diameter could increase the initial rotation stiffness of the joint to 1.9 times. The third influential variable was the number of bolts. Adding one bolt in each row (from 2 to 3) while still keeping the same bolt group size could improve the stiffness to 1.4 times. With greater number of bolts, the force distribution to each bolt could be less at the same moment level. The failure mode could change from the bolt failure to the plate failure. However, according to the numerical result, change of the number of bolts without increasing the bolt group size might not be efficient for increasing the initial stiffness.

Again, the two least influential variables to the change of the initial stiffness were the thickness of the member section and the thickness of the gusset plate. Both variables directly affected the ability to resist deformation of the bolt holes. The bigger difference between the gusset plate thickness and the member section thickness could lead to failure on a weaker element. Using thicker sections or plates could increase ability to resist moment, with higher stiffness and ductility up to a level. Upon increasing the thicknesses, it was likely that the bolt shear failure might occur.

In terms of moment capacity, it is illustrated in Figure 9 that the maximum moment that could occur in the connection was almost 50 kN-m. The increase of the moment capacity could be achieved by increasing the bolt size, the number of bolts and the bolt group size. It was not likely that increasing the thickness of the member section or the thickness of the gusset plate could upgrade the moment capacity of the connection beyond 35 kN-m, and therefore, these two variables were the least influential to the moment capacity. The findings support the results from the earlier research [30] that increase of the thickness has a limited effect on the strength and the stiffness of the bolted connection.

INTERNATIONAL JOURNAL OF APPLIED SCIENCE ENGINEERING AND MANAGEMENT

ISSN2454-9940

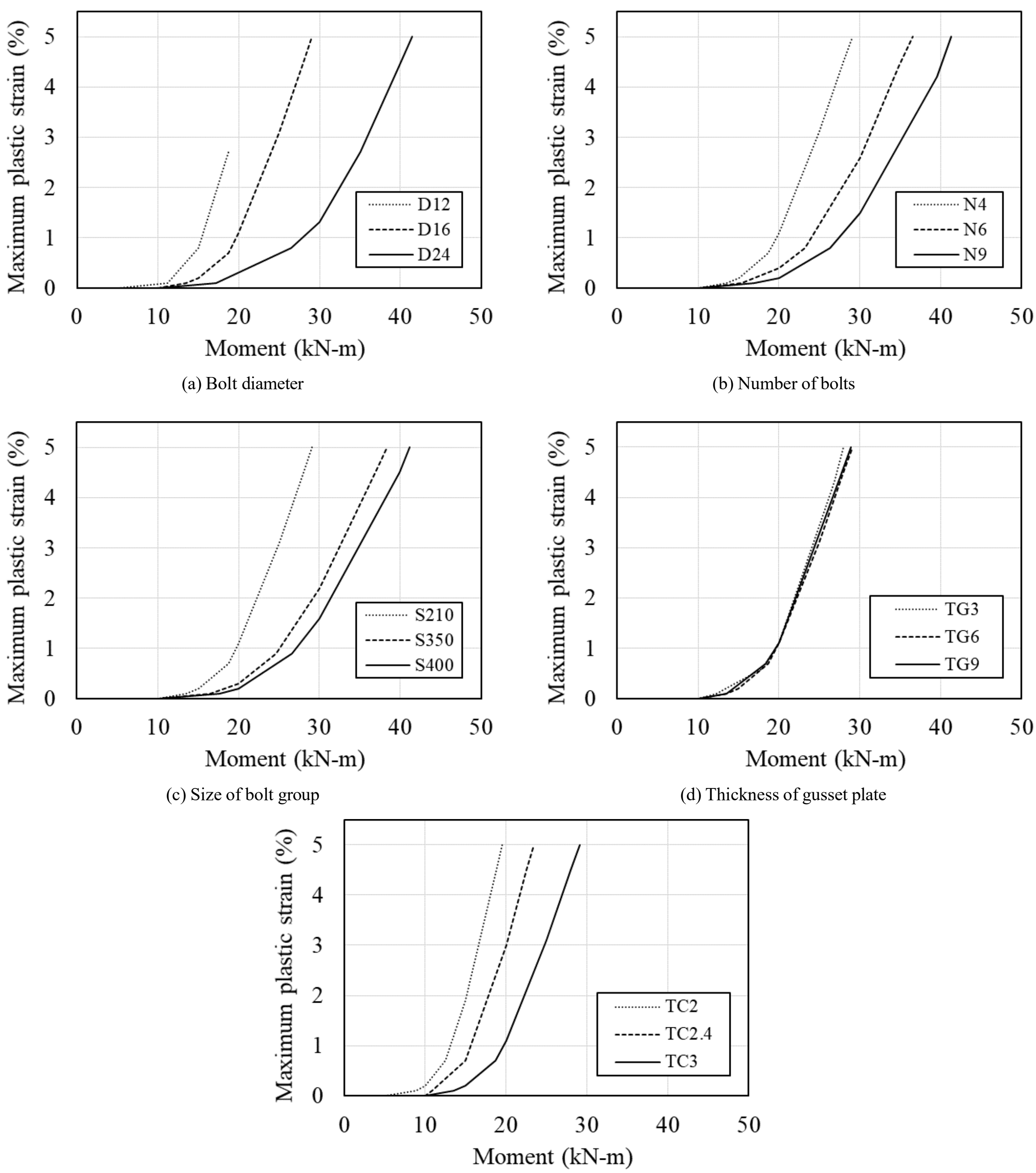
www.ijasem.org

Vol 12, Issue 3, 2018

6.2. Effect on Plastic Strain Development on the Steel Section

From the numerical results, it was obvious that the rotational behavior of the connection was nonlinear. The moment-rotation relationship was linear up to the point where yielding in the member section occurred. After the yielding, the plastic strain started to evolve on the steel section. The effects of different variables on the behavior of the bolted connection were explored by observing how the plastic strain was developed in the cold-formed steel section.

In Figure 13, the development of plastic strain up to the limit stage (at 5%) was observed. In Figure 14, the amount of moment during the plastic strain development from the onset stage (0.1%) to the limit stage (at 5%) was plotted in the parametric study. It was found that increasing the bolt diameter, the bolt group size, the number of bolts, and the thickness of the steel section could obviously delay the development of plastic strain. Increasing the thickness of the gusset plate, however, did not affect the level of plastic strain development. This might be because the thickness of the gusset plate was much more than the thickness of the member section.



(a) Bolt diameter

(b) Number of bolts

(c) Size of bolt group

(d) Thickness of gusset plate

(e) Thickness of cold-form steel section

Figure 13. Plastic strain development in the steel section with varying parameters

INTERNATIONAL JOURNAL OF APPLIED SCIENCE ENGINEERING AND MANAGEMENT

ISSN2454-9940

www.ijasem.org

Vol 12, Issue 3, 2018

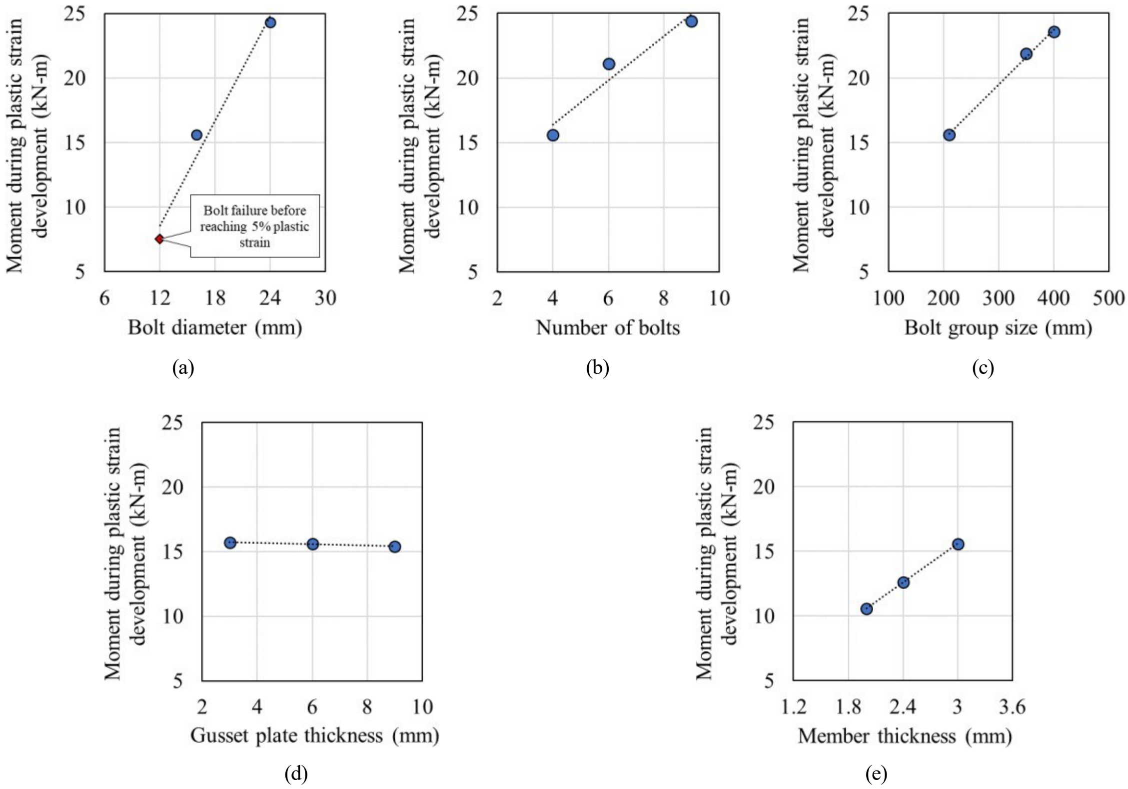


Figure 14. The amount of moment during plastic strain development from 0.1% to 5%.

6.3. Effect on Force Distribution in the Bolt Group

The development of the plastic strain on the steel section is related to the force distribution in the bolt group. A higher shear force in the bolt causes a higher bearing force in the member section. As a result, the plastic strain near the edge of the bolt hole arises faster than at other sites, resulting in connection failure. The plate fails at the point of highest plastic strain development, resulting in bolt hole elongation on either the member section or the gusset plate. Failure of the connection can occur at different locations, depending on how the force is distributed. The bolt that is subjected to the largest shear force is likely to fail first by breakage of bolts under shear or by bolt hole elongation either in the connecting member or the gusset plate. In this subsection, a parametric study to determine force distribution in the bolt group is carried out for the member connection samples. Each bolt in the three bolt arrangements is named, as shown in Figure 15.

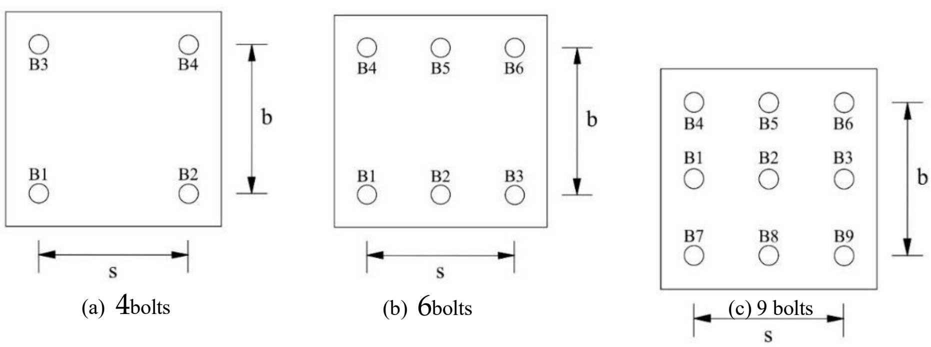


Figure 15. The bolt patterns

The distributions of shear forces in the bolts are collected in Tables 5 to 9 for the connection samples in each variable group. For comparison, the shear forces were measured when the plastic strain at any point in the steel section reached 5%. According to the numerical results, the transfer of bending moment through the connection to the bolts took the form of shear forces, which were not always evenly distributed in each bolt. With the same bolt pattern and group size, larger bolts have higher shear resistance when compared to the smaller bolts. Therefore, it could be seen from Table 5 that the connections consisting of larger bolts could resist higher moments and were controlled by bolt hole deformation rather than bolt failure. Adding more bolts in the bolt group of the same size could increase the moment capacity. It can

ISSN2454-9940
www.ijasem.org
Vol 12, Issue 3, 2018

be seen from Table 6 that shear forces in the bolts were not equal in the bolt group for the 6-bolt system and the 9-bolt system; more forces appeared in the bolts on the outer perimeter of the bolt group. Despite the uneven distribution, the forces appeared to be almost proportional to the distances from each bolt to the center of the bolt group. This finding confirmed the validity of the proposed equations from the previous study [10]. By comparing Tables 5 and 6, using a larger size of bolts appeared to be more efficient in increasing the moment resistance than adding the number of bolts, considering the same bolt group size. It depended, however, on the bolt arrangement as well. The layout that evenly distributes the bolts from the center of the bolt group would be the most efficient.

Table 5. Shear force distribution in the bolt group with different bolt diameters

Sample	Bolt diameter *D* (mm)	Moment at 5% plastic strain (kN-m)	Shear in bolt (kN)			
			B1	B2	B3	B4
D12	12	*18.7	31.5	31.5	31.5	31.5
D16	16	29.1	48.9	49.7	48.9	48.7
D24	24	41.5	69.0	73.7	69.8	67.5

*Remark :Failure of the connection is due to shear failure of bolts.

Table 6. Shear force distribution in the bolt group with different number of bolts

Sample	Number of bolts *N*	Moment at 5% plastic strain (kN-m)	Shear in bolt (kN)								
			B1	B2	B3	B4	B5	B6	B7	B8	B9
N4	4	29.1	48.9	49.7	48.9	48.7	-	-	-	-	-
N6	6	36.5	48.0	38.0	49.2	48.1	37.7	47.3	-	-	-
N9	9	41.3	33.6	0.5	33.7	45.3	36.1	46.5	45.0	35.4	43.6

Table 7. Shear force distribution in the bolt group with different bolt group sizes

Sample	Bolt group size *S* (mm)	Moment at 5% plastic strain (kN-m)	Shear in bolt (kN)			
			B1	B2	B3	B4
S210	210	29.1	48.9	48.9	49.7	48.9
S350	350	38.3	46.8	46.8	49.0	46.4
S400	400	41.3	45.5	45.5	48.1	45.2

Table 8. Shear force distribution in the bolt group with different gusset plate thicknesses

Sample	Gusset plate thickness t_g (mm)	Moment at 5% plastic strain (kN-m)	Shear in bolt (kN)			
			B1	B2	B3	B4
TG3	3	28.0*	48.4	47.6	46.0	47.0
TG6	6	29.1	48.9	49.7	48.9	48.7
TG9	9	28.9	48.4	49.4	48.4	48.5

Remarks :*The plastic strain in the gusset plate reached 5%

Table 9. Force distribution at bolts for different member thicknesses

Sample	Member thickness t_c (mm)	Moment at 5% plastic strain (kN-m)	Shear in bolt (kN)			
			B1	B2	B3	B4
TC2	2	19.5	32.6	33.4	32.6	32.7
TC2.4	2.4	23.4	39.2	40.1	39.2	39.2
TC3	3	29.1	48.9	49.7	48.9	48.7

From Table 7, increasing bolt group size could greatly increase the joint moment capacity due to the increased moment arms. It was also observed that, for the greater distance between the bolts, the distribution of force to the bolts became more uneven. Tables 8 and 9 show that changes of the gusset plate thickness or the member thickness did not much affect the shear force distribution in the connection. From all of the samples, it was observed that the shear forces were dominant in the corner bolts rather than in the bolts around the center of the bolt group. The distribution of the equivalent plastic strain and the Von Mises stress in the specimens with three different bolt sizes (i.e., D12, D16, and D24 specimens) and with three numbers of bolts (i.e., N4, N6, and N9 specimens) is shown in Figures 16 and 17, respectively. With larger bolts or more bolts, the Von Mises stress distribution became more evenly distributed on the steel profile. It was also observed that larger plastic deformations appeared at the bolt holes closer to the member profile, where the moment was applied, than at the other end.

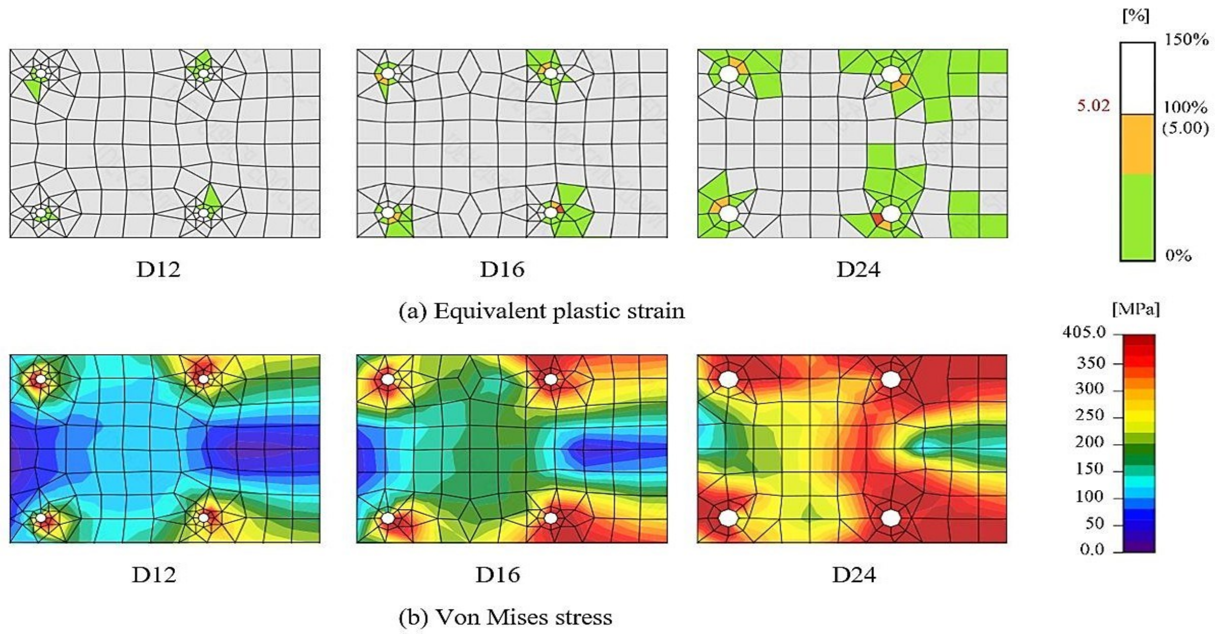


Figure 16. Distribution of equivalent plastic strain and Von Mises stress for D12 when the bolts failed and for D16 and D24 specimens when the plastic strain reached 5%

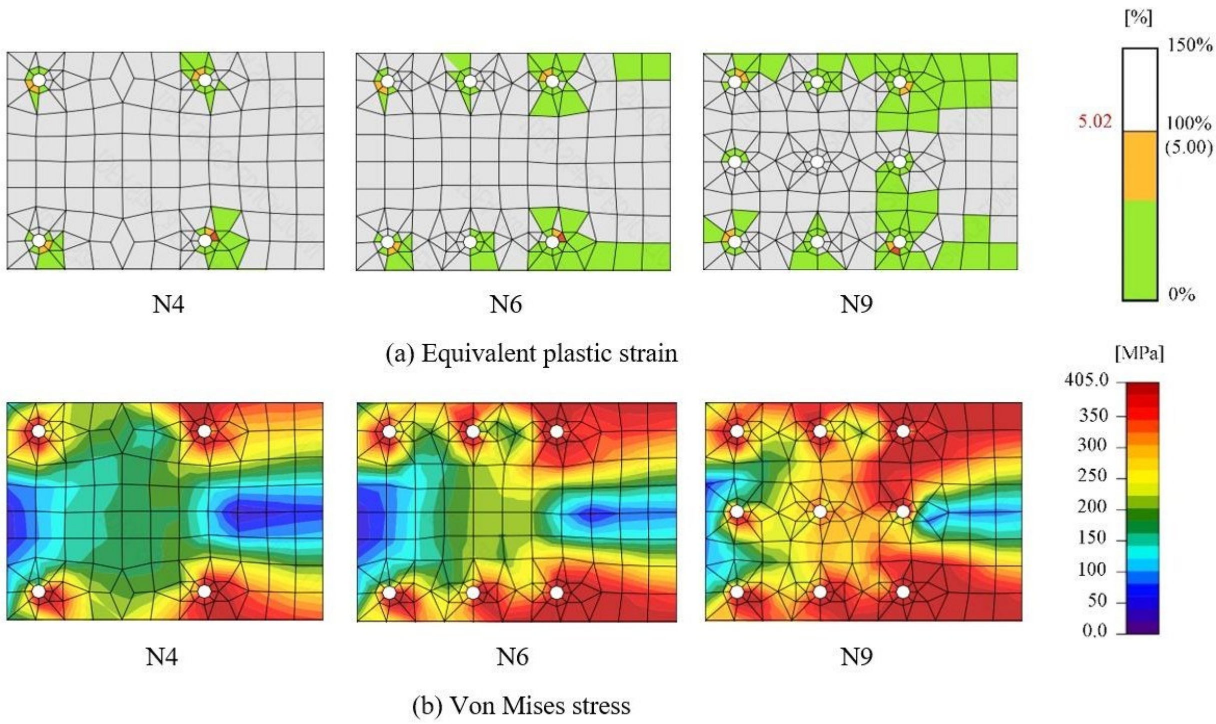


Figure 17. Distribution of equivalent plastic strain and Von Mises stress for N4, N6 and N9 specimens when the plastic strain reached 5%

7. Conclusions

The purpose of this research is to study variables affecting the rotational stiffness and behavior of the cold-formed steel connection assembled from single-lipped channel cold-formed steel sections by means of a bolt-gusset plate system. The variables involved in this study include the bolt size, the number of bolts, the bolt spacing, the member thickness, and the gusset plate thickness. The results of the parametric study can be summarized as follows:

- The given cold-formed steel connection using the bolt-gusset plate system exhibited semi-rigid rotational behavior, which was expressed through the nonlinear relationship between the moment and the angle of rotation of the connection. It was found that the rotational behavior of the connection depended on the details of the assembly.
- There were two failure types of the cold-formed steel connection in this study. The first type was failure due to excessive bolt hole elongation, leading to plate failure in either the cold-formed steel member or the connecting plate (the U-shaped strap/the gusset plate). The second pattern was failure due to the breakage of bolts under shear stress.
- For the given base connection, it was found that the variables that affected the initial rotational stiffness of the connection the most were the number of bolts and the bolt diameter. The stiffness may also be affected by the thickness of the member and the thickness of the U-shaped strap. It was, however, observed that adjusting the bolt spacing for this type of connection did not cause any noticeable change in the rotational behavior. Higher moment capacities were observed when the plate deformation reached its limit stage rather than when the bolts reached their capacity.
- For the given member connection, the most influential variables in the rotational behavior were the bolt group size (also implying the bolt spacing), the bolt diameter, and the number of bolts. The bolt group size was found to be a very influential parameter in the initial stiffness of the member connection. The thickness of the member section and the thickness of the gusset plate, on the other hand, might have only a little impact on stiffness and moment capacity.
- The distribution of shear forces in the bolt group was related to the distances from each bolt to the center of the bolt group. The forces were initially distributed according to the distance from the center of the bolt group. However, the force distribution in the bolt group appeared to vary during the plastic strain development. Larger plastic deformations appeared closer to the applied load at the bolt holes on the steel profile.

As the cold-formed steel structures are made of lightweight and thin sections, they may have a rather different load-bearing behavior from structures made of other materials. Because the results of this study are limited to the chosen bolt-gusset plate connection system based on a certain set of parameters, the research can be further expanded. The connection details may be more specific and suitable for applications in different structures, such as seismic-resistant structures [46–47]. Further investigation may be on other techniques of connecting members, such as the use of self-drilling screws instead of bolts, which can be more economical for structural assembly [48]. With a better understanding of the parameters involving bolt connection behavior, an efficient cold-formed steel structure design should be obtained.

8. Declarations

8.1. Data Availability Statement

The data presented in this study are available on request from the corresponding author.

8.2. Funding

The research was financed by National Research Council of Thailand, Grant number NCRT201/2563.

8.3. Conflicts of Interest

The authors declare no conflict of interest.

9. References

[1] Johnston, R. P. D., McGrath, T., Nanukuttan, S., Lim, J. B. P., Soutsos, M., Chiang, M. C., Masood, R., & Rahman, M. A. (2018). Sustainability of Cold-formed Steel Portal Frames in Developing Countries in the Context of Life Cycle Assessment and Life Cycle Costs. Structures, 13, 79–87. doi:10.1016/j.istruc.2017.11.003.

[2] Rondal, J. (2005). Introduction to Light Gauge Metal Structures. Light Gauge Metal Structures Recent Advances. CISM International Centre for Mechanical Sciences, 455. Springer, Vienna, Austria. doi:10.1007/3-211-38023-X_1.

[3] Lee, Y. H., Tan, C. S., Mohammad, S., Md Tahir, M., & Shek, P. N. (2014). Review on cold-formed steel connections. The Scientific World Journal, 2014, 1–11. doi:10.1155/2014/951216.

[4] Chung, K. F., & Lau, L. (1999). Experimental investigation on bolted moment connections among cold formed steel members. Engineering Structures, 21(10), 898–911. doi:10.1016/S0141-0296(98)00043-1.

[5] Lim, J. B. P., & Nethercot, D. A. (2003). Ultimate strength of bolted moment-connections between cold-formed steel members. Thin-Walled Structures, 41(11), 1019–1039. doi:10.1016/S0263-8231(03)00045-4.

[6] Yu, W. K., Chung, K. F., & Wong, M. F. (2005). Analysis of bolted moment connections in cold-formed steel beam-column sub-frames. Journal of Constructional Steel Research, 61(9), 1332–1352. doi:10.1016/j.jcsr.2005.03.001.

[7] Hadianfard, M. A., & Razani, R. (2003). Effects of semi-rigid behavior of connections in the reliability of steel frames. Structural Safety, 25(2), 123–138. doi:10.1016/S0167-4730(02)00046-2.

[8] Dundu, M., & Kemp, A. R. (2006). Strength requirements of single cold-formed channels connected back-to-back. Journal of Constructional Steel Research, 62(3), 250–261. doi:10.1016/j.jcsr.2005.07.006.

[9] Johnston, R. P., Wrzesien, A. M., Lim, J. B. P., Sonebi, M., & Armstrong, C. G. (2013). The effect of semi-rigid joints on the design of cold-formed steel portal frame structures. Civil and Environmental Research, 5, 1-5.

[10] Lim, J. B. P., & Nethercot, D. A. (2004). Stiffness prediction for bolted moment-connections between cold-formed steel members. Journal of Constructional Steel Research, 60(1), 85–107. doi:10.1016/S0143-974X(03)00105-6.

[11] Dubina, D., Stratan, A., & Nagy, Z. (2009). Full - Scale tests on cold-formed steel pitched-roof portal frames with bolted joints. Advanced Steel Construction, 5(2), 175–194.

[12] Wrzesien, A. M., Lim, J. B. P., Xu, Y., MacLeod, I. A., & Lawson, R. M. (2015). Effect of stressed skin action on the behaviour of cold-formed steel portal frames. Engineering Structures, 105, 123–136. doi:10.1016/j.engstruct.2015.09.026.

[13] Ali, B. A., Saad, S., & Osman, M. H. (2010). Cold-formed steel frame with bolted moment connections. International Journal of Civil & Structural Engineering, 1(3), 534-544.

[14] Ho, H. C., & Chung, K. F. (2006). Structural behavior of lapped cold-formed steel Z sections with generic bolted configurations. Thin-Walled Structures, 44(4), 466–480. doi:10.1016/j.tws.2006.03.012.

[15] ANSI/AISC 360-16. (2016). Specification for Structural Steel Buildings. American institute of steel construction, Chicago, United States.

[16] EN 1993-1-8 (2005). Eurocode 3: Design of steel structures-Part 1-8: Design of joints. European Committee for Standardization, Brussels, Belgium.

[17] Wong, M. F., & Chung, K. F. (2002). Structural behaviour of bolted moment connections in cold-formed steel beam-column sub-frames. Journal of Constructional Steel Research, 58(2), 253–274. doi:10.1016/S0143-974X(01)00044-X.

[18] Maali, M., Sagiroglu, M., & Semih Solak, M. (2018). Experimental behavior of screwed beam-to-column connections in cold-formed steel frames. Arabian Journal of Geosciences, 11(9), 205. doi:10.1007/s12517-018-3540-4.

[19] Rinchen, & Rasmussen, K. J. R. (2019). Behaviour and modelling of connections in cold-formed steel single C-section portal frames. Thin-Walled Structures, 143, 106233. doi:10.1016/j.tws.2019.106233.

[20] Hazlan, A., Tahir, M., Sulaiman, A., & Mahendran, M. (2010). Bolted beam-column moment connections between cold-formed steel members. Incorporating Sustainable Practice in Mechanics and Structures of Materials, 655–660, Taylor & Francis Group, Milton Park Abingdon, United Kingdom. doi:10.1201/b10571-119.

[21] Zhang, W., Zhao, Z., & Liu, Y. (2023). Experimental investigation of header end-plate beam-to-column composite connections with single-corner gusset plates. Journal of Constructional Steel Research, 201, 107722. doi:10.1016/j.jcsr.2022.107722.

[22] Daz, C., Mart, P., Victoria, M., & Querin, O. M. (2011). Review on the modelling of joint behaviour in steel frames. Journal of Constructional Steel Research, 67(5), 741–758. doi:10.1016/j.jcsr.2010.12.014.

[23] Khalate, S., & Kulkarni, S. (2015). Finite element analysis of cold formed steel bolted connection. International Journal of Recent Technology and Engineering, 4(3), 23-28.

[24] El-Hadary, M. R., El-Aghoury, I. M., & Ibrahim, S. A. B. (2022). Behavior of different bolted connection configurations in frames composed of cold-formed sections. Ain Shams Engineering Journal, 13(1), 101500. doi:10.1016/j.asej.2021.05.014.

[25] Shahini, M. F., Bagheri Sabbagh, A., Davidson, P., Mirghaderi, R., & Torabian, S. (2022). Experiments on cold-formed steel moment-resisting connections with bolting friction-slip mechanism. Journal of Constructional Steel Research, 196, 107368. doi:10.1016/j.jcsr.2022.107368.

[26] Ye, J., Mojtabaei, S. M., Hajirasouliha, I., & Pilakoutas, K. (2020). Efficient design of cold-formed steel bolted-moment connections for earthquake resistant frames. Thin-Walled Structures, 150. doi:10.1016/j.tws.2018.12.015.

[27] Nagy, Zs., Dezo, A., & Muresan, A. A. (2019). Parametric study of cold formed steel joints using the component method. Advances in Engineering Materials, Structures and Systems: Innovations, Mechanics and Applications, 1089–1094, CRC Press, Boca Raton, United States. doi:10.1201/9780429426506-189.

[28] Steenhuis, M., Jaspart, J. P., Gomes, F., & Leino, T. (1998). Application of the component method to steel joints. COST C1 International Conference, 17-19 September, 1998, Liège, Belgium.

[29] Blum, H. B., & Li, Z. (2021). Sensitivity of design parameters on the stability of apex connections in cold-formed steel portal frames. Proceedings of the Annual Stability Conference Structural Stability Research Council 2021, SSRC 2021, 13–16.

[30] Wang, F., Han, Y., & Yang, J. (2021). Nonlinear FE analysis on stiffness and resistance of bolted cold-formed steel built-up joints. Structures, 33, 2520–2533. doi:10.1016/j.istruc.2021.04.098.

[31] Aminuddin, K., Saggaff, A., Tahir, M. M., Ngian, S. P., Sulaiman, A., Firdaus, M., & Aghlara, R. (2019). Analytical and Experimental Investigation on Slip-in Gusset Plate Connection for Double C-channel Sections of Cold-formed Steel. The Open Civil Engineering Journal, 13(1), 210–217. doi:10.2174/1874149501913010210.

[32] Anwer, B., Saad, S., & Osman, H. (2012). Structural Performance of Bolted Moment Connections among Single Cold-Formed Channel Sections. International Journal of Engineering & Technology, 2(4), 599–607.

[33] Freya, R., Senthil, R., Merin, W. J., Saravanakumar, R., Kuber, K., & Gowtham, M. (2016). Behaviour of cold-formed steel semi rigid connections. International Specialty Conference on Cold-Formed Steel Structures, 9-10 November, 2016, Baltimore, United States.

[34] Dubina, D., Stratan, A., Ciutina, A., Fulop, L., & Zsolt, N. (2018). Monotonic and Cyclic Performance of Joints of Cold Formed Steel Portal Frames. Thin-Walled Structures, 381–388, CRC Press, Boca Raton, United States. doi:10.1201/9781351077309-42.

[35] Aly, E. H. A. H., Hanna, M. T., & El-Mahdy, G. M. (2018). Strength and Ductility of Steel Cold-Formed Section Beam to Column Bolted Connections. Facing the Challenges in Structural Engineering. GeoMEast 2017, Sustainable Civil Infrastructures, Springer, Cham, Switzerland. doi:10.1007/978-3-319-61914-9_33.

[36] IDEA StatiCa. (2021). Fast Connection Design. IDEA StatiCa, Brno, Czech Republic. Available online: https://www.ideaStatiCa.com/connection-design (accessed on October 2023).

[37] BlueScope Steel Limited. (2017). LYSAGHT Zeds and CEES user guide for design and installation professionals. BlueScope Steel Limited, Melbourne, Australia.

[38] AISI S100. (2007). North American Specification for the Design of Cold-Formed Steel Structural Members. American Iron and Steel Institute (AISI), Washington, United States.

[39] DPT 1311-50. (2007). Wind Loading Calculation and Response of Buildings. Department of Public Works and Town & Country Planning, Bangkok, Thailand. (In Thai).

[40] Shedblog. (2011). Strap cast in slab footing cleat bracket. Available online: https://shedblog.com.au/building-steel-shed-portal-frames/strap-cast-in-slab-footing-cleat-bracket-2/ (accessed on October 2023).

[41] Kit Buildings Direct. (2020). Steel Framed Buildings Corner. Available online: https://www.kitbuildingsdirect.co.uk/ (accessed on October 2023).

[42] Wald, F., Šabatka, L., Bajer, M., Kožich, M., Vild, M., Golubiatnikov, K., ... & Kuříková, M. (2021). Component-based finite element design of steel connections. Czech Technical University, Prague, Czech.

[43] Bučmys, Ž., & Daniunas, A. (2017). Component Method in the Strength Evaluation of Cold-formed Steel Joints. Procedia Engineering, 172, 143–148. doi:10.1016/j.proeng.2017.02.036.

[44] Bučmys, Ž., Daniūnas, A., Jaspart, J. P., & Demonceau, J. F. (2018). A component method for cold-formed steel beam-to-column bolted gusset plate joints. Thin-Walled Structures, 123, 520–527. doi:10.1016/j.tws.2016.10.022.

[45] EN 1993-1-5. (2006). Eurocode 3: Design of structures-Part 1-5: Plated structural elements. European Committee for Standardization (CEN), Brussels, Belgium.

[46] Papargyriou, I., Mojtabaei, S. M., Hajirasouliha, I., Becque, J., & Pilakoutas, K. (2022). Cold-formed steel beam-to-column bolted connections for seismic applications. Thin-Walled Structures, 172, 108876. doi:10.1016/j.tws.2021.108876.

[47] Yin, L., Niu, Y., Quan, G., Gao, H., & Ye, J. (2023). A numerical investigation of new types of bolted joints for cold-formed steel moment-resisting frame buildings. Journal of Building Engineering, 65, 105738. doi:10.1016/j.jobe.2022.105738.

[48] Maali, M. S., Maali, M., & Yazici, C. (2023). Experimental investigation of screwed beam-column connection using cold-formed steel back-to-back sections with gusset-plate. Structures, 47, 2025–2036. doi:10.1016/j.istruc.2022.12.030.