FEM analyses on beam-to-column connection for a thin-walled cold-formed steel frame

V M Venghiac1, I Olteanu-Donțov1, M S Alexa-Stratulat1, G Țăranu1, S A Băetu1, I O Toma1
1 The “Gheorghe Asachi” Technical University of Iasi, Faculty of Civil Engineering and Building Services, no. 1, Prof.dr.doc. D. Mangeron Street, 700050, Iasi, Romania
E-mail: ionut.ovidiu.toma@tuiasi.ro

Abstract. The advantage of fast construction for thin-walled cold-formed steel ground floor structures is somehow lessened by the challenges the engineers encounter when designing the connections. The horizontal forces induced by the seismic motions and not only, represent a serious threat to flexible structures. The paper presents the results obtained from comparative FEM analyses of different types and layouts of beam-to-column connection in case of a thin-walled cold-formed steel frame. The obtained results were analysed and compared from the point of view of maximum vertical displacements of the beam free end, the load-displacement diagrams as well as the principal stresses. The objective was to obtain the most efficient type of connection that ensures the safe transfer of internal forces from the beam to the column.

1. Introduction
Cold formed steel (CFS) structures have seen a tremendous expansion over the past decades being used in a variety of applications ranging from civil engineering to mechanical and even aeronautical engineering [1]. Their first use in civil engineering was somehow limited to secondary structural elements such as roof purlins, side-rails for wall cladding, a.s.o. However, due to their higher strength per unit weight compared to hot rolled steel elements, ease and speed of construction coupled with a higher flexibility in manufacturing, the demand for CFS elements in structural applications has spiked [2, 3]. This resulted in extensive research works being conducted to assess the behaviour to CFS structures.

Such structures are prone to premature local buckling and instability issues due to the small thickness of CFS sections that typically ranges from 0.9 mm to 3.2 mm [4, 5]. This may result in limited applications especially for structures expected to withstand extreme loading scenarios such as moderate to strong earthquakes. The seismic behaviour has been extensively investigated, with the majority of research works being focused on the CFS wall frame panels as one of the major lateral load bearing systems [6]. Only recently, the application of CFS members to moment-resisting frames has started to gain attention and therefore the information available is rather scarce [1, 7].

One of the main advantages of using moment resisting CFS frames is the higher flexibility they offer for space planning and future changes in building destination and intended use. Additionally, moment resisting CFS frames could improve the energy dissipation capacity of a CFS structure during significant seismic events [1, 7].

An important part in investigating the seismic behaviour of any structural system is the correct assessment of the joint behaviour, be it for concrete [8, 9], steel [10-12], timber, CFS [13-15] or aluminium structures.

Numerical simulations have quickly become an indispensable and efficient tool [16, 17] for assessing the behaviour of different structural systems and/or subsystems to a variety of loading scenarios. Once a numerical model was validated, it could be used for a wide range of parameter variations in order to grasp the influencing factors that may govern the response of members or structures to extreme loading events such as moderate to strong earthquakes [15, 18, 19].
Finite Element (FE) modelling is extensively used to assess the behaviour of CFS connections with various configurations [15, 20]. The results of previous studies highlighted the importance of detailed FE models to predict the behaviour of CFS connections under either monotonic or cyclic loading [21, 22].

The paper presents the results obtained by means of linear and non-linear static analyses on the connection joint between the CFS beam and the column of a portal frame. Four different configurations were considered consisting in the geometry of the node, presence of stiffening elements and the number of bolts. The obtained results show significant influence of the node geometry and configuration on the distribution of stresses in all components of the considered joint.

2. Methodology

2.1. Geometry and dimensions

The geometry of the considered model is shown in figure 1a. Both the beam and the column are made of two 150×45×10 mm CFS C-shape lipped profiles, figure 1b, positioned back-to-back, having a thickness of 2 mm.

![Figure 1. Geometry of the model](image)

2.2. Material

The CFS profiles were made of S235 mild steel with a longitudinal modulus of elasticity, \( E = 210 \) GPa, shear modulus, \( G = 81 \) GPa, Poisson’s ratio 0.3 and a yield strength of \( f_y = 235 \) MPa.

2.3. Numerical model

The numerical model was generated following the geometry given in figure 1. ANSYS 2021R1 [23] computer package was used to run the analyses. Four distinct configurations of the connection joint between the beams and the column were considered, as described in table 1.

The maximum mesh size was 20 mm resulting in an average number of 12500 elements for the considered configurations. SHELL181 quadratic plane stress element was used to generate the mesh, as seen in figure 2.

The joint assembly was considered to be fixed at the lower end of the column and the load was applied incrementally at the free end of the beam, pointing downward. Static non-linear finite element analysis (FEA) was run in order to check potential local losses of stability. The obtained results were analysed and compared from the point of view of maximum vertical displacements of the beam free end, the load-displacement diagrams as well as the principal stresses.
The main parameters of the numerical analysis were: the shape of the gusset/connection plate, the influence of stiffeners and the diameter of the bolts.

**Table 1. Joint layouts considered in the research.**

| Designation | Description | Layout |
|-------------|-------------|--------|
| Model 1     | Connection plate cut at 90°; thickness of 4mm; M10 bolts – bonded contacts between bolts and steel plate | ![Model 1 Layout](image1) |
| Model 2     | Connection plate cut at 90°; thickness of 4mm; M10 bolts (bond contact); exterior stiffening plates – the plate was not connected to the top flange of the beam | ![Model 2 Layout](image2) |
| Model 3     | Connection plate cut at 45°; thickness of 4mm; M10 bolts (bond contact); exterior and interior stiffening plates | ![Model 3 Layout](image3) |
| Model 4     | Connection plate cut at 45°; thickness of 4mm; M16 bolts (bond contact); exterior and interior stiffening plates | ![Model 4 Layout](image4) |

a. Model 1 b. Model 2
c. Mode 3  
d. Model 4

3. Results and discussions

3.1. Displacements

The magnitudes of the vertical displacement at 1000N are shown in figure 3. The highest vertical displacement was obtained for Model 1, 4.68 mm. The presence of the stiffening plate in Model 2 helped reducing the displacement to 4.12 mm, almost 14% smaller.

A significant reduction in the magnitude of the vertical displacement was observed for Model 3 for which the gusset/connection plate was cut at a 45° angle. The obtained vertical displacement was 2.45 mm, almost half the displacement obtained for Model 1. Increasing the bolt diameter from 10 mm to 16 mm helped further reducing of the displacement (increasing the stiffness of the node) by 4% to 2.34 mm.

Increasing the load to 2000 N resulted in failure of Models 1 and 2. They both failed due to lateral buckling of the connection plate, as it can be seen from figure 4a and 4b. The abnormal high values for the displacement of the free end in case of Models 1 and 2 are a clear indication of the failure.

On the other hand, Models 3 and 4, for which a different shape of the connection plate was adopted and additional stiffening plates were used, exhibited displacements of 4.90 mm and 4.69 mm, respectively. Comparing the obtained results with the data for an intensity of the applied load of 1000 N, a linear variation could be observed between the two loading steps.
Figure 3. Vertical displacements at 1000 N

Figure 4. Vertical displacements at 2000 N
3.2. Principal stresses

The intensity of the principal stresses was obtained for each calculation step of the numerical analysis. Figure 5 presents the principal stress distribution and the intensity of principal stresses for the magnitude of the applied load of 1000 N. For each of the considered models it can be observed that stress concentrations occurred at the level of the bolts.

The shape of the connection plate plays an important role, as it can be seen from the maximum values of the principal stresses of Model 3 compared to Models 1 and 2, shown in figure 5. There is a 23.3% reduction in the magnitude of stresses for the same intensity of the applied load.

The diameter of the bolt seems to be another significant influencing factor. The stress level drops by 46.15% when increasing the diameter of the bolt from 10 mm to 16 mm, as seen from figure 5d.

The same trends were observed for the ultimate loads obtained for each model. Model 1 failed due to local buckling of the connection plate at a load equal to 1376.8 N. Model 2 failed also due to lateral buckling of the connection plate, at the beam level, but at a higher load, 1889.3 N. This means a 37.22% increase in the load carrying capacity. This behaviour could be explained by the fact that the stiffening plate was not connected to the upper flanges of the beam. Model 3, figure 5c, showed stress concentrations in the upper row of bolts located at the beam end that exceeded the yield strength when the load reached 2400 N. The failure of the assembly changed from loss of stability to local crushing. Model 4 was able to resist the highest load, up to 4226.8 N. The increase of the bolt diameter did not change the failure mode compared to Model 3 but changed the location where the stress concentrations occurred. According to the numerical investigations, the part of the model that reached the yield strength was located at the level of the top row of bolts in the column, figure 5d. A further increase of the load resulted in general loss of stability of the model.

![Figure 5. Principal stresses at 1000N](image-url)
Based on the obtained data, the load carrying capacity of Model 4 increased by 307% compared to Model 1, by 222% compared to Model 2 and by 167% compared to Model 3. The general behaviour of the joint obtained by numerical simulations in the present study is consistent to the results presented in the scientific literature [24].

3.3. Load-displacement curves
The load-displacement curves of the four models are shown in figure 6. It can be observed that Models 1 and 2 have similar initial stiffness, despite Model 2 having an additional stiffening plate. Similarly, Models 3 and 4 have comparative initial stiffness with Model 4 exhibiting a slightly larger value due to the larger diameter bolts used to connect the elements.

The stiffening plate in Model 2 leads to a higher resisting load before local yielding of the material occurs coupled with the loss of stability.

![Figure 6. Load-displacement curves](image)

The data is consistent with the observed patterns presented in figure 4 and figure 5. The loss of stability lead to very large displacements recorded for Models 1 and 2, as seen in figure 4a and 4b. For the same intensity of the applied load, the maximum principal stresses reached the yield strength of the material.

4. Conclusions
The paper presents the results obtained by means of non-linear static analysis numerical simulations of a beam-to-column connection joint (eave joint) under four different configurations. Based on the obtained results, the following conclusions can be drawn:

A significant reduction in the magnitude of the vertical displacement was observed for Model 3 for which the gusset/connection plate was cut at a 45° angle. The obtained vertical displacement is 2.45 mm, almost half the displacement obtained for Model 1. Increasing the diameter of the bolt results in a further decrease of the vertical displacement because the joint becomes more stiff.

The shape of the connection plate plays an important role, as it can be seen from the maximum values of the principal stresses of Model 3 compared to Models 1 and 2. There is a 23.3% reduction in the magnitude of stresses for the same intensity of the applied load. The diameter of the bolt is another
significant influencing factor. The stress level drops by 46.15% when increasing the diameter of the bolt from 10 mm to 16 mm.

The numerical analyses are conducted in order to shed some light in the failure mode of the connection joint between a beam and a column made of CFS C-shaped lipped profiles. The model needs further calibration and validation based on experimentally obtained data during the next stage of the research. Numerical simulations based on the finite element method are versatile tools that allow for large parametric analyses in order to optimize the design process and to reach a balance between ultimate strength versus material consumption.

Acknowledgments
The authors wish to express their gratitude to the partnership between The “Gheorghe Asachi” Technical University of Iasi, INAS S.A. and ANSYS for making available the full research license for Ansys 2021R1 used in conducting the research. The authors thank S.C. Panoterm Import-Export S.R.L. for supplying the CFS profiles used in the research as part of the grant no. 4887/03.2019.

5. References
[1] Mojtabaei S M, Kabir M Z, Hajirasouliha I and Kargar M 2018 Analytical and experimental study on the seismic performance of cold-formed steel frames Journal of Construction Steel Research 143 18-31
[2] Rosca O V 2011 Experimental tests on thin-walled steel roof profiles Bulletin of the Polytechnic Institute of Iasi, Constructions. Architecture Section LIV (4) 85-95
[3] Karabulut B and Soyoz S 2017 Experimental and analytical studies on different configurations of cold-formed steel structures Journal of Construction Steel Research 133 535–46
[4] Lee Y H, Tan C S, Mohammad S, Tahir M M and Shek P N 2014 Review on cold-formed steel connections The Scientific World Journal 2014 ID. 951216
[5] Smith B H, Arwade S R, Schafer B W and Moen C D 2016 Design component and system reliability in a low rise cold formed steel framed commercial building Engineering Structures 127 434–46
[6] Schafer B W, Ayhan D, Liu P, Padilla-Llano D, Peterman K D, Stehman M, Buonopane S G, M. Eatherton M, Madsen R, Manley B, Moen C D, Nakata N, Rogers C and Yu C 2016 Seismic response and engineering of cold-formed steel framed buildings Structures 8 (Part 2) 197–212
[7] Bagheri Sabbagh A, Petkovski M, Pilakoutas K and Mirghaderi R 2012 Experimental work on cold-formed steel elements for earthquake resilient moment frame buildings Engineering Structures 42 371–386
[8] Campian C M, Pop M and Chira N 2017 The influence of the connections between concrete and steel on the joints modelling ce/papers – Proc. of Eurosteel, September 2017, Copenhagen, Denmark 1(2-3) 4680-7.
[9] Sococol I, Mihai P and Olteanu-Dontov I 2019 Ductility-concept for improving the seismic response for structural reinforced concrete frame systems Bulletin of the Polytechnic Institute of Iasi, Constructions. Architecture Section 65(1) 17-30
[10] Budescu M, Ciogradi I P and Rosca OV 2005 Experimental studies of a series of high strength friction grip bolted joints Bulletin of the Polytechnic Institute of Iasi, Constructions. Architecture Section LI(1-2) 23-9
[11] Melenciuc S C, Stefanucu A I and Olteanu I 2009 Seismic design of steel connections Bulletin of the Polytechnic Institute of Iasi, Constructions. Architecture Section LV(1) 49-56
[12] Mathe A E, Catarig A and Moldovan I 2015 Statics and kinematics of semirigid steel frames under seismic action Journal of Applied Engineering Sciences 5(2) 59-64
[13] Lim J B P, Hancock G J, Clifton G C, Pham C H and Das R 2016 DSM for ultimate strength of bolted moment-connections between cold-formed steel channel members *Journal of Construction Steel Research* **117** 196-203.

[14] Huynh M T, Pham C H and Hancock G J 2020 Design of screwed connections in cold-formed steels in shear *Thin-Walled Structures* **154** ID.106817

[15] Ye J, Mojtbaei S M, Hajiriasouliha I and Pilakoutas K 2020 Efficient design of cold-formed steel bolted-moment connections for earthquake resistant frames *Thin-Walled Structures* **150** ID.105926

[16] Pop M, Campian C M and Chira N 2019 Parametric study in composite joints configuration *IOP Conference Series Materials Science and Engineering (Proc. of International Conference Computational Civil Engineering, May 2019, Iasi, Romania)* **586** ID.012014

[17] Toma A M, Taranu G, Nishida T, Mathe A E, Campian C M, Baccay M and Toma I O 2020 Assessing the seismic performance of a R.C. frame structure by numerical simulations – an efficient tool for a sustainable future *Proc. of the International Conference on Building Services and Energy Efficiency, June 2020, Iasi, Romania* 231-240

[18] Mathe A E, Popa A G and Campian C M 2011 The influence of semi-rigid connections upon the performance of steel structures seismically excited *Acta Technica Napocensis: Civil Engineering and Architecture* **54**(3) 268-279

[19] Toma I O, Mihai P, Venghiac V M, Taranu G, Baetu S A, Toma A M and Petrescu C T 2020 Numerical simulations of the seismic behavior of a damaged rc frame retrofitted with composite fabric *Proc. of the World Conference on Earthquake Engineering, September 2020, Sendai, Japan* PaperID 3f-0019

[20] Ye J, Becque J, Hajiriasouliha I, Mojtbaei S M and Lim JB P 2018 Development of optimum cold-formed steel sections for maximum energy dissipation in uniaxial bending *Engineering Structures* **161** 55-67

[21] Bagheri Sabbagh A, Petkovski M, Pilakoutas K and Mirghaderi R 2013 Cyclic behaviour of bolted cold-formed steel moment connections: FE modelling including slip *Journal of Construction Steel Research* **80** 100-8

[22] Ozturk F and Pul S 2015 Experimental and numerical study on a full scale apex connection of cold-formed steel portal frames *Thin-Walled Structures* **94** 79-88

[23] Ansys 2021 R1, [https://www.ansys.com/products/structures/ansys-mechanical](https://www.ansys.com/products/structures/ansys-mechanical)

[24] Shahini M, Bagheri Sabbagh A, Davidson P, Mirghaderi R 2019 Development of cold-formed steel moment-resisting connections with bolting friction-slip mechanism for seismic applications *Thin-Walled Structures* **141** 217-231.