PARAMETRIC STUDY OF COLD FORMED STEEL JOINTS USING THE COMPONENT METHOD

2 authors:

Nagy Zsolt
Universitatea Tehnica Cluj-Napoca

Andrea Dezo
Universitatea Tehnica Cluj-Napoca

PARAMETRIC STUDY OF COLD FORMED STEEL JOINTS USING THE COMPONENT METHOD

Zs. Nagy & A. Dező
Technical University of Cluj-Napoca, Engineering Structures Department, Romania

A. A. Mureșan
Gordias SRL, Cluj-Napoca, Romania

ABSTRACT: In previous studies, two types of joints formed by back-to-back cold-formed C profiles as beam and SHS profile as column were investigated experimentally by subjecting them to monotonic and cyclic loads. Using the experimental data, the Finite Element (FE) models of the proposed joint configurations were calibrated. The two joint typology performance was then evaluated analytically with the component method following EN 1993-1-8 assisted by the calibrated FE models. The paper presents a parametric study of the joint's performance using component method with the purpose to assess its mechanical behavior for different thicknesses and materials of the cold-formed C profiles.

1 INTRODUCTION

The cold-formed structural elements are widely used in civil engineering due to several advantages such as light weight, high material strength, economical manufacture, short erecting time and good strength-to-weight ratio. Thus, in the past years there has been an increased interest in researching the behavior of cold-formed profiles as load bearing structural elements. The past studies focused on the buckling behavior of cold-formed structural elements and on the mechanical behavior and performance of cold-formed steel joints. Lee et al (2014) and Komara et al (2017) presented reviews on the current research of different types of cold-formed steel joints and their performance. Nagy et al (2006), Lim & Nethercot (2004), Chung & Lau (1999) and Öztürk & Pul (2015) demonstrated that bolted joints in cold-formed steel portal frames have, in most of the cases, semi-rigid behavior. Lim & Nethercot (2003) and Wong & Chung (2002) also showed that semi-rigid bolted joints are partially resistant.

Besides the bearing effect (i.e. bolt hole elongation), an important contribution to the global flexibility of the joints is the deformation induced by the local buckling or distortion of the thin-walled profiles. The premature local buckling of the cold-formed profile can cause the failure of the joint itself well below the expected load bearing capacity if the joint is unwisely configured. Dundu & Kemp (2006) observed that in case of back-to-back bolted connections, when the bolts are installed only on the web of the cold-formed steel sections, the local buckling becomes more critical from stress concentrations, shear lag and bearing deformations around bolt holes. The studies of Lim & Nethercot (2004), Yu et al (2005) and Ho & Chung (2006) showed that in case of usual cold-formed steel sections, the bearing work of bolts associated with elastic-plastic elongation of bolt holes is the most important component which controls the stiffness and the bearing capacity of such type of joints, as observed from experimental results and numerical simulations. The flanges in tension/compression due to bending and the web in shear due to transverse loading have a significantly lower contribution to the joint's stiffness and bearing capacity. In case of non-conventional joints, the identification of the relevant components is difficult. Therefore, Finite Element Analysis (FEA) based on calibrated models may be an alternative to develop analytical solutions with the component method, described in Eurocode 3 (EN 1993-1-8), even though the cold-formed steel joints are not covered by it.

The following paper presents a parametric study on two types of beam-to-column joints consisting in back-to-back cold-formed C profiles for beams and SHS-sections for columns using component method. The analyzed configurations are Diaphragm Connection for Beam to Column joint (DCBC) and Hollo Bolts for Beam to Column joint (HBBC). The purpose of the study is to assess the mechanical behavior of the proposed joint configurations in case of different thicknesses and materials of the cold-formed C profiles.

The back-to-back C profiles beams to SHS column joints were initially studied experimentally by Nagy et al. (2017) where the joints were tested under monotonic and cyclic loading. Then Nagy et al.

(2018a) investigated the proposed beam-to-column joint with Finite Element Method (FEM) and the models were calibrated with the experimental data previously determined. Finally, the component method was applied in Nagy et al. (2018b) for the analytical evaluation of the proposed joint configuration's performance. In case of joints with cold-formed elements, component method was applied assisted by FEA where the identification of components was performed with calibrated models.

2 THE CONFIGURATION OF THE BEAM-TO-COLUMN JOINT

In the following study, two joint configurations went through a parametric study using the component method described in Eurocode 3 (EN 1998-1-8). The joint configurations were as follows: (i) Diaphragm Connection for Beam to Column joint (DCBC) and (ii) Hollo Bolts for Beam to Column joint (HBBC). The following paragraphs describe the two joint configurations.

2.1 *Diaphragm Connection for Beam to Column Joint (DCBC)*

The DCBC joint (see Figure 1) consists in two cold-formed C-section profiles having the size C 300/3 and a column having the cross section SHS 200/12,5. The two members are connected by two welded flange diaphragms having the thickness 10 mm, thus the name "diaphragm connection". Each diaphragm is fixed to the beam's flange by four M20 10.9 grade bolts.

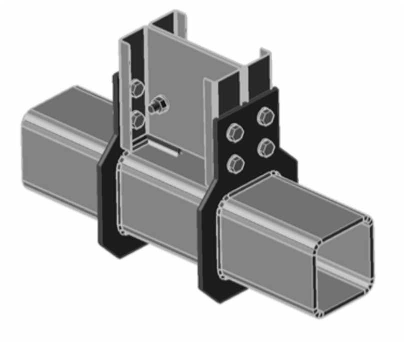

Figure 1. The Diaphragm Connection for Beam to Column Joint (DCBC).

2.2 *Hollo Bolts for Beam to Column Joint (HBBC)*

The HBBC joint (see Figure 2) also consists in two cold-formed C300/3 profiles and a column with the cross section SHS 200/12.5. Unlike the previous joint configuration, the HBBC has two T-stubs with the thickness 10 mm with stiffeners placed at the upper and lower flanges of the C-profiles. The stiffeners have the thickness 8 mm. Each T-stub is fixed to the column by four M20 hollo bolts, to the beam's flange by four M20 10.9 grade bolts and to the beam's web by one M20 10.9 grade bolt.

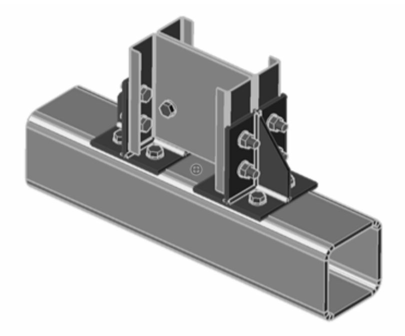

Figure 2. The Hollo Bolts for Beam to Column Joint (HBBC).

3 THE COMPONENT METHOD

The component method is a general design method for joints in steel structures and is implemented in Eurocode 3, part 1-8 (EN 1993-1-8). The method consists in the representation of the joint by a certain number of components and the evaluation of the force-displacement relationship of each component. The components which partially represent the joint behavior due to a single action are assembled according to the distribution of the internal forces within the joint.

The component method consists in the following steps (Jaspart, 2000): (i) the identification of the active components of the joint, (ii) the evaluation of the mechanical properties of individual components (i.e. bearing capacity and stiffness) and (iii) the assembly of the components for the evaluation of the bearing capacity and stiffness of the whole joint.

The component method was applied to the DCBC and HBBC joints in previous studies (Nagy et al, 2006, 2018b). In the case of cold-formed connections, the identification of the active components was based on experimental studies (Nagy et al, 2017) and on calibrated FEM models (Nagy et al, 2018a).

For the DCBC configuration, the following active components for the determination of the bearing capacity of the joint were identified, according to Nagy et al (2018a): (i) bracket in tension/compression, (ii) bracket in bearing (i.e. bolt hole elongation), (iii) bolts in shear, (iv) cold-formed C profiles in bearing and (v) cold-formed C profiles in tension/compression. The active components used to determine the stiffness of the joint in case of DCBC configuration are: (i) bolts in shear, (ii) bolts in bearing on the cold-formed C profile, (iii) bolts in bearing on the bracket and (iv) lower bracket in bending.

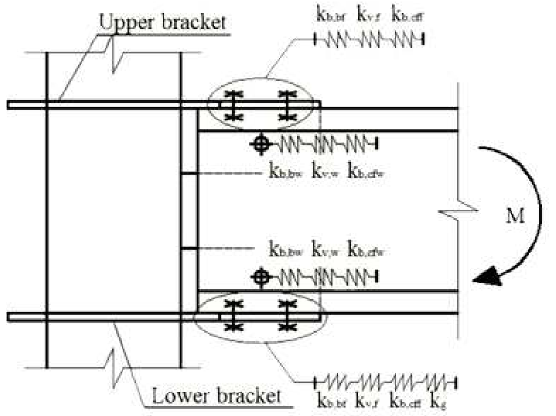

Figure 3. The active components of the DCBC joint (Nagy et al, 2018b).

In case of HBBC joint, some of the identified active components were similar to the DCBC joint and the other part consisted in components specific to the HBBC. The identified active components specific to HBBC configuration used to evaluate the joint's bearing capacity are, according to Nagy et al (2018a): (i) SHS column web in shear, (ii) SHS column flange in bending, (iii) hollo bolt in tension and (iv) bracket in tension/compression. The active components specific to HBBC configuration for the evaluation of the joint's stiffness are: (i) hollo bolt in tension, (ii) SHS column in transverse compression and tension (i.e. chord face failure), (iii) hollo bolt in shear, (iv) hollo bolt in bearing on the SHS column and (v) hollo bolt in bearing on the bracket.

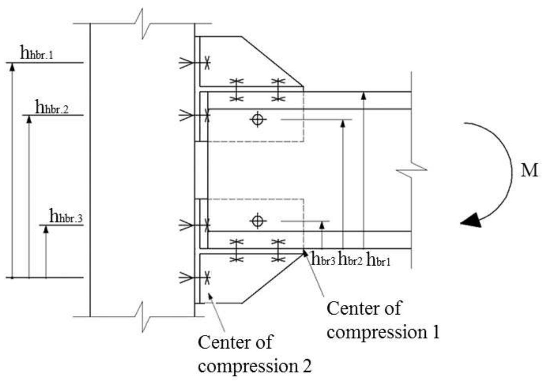

Figure 4. The active components of the HBBC joint (Nagy et al, 2018b).

4 THE PARAMETRIC STUDY

The parametric study was performed with the component method for the DCBC and the HBBC joints. In the component method analysis, the resistant moment $M_{C,Rd}$ and the initial rotational stiffness $S_{j,ini}$ of the two proposed joint configurations were determined. The parametric study consisted in two steps: (i) in the first step, the thickness and the steel grade of the C profiles were modified and (ii) in the second step, the thickness and the steel grade of the C profiles and the diameter and the steel grade of the bolts were modified. The rest of the components remained unchanged.

The thickness of the C profiles ranged between *1.5 mm* and *3 mm*. The chosen steel grades were, according to EN 10326 (2004): S 220 GD, S 250 GD, S 280 GD, S 320 GD and S 350 GD. The diameter of the bolts was as follows: M12 (i.e. *12 mm*), M16 (i.e. *16 mm*), M18 (i.e. *18 mm*) and M20 (i.e. *20 mm*). The steel grades of the bolts ranged between 4.6 and 10.9 according to EN 1993-1-8 (2003), Section 3.1.1., Table 3.1.

The following paragraphs present the results of the parametric study for each type of joint.

After the parametric study, the design moment resistance $M_{C,Rd}$ and the initial rotational stiffness $S_{j,ini}$ determined from component method with mechanical properties from experimental data and the design moment resistance $M_{C,Rd}$ and the initial rotational stiffness $S_{j,ini}$ determined with mechanical properties from Eurocode 3 were compared for each type of joint. The mechanical properties of the beam from the experimental data were as follows: the yielding stress was $f_y=507\ N/mm^2$ and the ultimate stress was $f_u=577\ N/mm^2$. These values were determined from coupon tests performed on the sample. The diameter of the bolts used in the experiment was M20, while their steel grade was 10.9 according to EN 1993-1-8, namely the yielding stress of the bolts was $f_{yb}=900\ N/mm^2$ and the ultimate stress of the bolts was $f_{ub}=1000\ N/mm^2$. The initial rotational stiffness resulted from the component method with experimental data is: (i) for the DCBC joint is $S_{j,ini}=2279.77\ kNm/rad$ and (ii) for the HBBC joint is $S_{j,ini}=2127.02\ kNm/rad$. The moment resistance is, in both cases, $M_{C,Rd}=120.87\ kNm$, because according to Nagy et al (2018b), failure occurs in the back-to-back C profile beam.

The differences between the results are also presented in the following paragraphs.

4.1 *Parametric study of the DCBC joint*

Figure 5 presents the initial rotational stiffness $S_{j,ini}$ and Figure 6 presents the design moment resistance $M_{C,Rd}$ of the DCBC joint depending on the C profile's thickness and on the steel's yielding strength f_y. According to Figure 5 and Figure 6, as the thickness of the C profile and the yielding strength f_y increase, the initial rotational stiffness $S_{j,ini}$ and the design moment resistance $M_{C,Rd}$ also increase.

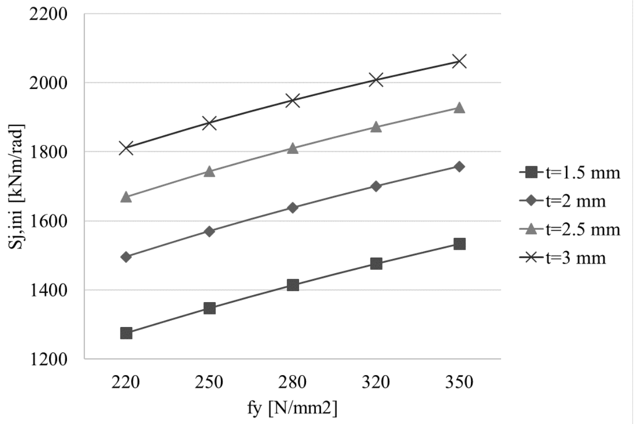

Figure 5. The initial rotational stiffness $S_{j,ini}$ of the DCBC joint depending on the C profile's thickness and on the steel's yielding strength f_y.

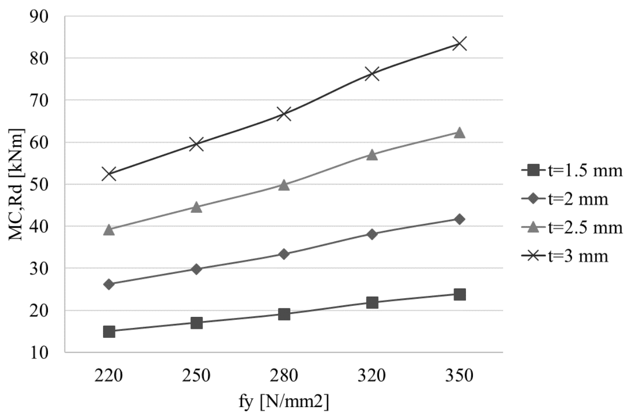

Figure 6. The design moment resistance $M_{C,Rd}$ of the DCBC joint depending on the C profile's thickness and on the steel's yielding strength f_y.

In Figure 7 is presented the initial rotational stiffness $S_{j,ini}$, while Figure 8 shows the design moment resistance $M_{C,Rd}$ of the DCBC joint depending on the bolt's diameter and steel grade, if the C profile's steel grade is S 350 GD (i.e. the yielding strength $f_y=350\ N/mm^2$ and the ultimate strength $f_u=420\ N/mm^2$) and the thickness of the profile is $t=3\ mm$. In Figure 7 and Figure 8 one can observe that as the diameter and steel grade of the bolt increase, the initial rotational stiffness and the design moment resistance also increase. However, in Figure 8 the design moment resistance has constant values for the M20 bolt regardless of the steel grade used, also showing that the failure of the joint occurs in the cold-formed profile beam. In case of the M12 bolts, according to Figure 8, the failure of the joint occurs in the bolts regardless of their steel grade. For the M16 and M18 bolts, starting from the bolt grade 4.6 (i.e. yielding strength $f_{yb}=240\ N/mm^2$, ultimate strength $f_{ub}=400\ N/mm^2$), the failure of the joint occurs in the bolts, but as the bolt grade increases, the failure of the joint occurs in the cold-formed profiles.

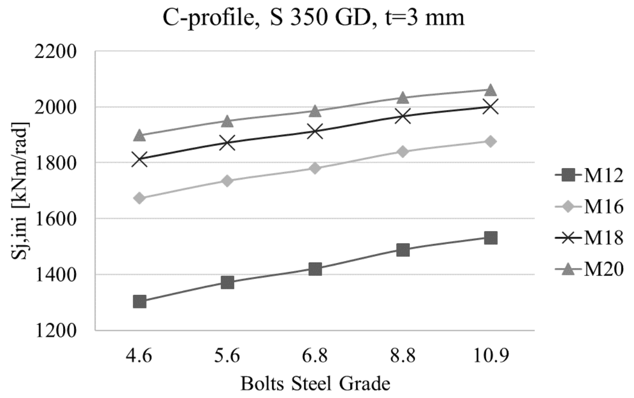

Figure 7. The initial rotational stiffness $S_{j,ini}$ of the DCBC joint depending on the bolt's diameter and steel grade.

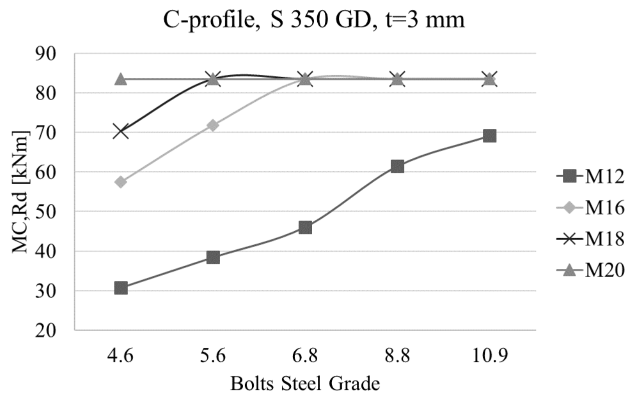

Figure 8. The design moment resistance $M_{C,Rd}$ of the DCBC joint depending on the bolt's diameter and steel grade.

The results of the component method when the C profile was 3 mm thick, having S 350 GD steel grade and M20 10.9 grade bolts was evaluated, yielding an initial rotational stiffness of $S_{j,ini}= 2062.02\ kNm/rad$, and a design moment resistance of $M_{C,Rd}=83.44\ kNm$. By comparing these results to the ones using the experimental data, we observed a difference of 9.55% between the initial rotational stiffness values, and 30.97 % between the design moment resistances.

4.2 *Parametric study of the HBBC joint*

Figure 9 presents the initial rotational stiffness $S_{j,ini}$ and Figure 10 presents the design moment resistance $M_{C,Rd}$ of the HBBC joint depending on the C profile's thickness and on the steel's yielding strength f_y. Similarly with the previous case, in Figure 9 and Figure 10 one can observe that as the thickness of the C profile and the yielding strength f_y increase, the initial rotational stiffness $S_{j,ini}$ and the design moment resistance $M_{C,Rd}$ also increase. However, the initial rotational stiffness of the HBBC joint has lower values than in the case of DCBC joint, as seen in Figure 11. The holes for hollo bolts drilled in the SHS column may contribute to the weakening of the joint's stiffness.

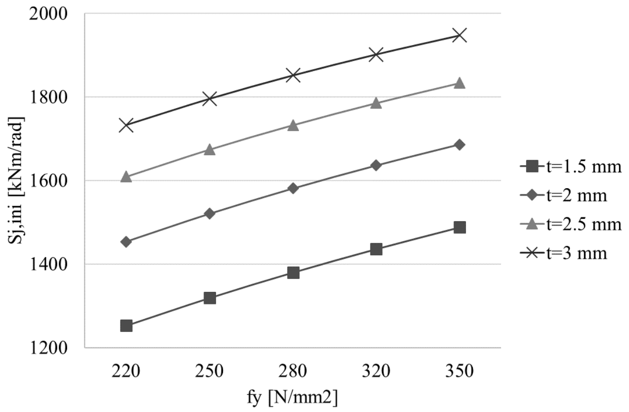

Figure 9. The initial rotational stiffness $S_{j,ini}$ of the HBBC joint depending on the C profile's thickness and on the steel's yielding strength f_y.

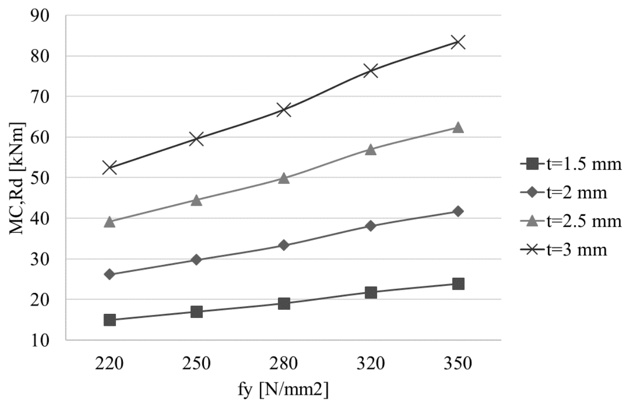

Figure 10. The resistant moment $M_{C,Rd}$ of the HBBC joint depending on the C profile's thickness and on the steel's yielding strength f_y.

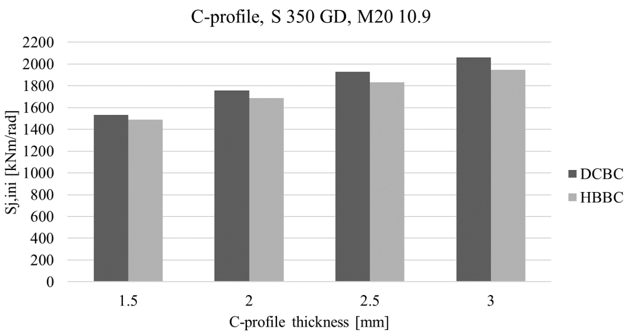

Figure 11. The differences between the initial rotational stiffness $S_{j,ini}$ of the DCBC and, respectively of the HBBC for different cold-formed profile thicknesses.

Figure 12 presents the initial rotational stiffness $S_{j,ini}$, while Figure 13 shows the design moment resistance $M_{C,Rd}$ of the HBBC joint depending on the bolt diameter and steel grade, if the C profile's steel grade is S 350 GD and the thickness is *t=3 mm*. As in the previous case, in Figure 12 and Figure 13 one can remark that as the diameter and steel grade of the bolt increase, the initial rotational stiffness and the design moment resistance also increase. Unlike the DCBC joint, the initial rotational stiffness of the HBBC joint has lower values due to the holes drilled in the SHS column for the hollo bolts, which decrease the overall stiffness of the joint. Similar with Figure 8 (the case of DCBC), in Figure 13 the design moment resistance is constant for the M20 bolts, regardless of the bolt steel grade meaning that the failure of the joint is produced by the cold-formed profiles. For the M12 bolts, the failure of the joint is produced by the bolts, regardless of their steel grade. In the case of M16 and M18 bolts, starting from the grade 4.6, the failure of the joint occurs in the bolts. As the bolt grade of M16 and M18 bolts increases, the failure of the joint occurs in the cold-formed profiles.

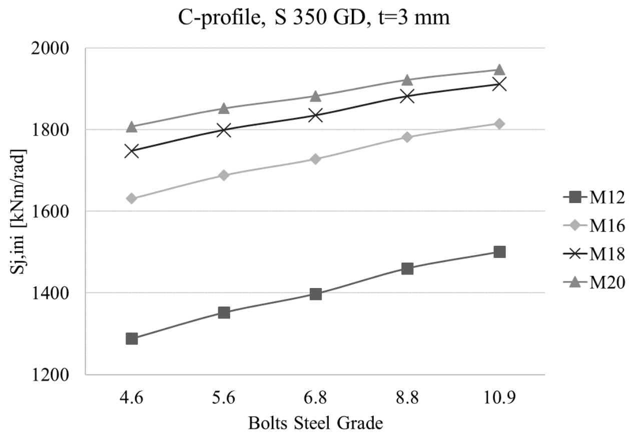

Figure 12. The initial rotational stiffness $S_{j,ini}$ of the HBBC joint depending on the bolt's diameter and steel grade.

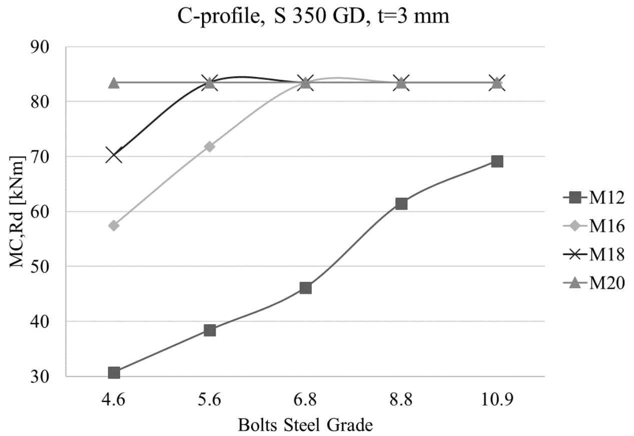

Figure 13. The design moment resistance $M_{C,Rd}$ of the HBBC joint depending on the bolt's diameter and steel grade.

As in the case of DCBC joint, for the comparison with the results determined from component method with mechanical properties from experimental data, the C profile with S 350 GD steel grade and thickness *t=3 mm* and M20 10.9 grade bolts were used. In the case of HBBC joint, the initial rotational stiffness is $S_{j,ini}$*=1946.99 kNm/rad*, while the design moment resistance is $M_{C,Rd}$*=83.44 kNm*, same as for DCBC joint. The design moment resistance has the same value for both DCBC and HBBC joint configurations, because according to Nagy et al (2018b), the failure of the joint occurs in the C profile beam. The difference between the results is: (i) in case of the initial rotational stiffness, the difference is *8.46%* and (ii) in case of the design moment resistance is *30.97%*.

5 CONCLUSIONS

According to the parametric study performed with component method for the DCBC and, respectively HBBC joint, the following observations can be noted. Firstly, from the graphs presented in Figures 5, 6, 9 and 10 one can remark that the values of the initial rotational stiffness $S_{j,ini}$ and of the design moment resistance $M_{C,Rd}$ increase as the C profile's thickness and its steel grade increase. Also, according to Figures 7, 8, 12 and 13, the values of the initial rotational stiffness and of the design moment resistance increase as the diameter and steel grade of the bolts increase. However, the initial rotational stiffness $S_{j,ini}$ of the HBBC joint has smaller values than in the case of DBBC joint (see Figure 11) due to the fact that the holes for hollo bolts drilled in the SHS column decrease the overall stiffness of the joint.

Secondly, from Figures 8 and 13, one can observe that the design moment resistance $M_{C,Rd}$ has constant values for the M20 bolts regardless of their steel grade. This means that if M20 bolts are used for the joint, the failure will be caused by the cold-formed profiles no matter what kind of bolt grade is used. If M12 bolts are used for the joints, then the failure of the joint will be caused by the bolts regardless of their steel grade.

Finally, the differences between the values of the initial rotational stiffness $S_{j,ini}$ determined from component method with mechanical properties from experimental data and the ones determined from component method with mechanical properties from Eurocode 3 were around 9% for both joint configurations. Also, the differences between the values of the design moment resistance $M_{C,Rd}$ determined from component method with mechanical properties from experimental data and the ones determined from component method with mechanical properties from Eurocode 3 were around 30% for both joint configurations. The design moment resistance $M_{C,Rd}$ has the same value for both DCBC and HBBC joints, because the failure of the joint is produced by the back-to-back C profiles beam, as observed by Nagy et al. (2018b).

6 REFERENCES

Chung, K. F., Lau L. 1999. Experimental investigation on bolted moment connections among cold formed steel members. *Engineering Structures* 21(1999): 898-911.

Dundu, M., Kemp A. R. 2006. Strength requirements of single cold-formed channels connected back-to-back. *Journal of Constructional Steel Research* 62(2006): 250-261.

European Committee for Standardization (ed.) 2003. *Eurocode 3: Design of steel structures: Part 1.8: Design of joints*. EN 1993-1-8.

European Committee for Standardization (ed.) 2004. *Continuously hot-dip coated strip and sheet of structural steel – Technical delivery conditions*. EN 10326.

Ho, H. C., Chung K. F. 2006. Structural behavior of lapped cold-formed steel Z sections with generic bolted configurations. *Thin-Walled Structures* 44(2006): 466-480.

Jaspart, J.P. 2000. General report: sessions on connections. *Journal of Constructional Steel Research* 55(2000): 69-89.

Komara, I., Wahyuni, E., Suprobo, P. 2017. A Study on Cold-Formed Steel Frame Connection: A review. *The Journal for Technology and Science* 28(3): 83-89.

Lee, Y. H., Tan, C. S., Mohamad, S., Tahir, S., Shek, P. N. 2014. Review on Cold-Formed Steel Connections. *The Scientific World Journal*: 1-11.

Lim, J. B. P., Nethercot D. A. 2003. Ultimate strength of bolted moment-connections between cold-formed steel members. *Thin-Walled Structures* 41(2003): 1019-1039.

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(2004): 85-107.

Nagy, Zs., Ballok, R., Dező, A. 2018a. FEM Investigation of Cold-Formed Joints for Multi-Storey Steel Frames. In EDlearing (ed.). *17th edition International Technical-Scientific Conference. Modern Technologies for the 3rd Millennium*, Oradea, Romania: 329-334.

Nagy, Zs., Dező, A, Ballok, R. 2018b. Application of the component method for cold formed joints assisted by finite element analysis. *Eighth International Conference on THIN-WALLED STRUCTURES – ICTWS 2018; Lisbon, Portugal, 24 – 27 July 2018.*

Nagy, Zs., Gîlia, L., Neagu, C. 2017. Experimental Investigation of Cold-Formed Joints for Multi-Storey Steel Framed Structures. *Proceedings of the Romanian Academy* 18(3): 256-264.

Nagy, Zs., Stratan, A., Dubina, D. 2006. Application of component method for bolted cold-formed steel joints. In CRC Press/Balkema, ISBN:10:0-415-40817-2 (ed.), *Proceedings of the international conference in metal structures: steel – a new and traditional material for building*, Poiana Brasov, Romania: 207-210.

Öztürk F., Pul, S. 2015. Experimental and numerical study on a full-scale apex connection of cold-formed steel portal frames. *Thin-Walled Structures* 94(2015): 79-88.

Wong, M. F., Chung K. F. 2002. Structural behavior of bolted moment connections in cold-formed steel beam-column sub-frames. *Journal of Constructional Steel Research* 58(2002): 253-274.

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(2005): 1332-1352.