ENGINEERING STRUCTURES AND TECHNOLOGIES
ISSN 2029-882X print/ISSN 2029-8838 online
2013 5(3): 113–122
doi:10.3846/2029882X.2013.869416

THE BEHAVIOR OF COLD FORMED STEEL STRUCTURE CONNECTIONS

Žilvinas Bučmys, Gintas Šaučiuvėnas

Vilnius Gediminas Technical University, Saulėtekio al. 11, 10223 Vilnius, Lithuania

Received 20 August 2013; accepted 17 October 2013

Abstract. Nowadays, there is a growing tendency in the use of cold formed constructions, which may be explained by good strength to cost ratio. Thus, the goal of this paper is to review the behaviour of cold formed steel connection. The article reviews most up to date publications on testing and description of beam-to-beam, beam-to-column and truss connections. In brief, the properties of cold formed steel connections have been found to be semi rigid, where the main factor affecting stiffness is bearing forces around the bolthole. The article also focuses on the technique to increase stiffness of cold formed connections.

Keywords: cold formed steel sections, the behaviour of connections, thin walled structures, joints, semi rigid behaviour, stiffness of cold formed connections.

Reference to this paper should be made as follows: Bučmys, Ž.; Šaučiuvėnas, G. 2013. The behavior of cold formed steel structure connections, *Engineering Structures and Technologies* 5(3): 113–122. http://dx.doi.org/10.3846/2029882X.2013.869416

Introduction

There is a growing trend of using light gouge sections in buildings. Thin-walled, cold formed steel sections can be used as load bearing structures in industrial buildings as purlins and lightweight steel trusses. Moreover, in multi-storey buildings, cold formed sections are used in lightweight steel-framed wall panels or lightweight portal frames. There is a very wide variety of cold formed sections, such as C-sections, Z-sections, sigma-sections, omega-sections, etc. The thickness ranges from 0.8 to 3.0 mm and the depth – from 50 to 350 mm. Typically, cold formed sections are made of higher yield stress steel (approximately 350 MPa).

There is a wide variety of welded and bolted connections of cold formed steel structures. Bolted connections are frequently used on construction sites because of easier assembly. The most common bolted connection solution has gusset plates for beam-to-column joints and sleeve or over-lapped systems for beam-to-beam connections.

The main advantages of thin walled structures are light weight, easy assembly on sites and possibility of prefabrication, which results in a low cost.

However, there are some disadvantages, the main of which is the lack of stability that causes local buckling and low ductility. This means that frames with thin-walled elements cannot create plastic hinges in cold formed steel beams. Design guides (EC 1993-1-3) suggest methods for strength calculation, but do not propose a method for the calculation of the moment–rotation relationship of connections.

The aim of this article is to review some recent progresses in cold formed joint stiffness calculation.

1. Cold formed steel connections

There is a growing trend of using cold formed purlins in construction. The main advantages of these light gouge constructions is their ability to carry heavy loads, fast and easy erection and relatively low price. Traditionally, the most widely used systems for roof or

Corresponding author:
G. Šaučiuvėnas E-mail: *gintas.sauciuvenas@vgtu.lt*

wall constructions are C-sections and Z-sections. Purlins can be connected using over-lapped, sleeve (Fig. 1) and gusset plate connections (Fig. 2).

Beam-to-column connections are mainly used in cold formed steel portal frame constructions. Beam-to-column can be connected using welds, bolts (Figs 3 and 4) or screws. In literature, there is a wide variety of experimental data of joint behaviour.

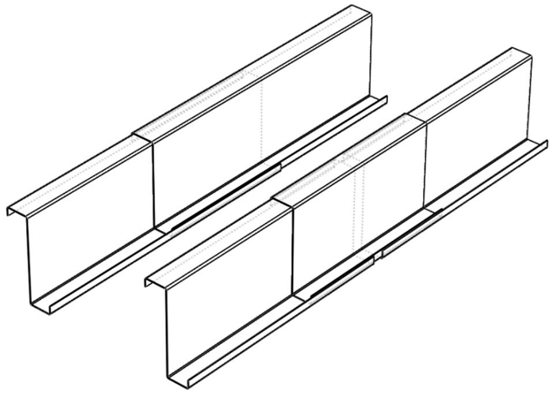

Fig. 1. Sleeve connection on the right; overlapped connection on the left

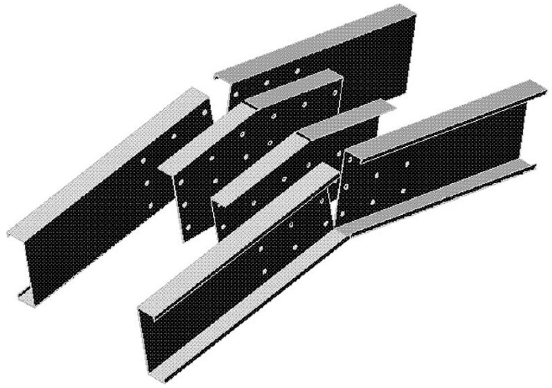

Fig. 2. Details of the arrangement for the apex joint (Lim, Nethercot 2003)

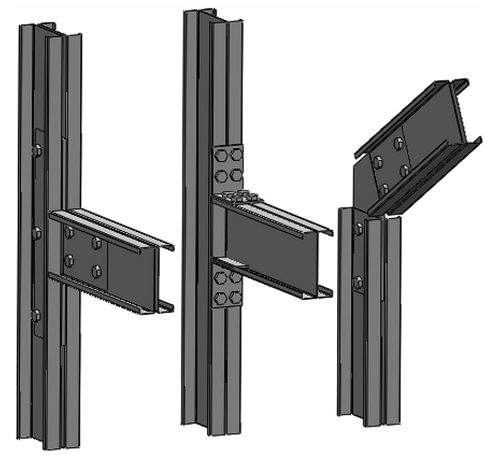

Fig. 3. Beam-to-column bolted connections. From the left: 90 degree connection using gusset plate and angles, 45 degree eave connection with gusset plate

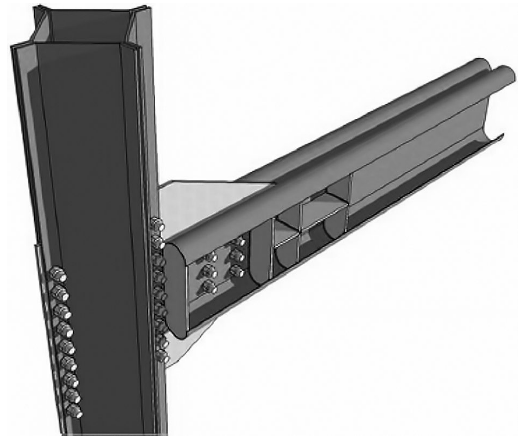

Fig. 4. Beam to column connection with stiffeners (Sabbagh *et al.* 2013)

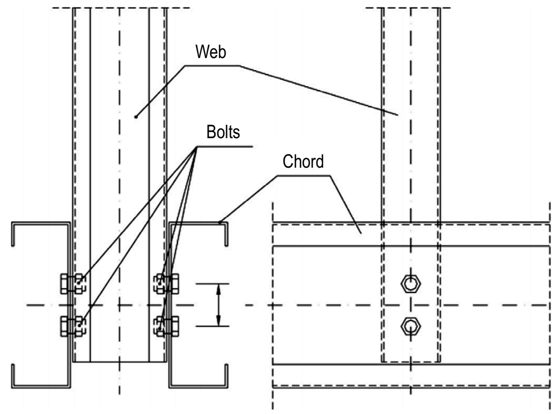

Fig. 5. Cold formed steel truss bolted connection (Zaharia, Dubina 2006)

Cold formed steel trusses are commonly used in roof constructions. Mostly trusses are made from C-sections and connected using gusset plates or directly with bolts (Fig. 5), self-drilling screws, mechanical clinching and welds. Cold formed trusses are lightweight constructions and have satisfying load capacity.

2. Components and failure modes of cold formed steel connections

One of the easiest ways to connect beam-to-column is using gusset plates and angles (Fig. 3). The components of cold formed steel beam to column connections:

- connectors (gusset plate and angles);
- sections of columns and beams;
- bolts.

The failure modes of beam-to-column connections (Fig. 6):

- failure of bolts (1);
- beam and column web (2), gusset plate (3) in bearing;

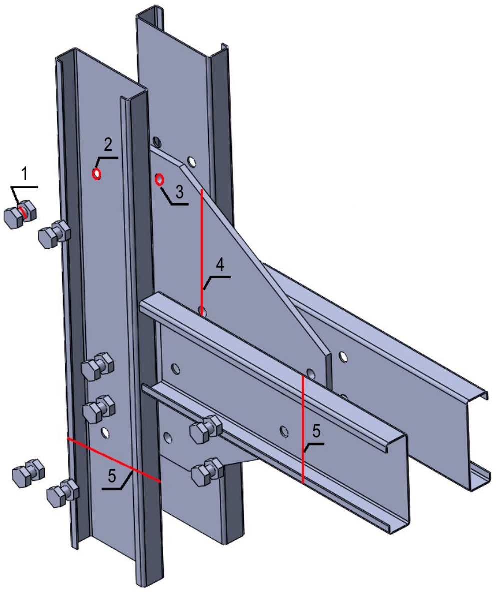

Fig. 6. Exploded view of beam-to-column connection with failure modes marked in red

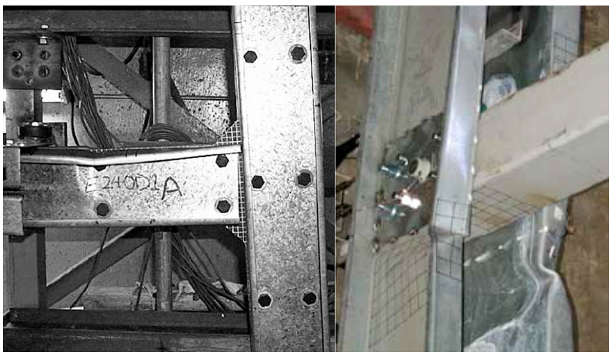

Fig. 7. Local buckling in beam and column sections (Uang *et al.* 2010; Wong, Chung 2002)

- failure of gusset plate (4);
- local buckling in beam and column sections (5) (Fig. 7).

The stiffness of beam-to-column connection depends of:

- the thickness, shape and geometry of the gusset plate and angles;
- the diameter and amount of bolts, the distance between bolts;
- the shape, depth, width and thickness of the column and beam section;
- the thickness of the section and the length of lap and sleeve of overlapped and sleeve connections.

3. The behaviour of cold formed steel connections

3.1. Beam-to-beam connections

Ho and Chung (2004) investigated the structural behaviour of over-lapped cold formed multi-span purlins. In their experiment, 26 specimens of doubled back-to-back Z-section were used. A single span structure of purlins was loaded using one point vertical load. Interconnections provided at regular intervals were used to prevent lateral buckling. Analysis showed that every specimen failed at the end of a lap under combined bending and shear. Moreover, the main influence to the strength and stiffness of construction was caused by both the lap length to the section depth ratio and the lap length to the test span ratio. The moment resistances of lapped connections increased from 80% to 140% of the moment capacities of connected sections then the length of the lap was increased from 1.2 to 6 times section depth. In another paper, after calibration against test data Chung and Ho (2005) proposed analysis and design method for over-lapped cold formed connections to calculate strength of the connection against combined bending and shear. Authors also suggested a method to calculate effective flexural rigidities (the maximum and the minimum values) of lapped connections. In the next article, Chung *et al.* (2005) proposed a numerical model that demonstrated a very good agreement with the data of previous natural experiments. It is worth mentioning, that bolts were simulated as springs and the formula to estimate bearing resistance was presented. Finally, Ho and Chung (2006a) continued their work and in the next paper, they offered the technique for global deformation calculation taking into account deformations due to applied moment or shear force and bearing deformation in the section web around the bolthole under the applied bolt force. Moreover, the authors proposed a method to calculate large deformations of lapped connections after a failure. Zhang and Tong (2008) investigated over-lapped cold formed single span of two-spanned purlins, that was connected with bolts and self-drilling screws at both flanges or at the top flange only (simulating the connection of roof sheet and purlin). The results showed that screws did not have much influence neither on the strength nor on the stiffness characteristics. In addition, they proposed an empiric method for estimation of flexural rigidity of lapped connections. Dubina and Ungureanu (2010) simulated the behaviour of cold formed steel double span purlin system under a uniform load using numerical analysis. Authors suggested the new approach by investigating the influence of the web crippling action at the edge of the lapped zone. They suggested checking web stability according to the methodology given in EN1993-1-3. Furthermore, they proposed analytical formula to calculate initial flexural rigidity from the bearing effect (scientists claim that the main factor affecting stiffness of this type of construction are bearing forces around the bolthole).

Another way of connecting purlins is using sleeve connections. Sleeve connections are easy to install and have good strength and stiffness characteristics. Gutierrez *et al.* (2011) examined this method by way of experiment and numerical model results. A single span sleeve system was loaded using one point load in the middle of the span. The results showed that the numerical model can accurately predict stiffness and strength of sleeve connections. Moreover, this method does not require material resources and is less time consuming. The experiment done by Yang and Liu (2012) included 20 specimens. All of them were single span purlins connected at the middle of the span and loaded to the connection. The failure mode of the lateral buckling was prevented using interconnections. The authors divided the behaviour of stiffness into 3 phases: firstly, deformation was linear because of friction forces between elements; secondly, the stiffness decreased, because friction forces overcame and slipping occurred (it mostly depends on the hole and bolt ratio); thirdly, stiffness increased since all bolts were in contact and deformation continued until construction failure. Typical failure mode was local buckling in purlin sections near outer bolts. They also proposed an analytical method to calculate purlin end rotation in every phase. The third phase only estimated web deformations in bearing. The technique overestimated web-bearing deformations in order to give the conservative results of stiffness.

Apex connections of bolted cold formed steel frame consist of two back-to-back purlins connected with an apex bracket. This type of construction can be used for freestanding ridge. Lim and Nethercot (2003), Elkersh (2010), Pernes and Nagy (2012) investigated apex connections using both an experiment and the finite element method. The researchers tested the single span purlin systems connected at the middle of the span and loaded right to the connection. Specimens were laterally supported to prevent out-of-plane displacement. The test results showed that there was a good agreement between both types of methods. The number of bolts and the distance between them made the greatest influence on the rotational stiffness of the joint. The flexural failure of the connected sections was the most critical. Lim and Nethercot (2004) suggested a method to calculate the rotational stiffness of a bolted connection, which evaluated only the deformations of bearing bolts. Galatanu *et al.* (2009) also investigated single span cold formed steel purlins that were connected using gusset plates. The load was applied as a concentrated force in the middle of the span. The specimens that were used in the experiment were strengthened using stiffeners. The stiffeners were cold formed steel profiles that were connected using bolts. The results showed that the stiffeners increased the bearing capacity up to 35%.

The typical failure modes for sleeve and overlapped connections were local buckling in the outer web of purlin section, tension fracture failure in the sleeve or lap section, local buckling in the compression flange of sleeve or lap section and elongation of boltholes. Failure modes for gusset plate connections are local buckling of gusset plate and purlins, and elongation of boltholes. Increased connection size and the amount of bolts in the connections were proposed as techniques to increase load bearing.

To sum up, the majority of publications agree, that cold formed beam to beam bolted connections are always semi-rigid and partially resistant. Various techniques to calculate moment–rotational behaviour were recommended. Lim and Nethercot (2004), Dubina and Ungureanu (2010), Ho and Chung (2006b) used natural experiments to find that the bearing forces around the bolthole were the main effect influencing the rotational stiffness. The presented methods for calculation of stiffness of connections resulted from the tension experiments of bolted connections and evaluated only the bolthole elongation. The biggest disadvantage is that the methods are not very accurate because they do not evaluate the deformations of purlins (web distortion, friction between components, flange deformations due to shear and bending). Yang and Liu (2012) separated the behaviour of connections into three phases and proposed formulas to calculate rotational stiffness for every phase, but the last one evaluates only stiffness due to the elongation of boltholes). The analysis of the proposed methods shows, that all methods are insufficiently good for accurate prediction of the behaviour of connections.

3.2. Beam-to-column connections

The most popular way to connect a beam to a column in portal frames is using gusset plates. Wong and Chung (2002) tested gusset plate connections. The scientists performed 16 beam-column sub-frame tests with different connection configurations under lateral loads. The load was applied as a concentrated horizontal force at the top of the column. To avoid lateral buckling, lateral supports were used. The authors investigated the influence of gusset plate thickness, the chamfers presence and the distance between bolts on the strength and stiffness of connections. The results showed that the difference of connection behaviour was greater when the distance between boltholes increased from 180 mm to 240 mm than from 150 mm to 180 mm. Connections with 10 mm thick gusset plates and 50 mm chamfers showed similar behaviour to those with 16 mm thick gusset plates but without chamfers. The moment resistance was from under 50% to over 85% of the moment capacities of connected sections. Subsequent to the analysis of the test data, Yu *et al.* (2005) suggested a method for semi-empirical design rule to calculate rotation stiffness of gusset plate connection. Dubina (2008) adapted EN1993-1-8 method of stiffness and strength calculation for cold formed knee joint calculation.

Sabbagh *et al.* (2011) investigated cold formed steel beam to column connections for seismic loads using finite element analysis. Pinned supports were modelled at both ends of columns and concentrated load was used at the end of beams. In numerical analysis, post-buckling behaviour of beam-to-column connection was modelled. Firstly, authors compared the behaviour of the beam with different types of flanges. The data showed that the beams with curved flanges demonstrated higher ductility than non-curved ones. Secondly, the optimum stiffener configuration in the connection to increase moment capacity and ductility was proposed. Test results showed that stiffeners could increase the moment strength by up to 35% and the ductility by up to 75%. In other papers, Sabbagh *et al.* (2012a, b, 2013) performed experiments and compared their results with previous numerical analysis. Experiment results demonstrated a reasonable agreement with the finite element method. Chung and Lau (1999) investigated the structural performance of three types of beam-to-column connections: triangular, rectangular and haunched. Hinged support was used at the end of the column and load was applied at the end of the beam with direction toward the end of the column. Lateral supports to prevent lateral buckling of beam and column members were used. The lateral torsional buckling of gusset plates was found to range from 48% to 77% and the maximum flexural failure in the connected members was found to be 84% of the moment capacity of the connected members. They showed that haunched configuration had the highest rotational stiffness of over 1500 kNm/rad.

Another practical way to connect a beam to a column is using steel angles. Hazlan and Mahen (2010) performed an experiment using four different types of bolted connections. The connections were made using different arrangement of cold formed steel angle connectors. A beam and a column were lipped C-sections. Pinned supports were used at the ends of the column and concentrated load was used at the end of the beam. Lateral restraints were used for the beam and the column. Analysis data showed that moment capacity ranged from 70% to 100% of the connected member capacity. Tan *et al.* (2013) performed a similar experiment as well as the numerical analysis of the beam to column joints, which were connected using angles. The results differed up to 7% and 9%. Authors claimed that using nonlinear numerical analysis is very important because cold formed steel joints rotate at large deformations.

There is a suitable way to connect a cold formed steel beam to a column directly using bolts without any other components. This method is easier to assemble and costs less. Uang *et al.* (2010) presented experimental data on connection of a double channel beam to a HSS column. Pinned supports were used at the ends of the column and concentrated vertical load was used at the end of the beam. A lateral bracing system was used to prevent the beam from lateral buckling. The specimens were tested under a cyclic load. A model to calculate bolted moment connections was proposed to evaluate the friction and bearing forces around the bolthole.

Another way to connect a beam to column members is using welds. Moazed *et al.* (2009) performed the finite element analysis of a T-joint connection of thin-walled square tubes. In the paper, the authors presented a method to calculate joint rotational stiffness. The method was proposed to use beam type finite elements with semi rigid joints instead of shell or solid elements that simplified the construction behaviour calculations.

The most frequent failure mode for connections using steel angles is local buckling in the column flange. Other failure modes may be local buckling in the beam section and elongation of boltholes. Typical failure modes for connections using gusset plates are the same as in beam-to-beam connections: local buckling in beam, column sections and gusset plate, elongation of boltholes. For beam-to-column connections, the most frequent failure mode is local buckling in the web, sometimes elongation of boltholes may occur.

The reviewed papers show that cold formed steel sections can be used in moment resisting frames. Beam-to-column connections of cold formed steel sections show sufficient amount of strength and stiffness. Moreover, data on the finite element method results showed a good agreement with experimental data.

The analysis of papers shows that the behaviour of cold formed beam-to-column connections is usually semi rigid. Several techniques to calculate the stiffness of beam-to-column connections were presented. Tan *et al.* (2013) suggested empirical formula for the stiffness calculation of bolted connections that were received from the data of natural experiments using curve-fitting software. Yu *et al.* (2005) presented semi-empirical formula for flexibility prediction of the bolted moment connections. The technique the nonlinear behaviour of connection describes as tri-linear curve. Moreover, the calculation of the bolthole elongation was not suggested and the method did not take into account the location of the bolts that connect the column and the gusset plate. The review of papers shows the lack of accurate techniques to estimate the behaviour of cold formed beam-to-column connections.

3.3. Connections of steel trusses

Dawe *et al.* (2010) tested steel roof trusses. C-section truss elements were connected using self-drilling screws and steel plates. Structures were tested under a panel point loading. Various configurations of bolted joints were made by adding stiffeners. The stiffeners increased the load capacity by 24%. Zaharia and Dubina (2006) investigated the behaviour of bolted steel truss connections. Beams were made from C-sections connected using bolts. Based on experimental and numerical results, a method of joint rotational stiffness was proposed. The method evaluated only the bearing forces, as they are the main parameter for connection stiffness.

Pedreschi and Sinha (2008) studied mechanical clinching connection behaviour of cold formed steel trusses. Experiment results showed, that increasing number of clinches in connections increased stiffness and changed failure mode from failure of the connection to failure of the chord section. Moreover, clinch failure mode led to high ductility.

Typical failure mode for bolted truss connections was local buckling in sections and elongation of boltholes. Possible failure modes using mechanical clinching connection are clinch failure and local buckling in sections. Clinch failure is generally ductile.

To sum up, cold formed truss connections can be made using bolts, self-drilling screws and mechanical clinches. Experiment results show that truss systems made from cold formed sections are capable of taking roof loads.

3.4. Cold formed steel frames

The literature also provides experimental data of frames made from cold formed steel profiles. These types of experiments lead to better understanding of the influence cold formed steel connections have on the behaviour of the whole structure. Therefore, cold formed steel connections are usually semi-rigid. It is very important to investigate the influence of connections on stability and strength of frames.

Ali *et al.* (2010) investigated cold formed steel frames. Columns and beams were formed from single cold formed C-sections. The scientists performed 10 frame tests with different amount of bolts in the column base connections. Concentrated horizontal load at the top of the column and lateral restraints were used. Results showed that the column base with four bolts had rotational stiffness of approx. 90 kNm/rad and made a huge influence on the strength of the entire structure. Moreover, the strength of the frame was proportional to the strength of the column base connection. In the following article, Ali *et al.* (2011) performed a numerical analysis and compared the results with previous experiment data. Both data showed satisfactory agreement. Dubina (2008) tested two types of cold formed steel portal frames. Columns and beams were made from lipped back-to-back C-sections. All connections were made using gusset plates. Pinned supports were used for column base connections. The result showed that behaviour of connections was always semi rigid. It was claimed that the strength and the stiffness characteristics of the structures could be overestimated if connections were assumed rigid. Tan (2001) examined frames with angle connections experimentally and concluded that the section properties had a greater influence on the frame behaviour than the connection properties. Enlarging lip size from 0 to 12 mm increased the ultimate load of the frame by 46% and modifying connection thickness from 4.7 to 8.4 mm by 16%. Tan also proposed empirical formula to calculate joint rotation what was in a good agreement with the test results.

Reviewed papers prove that cold formed steel frames can be used in construction. These light gauge metal frames show high strength properties to carry roof loads. Moreover, the connections of cold formed steel sections demonstrate good stiffness properties.

4. Numerical investigation of cold formed steel beam to column gusset plate bolted connection

The literature review shows that many researchers only considered the web bearing in the calculation of the connection stiffness. Thus, a numerical analysis was made to investigate the influence of gusset plate shape and thickness on the stiffness of all connections. The distance between bolts and the diameter of bolts were the same; therefore, bearing deformations were considered as a constant. It should be mentioned that the numerical model was calibrated using the results of Wong and Chung (2002) experiments. The difference between the results showed satisfactory agreement (failure mode was the same, strength differed by 0.2% and the ultimate rotation differed by 16.0%).

4.1. The geometry of models

The numerical experiment was performed using Ansys 14.0 software. The rotation and the strength of connection were investigated. The total of ten models were modelled. Half of all specimens were modelled from sections C150×40×1.5 and the other half C150×40×2.5 (height, width and thickness of a section) were modelled. Eight specimens were with gusset plates and the other two were connected without them (using hinged connections) (Fig. 8). The thicknesses of the gusset plate were 6 mm and 12 mm. Two different types of shapes of gusset plates were modelled (Fig. 9). Gusset plates were modelled from S355 steel, the beam and the column were modelled from G350 steel and bolts were modelled from 4.6 steel. The diameter of bolts was 16 mm.

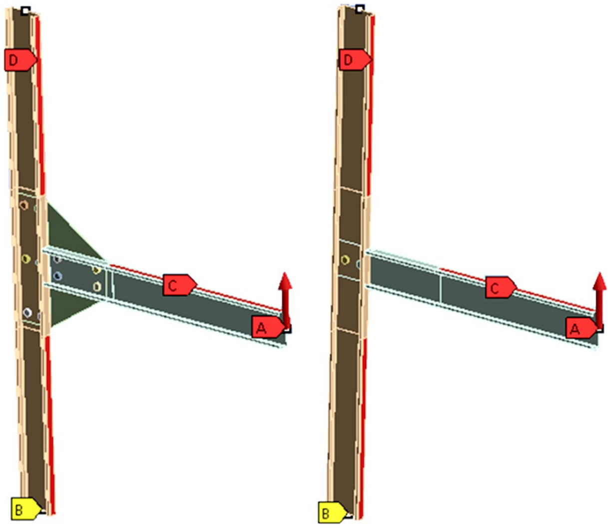

Fig. 8. Numerical simulation models. Bolted connection with gusset plate on the left; without gusset plate on the right (A – load point, B – hinged supports on the top and on the bottom of the columns, C and D – lateral supports)

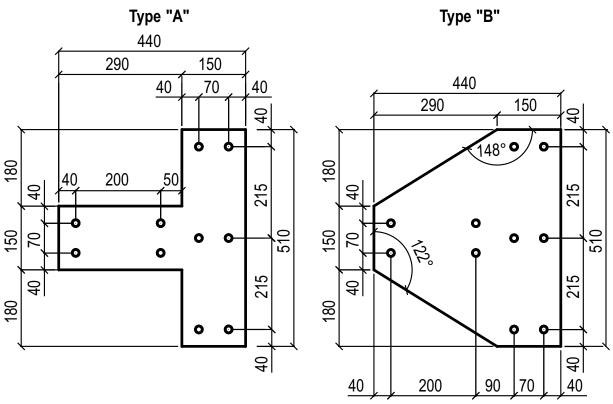

Fig. 9. Geometry of gusset plates

4.2. The results of numerical analysis

The data of the analysis showed the following results (Figs 10 and 11):

- The increase in the thickness of the A type gusset plate from 6 mm to 12 mm decreased the rotation by 34.7% and 58% for connections modelled from C150×40×1.5 and C150×40×2.5 sections, respectively;
- The increase in the thickness of the B type gusset plate from 6 mm to 12 mm decreased the rotation by 52.1% and 56.9% for connections modelled from C150×40×1.5 and C150×40×2.5 sections, respectively;
- Changing the type of 6 mm thick gusset plate from type A to type B resulted in the rotation decrease of 3.0% and 36.0% for connections modelled from C150×40×1.5 and C150×40×2.5 sections, respectively;

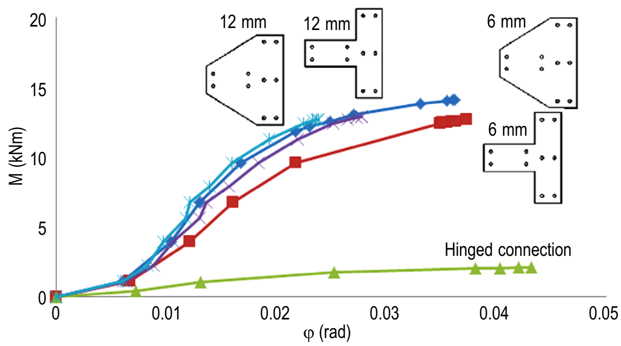

Fig. 10. Results of the connections of C150×40×1.5 sections

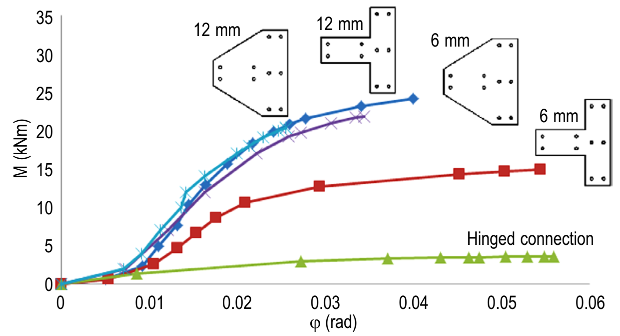

Fig. 11. Results of the connections of C150×40×2.5 sections

– Changing the type of 12 mm thick gusset plate from type A to type B resulted in the rotation decrease of 16.4% and 34.9% for connections modelled from C150×40×1.5 and C150×40×2.5 sections, respectively.

The data of the results showed that the influence of the gusset plate shape and thickness is high. In case of the gusset plate failure due to bending and shear, the rotation of the A type 6 mm thick connection increased almost twice (Fig. 11). It demonstrates that it is not enough to evaluate only the elongation of boltholes to predict the stiffness of a gusset plate bolted connection. Moreover, further research of optimal shape and thickness of gusset plate is necessary.

Three failure modes were observed during the numerical analysis: local buckling in the beam web, bearing forces in the web and gusset plate failure due to bending and shear. All connections that were made of C150×40×2.5 sections, except for the hinged connection, failed due to local buckling. All connections that were made of C150×40×1.5 sections, except for the hinged connection and the connection with A type 6 mm thick gusset plate, failed due to local buckling. The connection with A type 6 mm thick gusset plate failed due to bending and shear in gusset plate. Both hinged connections failed due to bearing forces in the web.

Summary and conclusions

According to the literature of the review and performed numerical analysis, the following conclusions are made:

1. Generally, cold formed steel construction failure modes are local stability loss in sections and elongation of boltholes;
2. There is a wide variety of techniques to connect cold formed sections using gusset plates, rivets, self-drilling screws, bolts, mechanical clinches or welds;
3. The majority of authors that tested cold formed steel connections agreed that the bearing forces around the bolthole have the greatest influence for bolted connection flexibility. Therefore, the increasing distance between bolts and amount of bolts increases the stiffness of a connection;
4. The behaviour of cold formed steel connections is in most cases semi-rigid and hinged. The literature proposes various types of stiffeners to achieve almost rigid behaviour of cold formed steel connections;
5. Chamfers show obvious effect on gusset plate connection stiffness;
6. Suitable stiffener configuration increases the moment capacity and ductility in connections;
7. The data of numerical analysis presented in this paper showed that the shape and the thickness have high influence on the stiffness of bolted connections. It was demonstrated that it is not enough to evaluate only the elongation of boltholes calculating the stiffness of cold formed steel connections;
8. There is no experiment where the influence of axial and shear load would be investigated in terms of the behaviour of connections;
9. The literature review showed that cold formed steel sections still have to be investigated. There is a lack of methods to predict the stiffness of connections.

Acknowledgements

This work has been supported by the European Social Fund as part of the project "Development and application of innovative research methods and solutions for traffic structures, vehicles and their flows", project code VP1-3.1-ŠMM-08-K-01-020.

References

Ali, B. A.; Saad, S.; Osman, M. H. 2010. Cold-formed steel frame with bolted moment connections, *International Journal of Civil and Structural Engineering* 3: 534–544.

Ali, B. A.; Saad, S.; Osman, M. H.; Ahmad, Y. 2011. Finite element analysis of cold-formed steel connections, *International Journal of Engineering* 2: 185–193.

Chung, K. F.; Ho, H. C. 2005. Analysis and design of lapped connections between cold-formed steel Z sections, *Thin-Walled Structures* 43(7): 1071–1090. http://dx.doi.org/10.1016/j.tws.2004.11.005

Chung, K. F.; Ho, H. C.; Wang, A. J. 2005. An investigation into deformation characteristics of lapped connections between cold-formed steel z sections, *Steel Structures* 5: 23–32.

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

Dawe, J. L.; Liu, Y.; Li, J. Y. 2010. Strength and behaviour of cold-formed steel offset trusses, *Journal of Constructional Steel Research* 66(4): 556–565. http://dx.doi.org/10.1016/j.jcsr.2009.10.015

Dubina, D. 2008. Structural analysis and design assisted by testing of cold-formed steel structures, *Thin-walled Structures* 46(7–9): 741–764. http://dx.doi.org/10.1016/j.tws.2008.01.030

Dubina, D.; Ungureanu, V. 2010. Behaviour of multi-span cold-formed Z-purlins with bolted lapped connections, *Thin-walled Structures* 48(10–11): 866–871. http://dx.doi.org/10.1016/j.tws.2010.04.003

Elkersh, I. 2010. Experimental investigation of bolted cold formed steel frame apex connections under pure moment, *Ain Shams Engineering Journal* 1(1): 11–20. http://dx.doi.org/10.1016/j.asej.2010.09.002

Eurocode 3. EN 1993-1-3. Supplementary rules for cold formed thin gauge members and sheeting. Part 1–3: *General Rules*. Brussels, 2006. 130 p.

Eurocode 3. EN 1993-1-8. Design of steel structures. Part 1–8: *Design of joints*. Brussels, 2005.

Galatanu, T. F.; Ciongradi, I. P.; Budescu, M.; Rosca, O. 2009. Experimental studies of the back-to-back connected cold formed steel profile bolted joints, *Gheorghe Asachi Technical University of Jassy:* 55.

Gutierrez, R.; Loureiro, A.; Lopez, M.; Moreno, A. 2011. Analysis of cold-formed purlins with slotted sleeve connections, *Thin-walled Structures* 49(7): 833–841. http://dx.doi.org/10.1016/j.tws.2011.02.008

Hazlan, A. H.; Mahen, M. 2010. Bolted beam-column moment connections between cold-formed steel members, in *21st Australasian Conference on the Mechanics of Structures and Materials (ACMSM 21),* 2005, Victoria University, Melbourne.

Ho, H. C.; Chung, K. F. 2004. Experimental investigation into the structural behaviour of lapped connections between cold-formed steel Z sections, *Thin-walled Structures* 42(7): 1013–1033. http://dx.doi.org/10.1016/j.tws.2004.03.022

Ho, H. C.; Chung, K. F. 2006a. Analytical prediction on deformation characteristics of lapped connections between cold-formed steel Z sections, *Thin-walled Structures* 44(1): 115–130. http://dx.doi.org/10.1016/j.tws.2005.08.009

Ho, H. C.; Chung, K. F. 2006b. Structural behavior of lapped cold-formed steel Z sections with generic bolted configurations, *Thin-walled Structures* 44(4): 466–480. http://dx.doi.org/10.1016/j.tws.2006.03.012

Yang, J.; Liu, Q. 2012. Sleeve connections of cold-formed steel sigma purlins, *Engineering Structures* 43: 245–258. http://dx.doi.org/10.1016/j.engstruct.2012.05.021

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. http://dx.doi.org/10.1016/j.jcsr.2005.03.001

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. http://dx.doi.org/10.1016/S0263-8231(03)00045-4

Lim, J. B. P.; Nethercot, D. A. 2004. Stiffness prediction for bolted momentconnections between cold-formed steel members, *Journal of Constructional Steel Research* 60(1): 85–107. http://dx.doi.org/10.1016/S0143-974X(03)00105-6

Moazed, R.; Szyszkowski, W.; Fotouhi, R. 2009. The in-plane behaviour and FE modeling of a T-joint connection of thin-walled square tubes, *Thin-walled Structures* 47(6–7): 816–825. http://dx.doi.org/10.1016/j.tws.2009.01.006

Pedreschi, R. F.; Sinha, B. P. 2008. An experimental study of cold formed steel trusses using mechanical clinching, *Construction and Building Materials* 22(5): 921–931. http://dx.doi.org/10.1016/j.conbuildmat.2006.12.014

Pernes, P. M.; Nagy, Z. 2012. FE modeling of cold-formed steel bolted joints in pitch-roof portal frames, *Acta Technica Napocensis: Civil Engineering & Architecture* 3: 234–242.

Sabbagh, A. B.; Petkovski, M.; Pilakoutas, K.; Mirghaderi, R. 2011. Ductile moment – resisting frames using cold – formed steel sections: an analytical investigation, *Journal of Constructional Steel Research* 67(4): 634–646. http://dx.doi.org/10.1016/j.jcsr.2010.11.016

Sabbagh, A. B.; Petkovski, M.; Pilakoutas, K.; Mirghaderi, R. 2012a. Development of cold-formed steel elements for earthquake resistant moment frame buildings, *Thin-walled Structures* 53: 99–108. http://dx.doi.org/10.1016/j.tws.2012.01.004

Sabbagh, A. B.; Petkovski, M.; Pilakoutas, K.; Mirghaderi, R. 2012b. Experimental work on cold-formed steel elements for earthquake resilient moment frame buildings, *Engineering Structures* 42: 371–386. http://dx.doi.org/10.1016/j.engstruct.2012.04.025

Sabbagh, A. B; Petkovski, M.; Pilakoutas, K.; Mirghaderi, R. 2013. Cyclic behaviour of bolted cold-formed steel moment connections: FE modelling including slip, *Journal of Constructional Steel Research* 80: 100–108. http://dx.doi.org/10.1016/j.jcsr.2012.09.010

Tan S. H. 2001. Channel frames with semi-rigid joints, *Computers & Structures* 79(7): 715–725. http://dx.doi.org/10.1016/S0045-7949(00)00173-5

Tan, C. S.; Lee, Y. H.; Lee, Y. L.; Mohammad, S.; Sulaiman, A.; Tahir, M. M.; Shek, P. N. 2013. Numerical simulation of cold-formed steel top-seat flange cleat connection, *Jurnal Teknologi* 3: 63–71.

Uang, C. M.; Hong, J. K.; Sato, A.; Wood, K. 2010. Cold-formed steel special bolted moment frames: cyclic testing and numerical modeling of moment connections, *Journal of Structural Engineering* 8.

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. http://dx.doi.org/10.1016/S0143-974X(01)00044-X

Zaharia, R.; Dubina, D. 2006. Stiffness of joints in bolted connected cold-formed steel trusses, *Journal of Constructional Steel Research* 62(3): 240–249. http://dx.doi.org/10.1016/j.jcsr.2005.07.002

Zhang, L.; Tong, G. 2008. Moment resistance and flexural rigidity of lapped connections in multi-span cold-formed Z purlin systems, *Thin-walled Structures* 46(5): 551–560. http://dx.doi.org/10.1016/j.tws.2007.10.010

ŠALTAI FORMUOTŲ PLIENINIŲ KONSTRUKCIJŲ MAZGŲ ELGSENA

Ž. Bučmys, G. Šaučiuvėnas

Santrauka. Apžvelgti mokslinės literatūros šaltiniai apie šaltai formuotų profiliuočių naudojimą ir jų jungtis. Kadangi šie profiliuočiai yra gerų stiprumo savybių ir pigiau atsieina, tad nūdienos statyboje taikomi vis dažniau. Apibendrinami naujausi sijos su sija, sijos ir kolonos bei santvaros mazgų elgsenos tyrimai. Šaltai formuotų elementų jungčių elgsena yra kaip pusiau standžių mazgų. Pagrindinis veiksnys, turintis įtaką mazgo standžiui, yra varžtų glemžiamasis poveikis plonasienių elementų skerspjūvių sienelei. Pateikiama rekomendacijų, kaip didinti šaltai formuotų profiliuočių jungčių standumą.

Reikšminiai žodžiai: šaltai formuoti plieniniai profiliuočiai, jungčių elgsena, plonasienės konstrukcijos, mazgai, pusiau standūs mazgai.

Žilvinas BUČMYS. PhD student at the Department of Steel and Timber Structures, Vilnius Gediminas Technical University (VGTU), Lithuania. Žilvinas Bučmys received his Bachelor's degree in Civil Engineering in 2009 and Master's degree in Civil Engineering at Vilnius Gediminas Technical University in 2012. Research interests: behaviour of cold formed steel connections, calculation methods of cold formed steel.

Gintas ŠAUČIUVĖNAS. Assoc. Prof. Dr, Department of Steel and Timber Structures, Vilnius Gediminas Technical University. Research interests: evaluation of existing steel and timber structures, evaluation of fire resistance of steel and timber structures, behaviour of the elements and structures made of steel, timber, glass and steel-concrete composite.