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ANALYTICAL AND EXPERIMENTAL INVESTIGATION OF COLD-FORMED STEEL BEAM-TO-COLUMN BOLTED GUSSET-PLATE JOINTS

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Abstract. Nowadays, cold-formed constructions are being used more frequently on construction sites because of the good strength-to-cost ratio. However, insufficient studies are published examining the properties of these constructions. This paper investigates the behaviour of bolted gusset-plate joint since it is one of the easiest ways to connect a beam to a column. The paper presents analytical calculations using the component method and experimental test results. The joint was investigated using the mechanical model of three springs. The mechanical model for the calculation of bolt group stiffness was created according to Eurocode 3 Part 1–8 Stiffness formulations (EN 1993-1-8 2005) (originally, it is recommended for elements that are 4 mm and thicker). The technique for the calculation of the stiffness of a gusset plate is presented. The strength of the joint was calculated using the technique introduced in Eurocode 3 (EN 1993-1-8 2005; EN 1993-1-1 2005; EN 1993-1-3 2006; EN 1993-1-5 2006).

Keywords: semi-rigid joints, component method, cold-formed steel joints.

Introduction

Cold-formed thin-walled sections are widely used as bearing structures. In most cases, thin-walled sections are used as purlins, steel trusses and for lightweight frames. There is a wide variety of cold-formed sections (such as Z-sections, C-sections, sigma-sections, omega-sections, etc.). The joints should be simple, fast and easy installed. One of the most practical ways to connect joint elements is using gusset plates and bolts.

The joints of steel structures are usually divided into rigid or hinged. In the last few decades, the concept of semi-rigid joints became more popular. The researches of steel frames with semi-rigid joints showed that the stiffness of joints had a significant influence on the behaviour of frame elements (Del Savio *et al.* 2009; Díaz *et al.* 2011; Daniūnas, Urbonas 2008, 2010). The real behaviour of the framework allows designing safer structures and reaching the economic benefit (Daniūnas, Urbonas 2013; Ćirović *et al.* 2014; Fayyadh, Razak 2014; Kala 2015; Misiūnaitė *et al.* 2012; Talaslioglu 2015).

In recent years, researches on thin-walled sections focused on the experimental investigation of beam-to-beam joints and beam elements (D'Aniello *et al.* 2014). Previous studies present a wide variety of experimental tests and numerical simulation results; however, most of them lack analytical calculation of joint stiffness. Investigators who performed laboratory tests on beam-to-beam over-lapped (Ho, Chung 2004, 2006a, 2006b; Chung, Ho 2005; Chung *et al.* 2005; Dubina, Ungureanu 2010), sleeve (Gutierrez *et al.* 2011; Yang, Liu 2012) and apex (Lim, Nethercot 2003, 2004; Elkersh 2010; Pernes, Nagy 2012; Öztürk, Pul 2015) cold-formed joints and analysed data agreed that the behaviour of this joints is semi-rigid.

In most cases, beam-to-column joints were also investigated only using experimental tests and not analytical methods. Wong and Chung (2002) executed beam-to-column sub-frame tests with different configurations of the bolted gusset-plate joints. The authors investigated the influence of gusset-plate thickness, the chamfer presence and the distance between bolts on the strength and stiffness of joints. Lateral restraints for columns and beams were used. It was found that the geometry of a gusset plate has a large influence on the behaviour of the joint. In their study, Yu *et al.* (2005) presented semi-empirical design method to calculate rotation stiffness of bolted gusset-plate joint. This method is not very easy to use because the designer needs to predict bearing deformation of the section web around the hole of the bolt. Sabbagh *et al.* (2011, 2012, 2013) executed the tests on beam-to-column joints with gusset-plate joints under cyclic loads to consider different stiffeners of the beams. Lateral restraints for a beam and a column were used and the optimum configuration of stiffeners was proposed. Bučmys *et al.* (2014) investigated the influence of lateral restraints on the stiffness and strength of the joint. In the

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paper by Bučmys and Šaučiuvėnas (2013), the simulation data of the behaviour of beam-to-column joints with a gusset plate was presented. The authors examined the behaviour of the joint with a gusset plate to take into account the variety of thicknesses and the geometry of the gusset plate. All the researchers mentioned in this paragraph performed their tests additionally using lateral restraints for their specimens because they make it easier to investigate the behaviour of the joints. The researchers who performed experimental tests on this type of joints agreed that in most cases, their behaviour is semi-rigid. It shows that analytical formulation of stiffness calculation is necessary because joint stiffness has an impact on the forces and deflections of the structure.

The component method to calculate the characteristics of joints is applied in Eurocode 3 (EN 1993-1-8 2005). Stiffness calculation is suitable for elements of 4 mm and thicker. The goal of this paper was to test if Eurocode 3 (EN 1993-1-8 2005) is suitable for the stiffness calculation of thin-walled cold-formed joints with element thickness of less than 4 mm. The results of the investigation show that presented analytical formulations are suitable for the stiffness calculation of a bolt group. Moreover, in the Eurocode 3 (EN 1993-1-8 2005), there is no information on how to calculate the gusset-plate stiffness. As a result, this paper presents analytical formula of gusset-plate stiffness calculation that is in good agreement with experimental data.

1. The model of rotational springs for stiffness and strength calculation of a bolted gusset-plate joint

1.1. Bolted gusset-plate joint

The joint under analysis consists of double back-to-back lipped sections, T-section gusset plate and bolts (Fig. 1). The failure modes of the joint:

- failure of bolts in shear;
- beam and column web, gusset plate in bearing;
- failure of the gusset plate in bending and shear;
- local buckling in the beam and column sections.

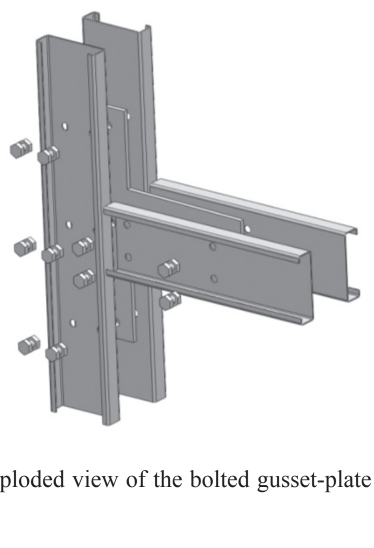


Fig. 1. The exploded view of the bolted gusset-plate joint

1.2. Component method

General assumption of the component method for steel joints is to decompose the joint into a limited number of basic components. The first stage step of the component method is to identify all basic components that are active in shear, bearing, compression and tension zones. The components are listed in Eurocode 3 (EN 1993-1-8 2005). The next stage step is to determine their local design resistances $F_{i,Rd}$ and stiffness coefficients k_i in order to calculate the global parameters of joint characteristic: the design moment resistance $F_{j,Rd}$ and the initial stiffness $S_{j,ini}$, respectively. All necessary formulae describing the design resistances and stiffness coefficients of the active components are provided in Eurocode 3 (EN 1993-1-8 2005). The main disadvantage is that Eurocode 3 (EN 1993-1-8 2005) describes only a limited number of components for basic joints. As a result, there are no design instructions on how to solve a bolted gusset-plate joint.

1.3. Stiffness calculation of joint springs

The model proposed for standard semi-rigid beam-to-column joints is adapted for cold-formed bolted gusset-plate joints. The represented model uses the component method and separates the joint into three separate rotational springs.

The model of the joint is depicted in Figure 2. The joint consists of three different rotational springs: beam bolt group, column bolt group and gusset plate.

The bolt groups of the joint that is under load F are affected by bending moments M_1 of the beam bolt group and by bending moment M_2 of the column bolt group. As a result, the bolt groups and the gusset plate should be investigated separately because the bolt groups are affected by different lever arms (Fig. 3):

- L_1 for beam bolt group and gusset plate;
- L_2 for column bolt group and gusset plate.

According to Eurocode 3 Part 1–8 (EN 1993-1-8 2005), the stiffness calculation of specimen both column and beam bolt groups consists of the following components:

- k_{12a} section web in bearing (overall 2 sections in bolt group);
- k_{12b} gusset plate in bearing;
- k_{11} bolts in shear (overall 2 shear planes in one bolt).

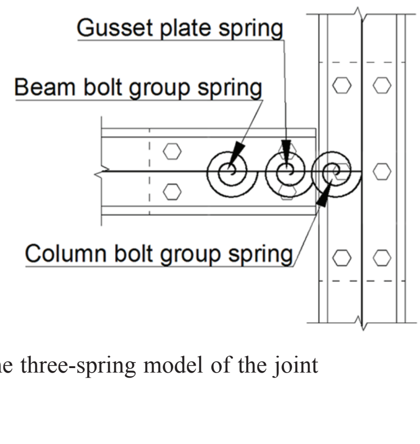


Fig. 2. The three-spring model of the joint

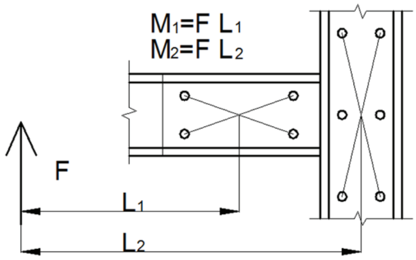


Fig. 3. Lever arm of bending moments

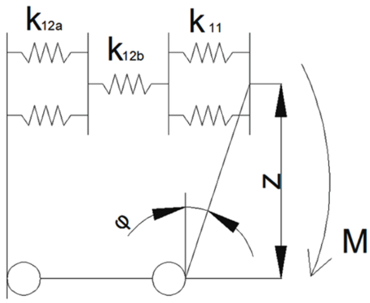


Fig. 4. The mechanical model of a bolt group spring

The mechanical model of the bolt group is given in Figure 4. The stiffness of components is taken according to Eurocode 3 (EN 1993-1-8 2005).

In Eurocode 3 Part 1–8 (EN 1993-1-8 2005), there is no suggestion for the stiffness calculation of a gusset plate. In this paper, the stiffness of a gusset plate was calculated according to this formula (Fig. 5):

$$S_{gp,ini} = \frac{M}{\varphi} = \frac{M_2}{\dfrac{2M_1 L_a + VL_a^2}{2EI_1} + \dfrac{3L_b^2 M_2}{2L_c EI_2}}, \qquad (1)$$

where: L_a – distance from the rotation centre of the beam bolt group to the edge of gusset plate; L_b – distance from the outer bolt centre of the column bolt group to the edge of gusset plate; L_c – distance between outer bolts of the column bolt group; I_1 – the moment of inertia of the beam outstand element; I_2 – the moment of inertia of the column outstand element; V – shear force due to the beam load.

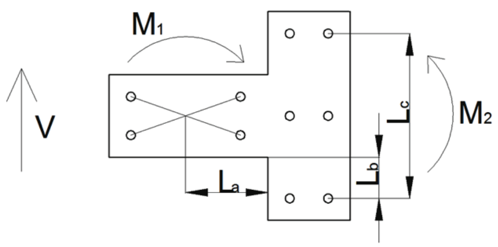


Fig. 5. The scheme for the rotation calculation of the gusset plate

1.4. Strength calculation of joint components

It is convenient to separate joint strength calculation in to five different parts:
- the bolt group of beam (1);
- the bolt group of column (2);
- the gusset plate (3);
- beam (4);
- column (5).

The strength of the basic components was calculated according to Eurocode 3. The bolt group of beam (1) and column (2) should be considered: column and beam web in bearing $F_{v,Rd}$, gusset plate in bearing $F_{b,Rd}$ and bolts in shear $F_{n,Rd}$. The gusset plate (3) should be considered: in bending $M_{c,Rd}$ and shear $V_{pl,Rd}$. The beam (4) and column (5) should be considered in bending $M_{c,Rd}$ and shear $V_{b,Rd}$. It should be mentioned that W_{pl} was considered in the calculation of the gusset plate resistance, assuming that lateral restraints would prevent the gusset plate from buckling.

2. Experimental test

2.1. Test specimen

Three specimens were investigated experimentally (Fig. 6). Gusset plates and cold-formed C-sections were made of steel grades S355 and S350GD+Z275, respectively. The yield and the ultimate strength of both steel grades were calculated by way of the coupon tests. As a result, the values were received for cold formed sections f_y = 380 MPa and f_u = 484 MPa, for gusset plate f_y = 442 MPa and f_u = 570 MPa, respectively. The specimens were connected using bolts with 8.8 classes. The diameter of bolt holes was 1 mm higher than the bolt diameter. The spacing between bolts connecting the beam channel and the column channel to the gusset plate were 200 mm and 150 mm, respectively. The specimens differed by bolt diameter and C-section thickness:
- the first specimen (M12 C15015 T12) was made of 12 mm diameter bolts, the C-section of 1.5 mm in

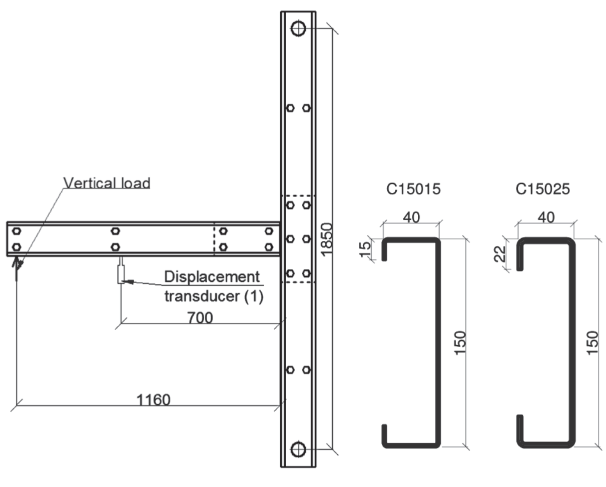


Fig. 6. Geometrical properties of the specimens

thickness and the gusset plate of 12 mm in thickness;
- the second specimen (M12 C15025 T12) s made of 12 mm diameter bolts, the C-section of 2.5 mm in thickness and the gusset plate of 12 mm in thickness;
- the third specimen (M16 C15015 T12) was made of 16 mm diameter bolts, the C-section of 1.5 mm in thickness and the gusset plate of 12 mm in thickness.

The load was transferred by the jackscrew to the end of the beam. Pinned supports were used at both endings of the columns. All the specimens were with lateral restraints to the beam and to the column (Fig. 7a). Lateral restraints prevented from lateral torsional buckling of the beam and column.

Two groups of transducers were added to measure two types of deflection:
- the 1st transducer for the beam deflection (Fig. 6). The data was used to compare the deflection of all three specimens;
- 2–5 transducers to measure the local deflection (Fig. 7b). The data was used to calculate *M*-φ curves of bolt groups and the gusset plate.

The deflection values that were measured by transducers of the beam bolt group consist of bending and shear deformations. The rotation of beam bolt group was calculated eliminating the deflection by shear force according this formula:

$$\begin{cases} \Delta_2 = \Delta_S + \Delta_{b,v} \\ \Delta_3 = \Delta_{b,v} - \Delta_S \end{cases} \rightarrow \begin{array}{l} \Delta_S = \Delta_{b,v} - \Delta_3 \\ \Delta_{b,v} = \dfrac{\Delta_2 + \Delta_3}{2} \end{array} \rightarrow \Delta_b = \frac{\Delta_{b,v}}{\cos\alpha}, \quad (2)$$

where: Δ_2 – deflection value measured by the second transducer; Δ_3 – deflection value measured by the third transducer; Δ_S – deformation due to shear; Δ_b – deflection due to the bending moment; $\Delta_{b,v}$ – vertical component of deflection due to the bending moment; $\Delta_{b,h}$ – horizontal component of deflection due to the bending moment.

The deformation scheme of the beam bolt group is presented in Figure 8.

The rotation of the column bolt group was calculated according this formula, disregarding the middle bolts:

$$\begin{cases} \Delta_4 = \Delta_{b,h} \\ \Delta_4 = \Delta_{b,h} \end{cases} \rightarrow \Delta_b = \frac{\Delta_{b,h}}{\sin\beta}, \quad (3)$$

where: Δ_4 – deflection value measured by the fourth transducer; Δ_S – deformation due to shear; Δ_b – deflection due to the bending moment; $\Delta_{b,v}$ – vertical component of deflection due to the bending moment; $\Delta_{b,h}$ – horizontal component of deflection due to the bending moment.

The deformation scheme of the column bolt group is presented in Figure 9.

a)

b)

Fig. 7. Specimen of the beam and column: a) lateral restraints; b) transducers for measurement of bolt groups and gusset-plate rotation

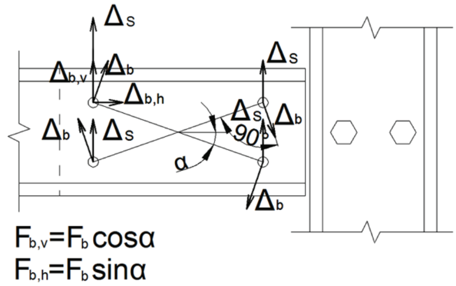


Fig. 8. Deformation scheme of the beam bolt group

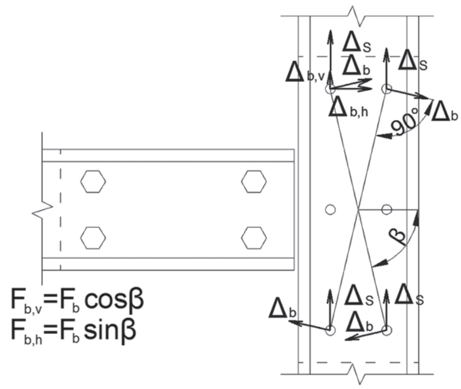
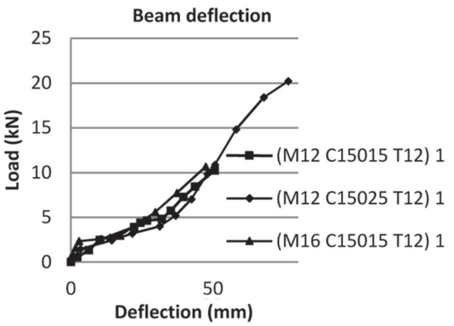
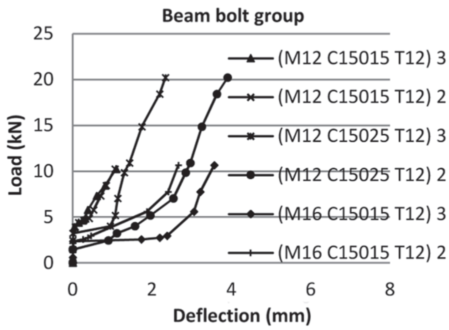


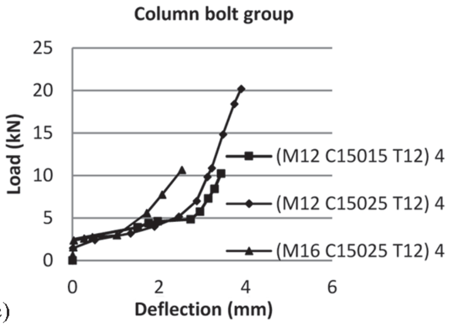
Fig. 9. Deformation scheme of the column bolt group



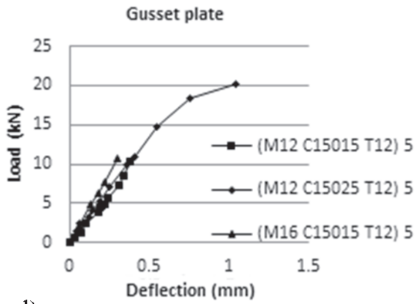
a)



b)



c)



d)

Fig. 10. The data of the transducers: a) the first transducer; b) the second and the third transducers; c) the fourth transducer; d) the fifth transducer

2.2. Experimental results

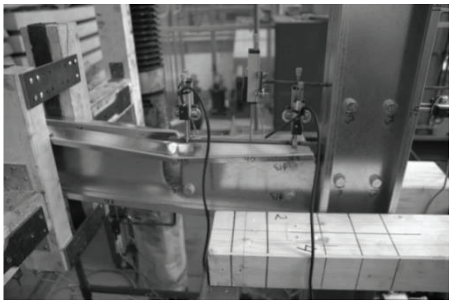
The experimental strength capacity of the first specimen (M12 C15015 T12) was 10.22 kN (Fig. 10a, b, c, d) and the failure mode was the local buckling in the beam (Fig. 11a). The experimental strength capacity of the second specimen (M12 C15025 T12) was 20.2 kN (Fig. 10a, b, c, d) and the failure mode was the bolt collapse due to shear (Fig. 11b). The investigation after the test showed that bearing deformations around the holes of bolts and local buckling deformations in the beam occurred. The load value when these deformations occurred is not known. The strength capacity of the third test (M16 C15015 T12) was 10.7 kN (Fig. 10a, b, c, d) with failure mode local buckling at the beam (Fig. 11c).

Experimental tests showed deformations of the joint (Fig. 10a, b, c, d). According to the data, there are three phases of the behaviour of the joints:

- Phase 1. Deformation is linear because of friction forces between elements;
- Phase 2. Stiffness decreases because friction forces overcome and slipping occurs (it mostly depends on the hole–bolt ratio);
- Phase 3. Stiffness increases since all bolts are in contact and deformation continues until construction failure.

The 2nd–4th transducers that measured bolt group deflection showed that deformations were elastic (Fig. 10 b, c). Only the 5th transducer of the second specimen (M12 C15025 T12) showed that the gusset plate reached plastic deformations (Fig. 10d).

a)



b)

c)

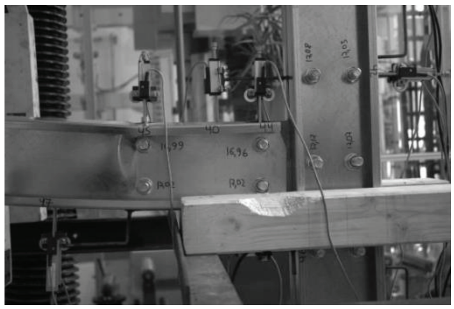


Fig. 11. The failure of specimens: a) M12 C15015 T12; b) M12 C15025 T12; c) M16 C15015 T12

3. Analytical calculation of stiffness and strength of the joints

3.1. Stiffness of the joints

In this chapter, experimentally and analytically calculated *M*-φ curves of three different springs are presented. Based on data from laboratory tests, experimental *M*-φ curves were calculated using Figure 8 and Figure 9 schemes, and analytical *M*-φ curves were calculated using the model described in Part 1.3 of the paper. As mentioned in Part 2.1 of this paper, steel strength properties of the gusset plate and cold-formed sections for analytical calculations were received from coupon tests, and characteristic strength values (f_y = 640 MPa and f_u = 800 MPa, respectively) were used for steel properties of bolts. The stiffness was calculated as $S_j = S_{j,ini} / 3$, then $M_{j,Ed} > 2/3 M_{j,Rd}$ according to Eurocode 3 (EN 1993-1-8 2005) design rules.

Experimental and analytical data of *M*-φ curves of the beam bolt group are presented in Figure 12a, b, c. It is hard to evaluate precisely the difference between Eurocode 3 (EN 1993-1-8 2005) model and data from experimental tests because experimental rotation values consist both of rotation and slipping of bolts. However, the graphs demonstrate that both data from laboratory tests and Eurocode 3 (EN 1993-1-8 2005) have a satisfactory agreement.

Experimental and analytical data of *M*-φ curves of the column bolt group are presented in Figure 13a, b, c. It is also hard to evaluate the difference between these two methods; however, the data from laboratory tests and Eurocode 3 (EN 1993-1-8 2005) show a good agreement.

As mentioned before, Eurocode 3 Part 1–8 (EN 1993-1-8 2005) is designed for elements with a thickness of 4 mm and thicker. The results of the laboratory tests and analytical calculation demonstrate that the stiffness could be also calculated for thin-walled elements with a thickness less than 4 mm; however, more work should be done to substantiate this conclusion.

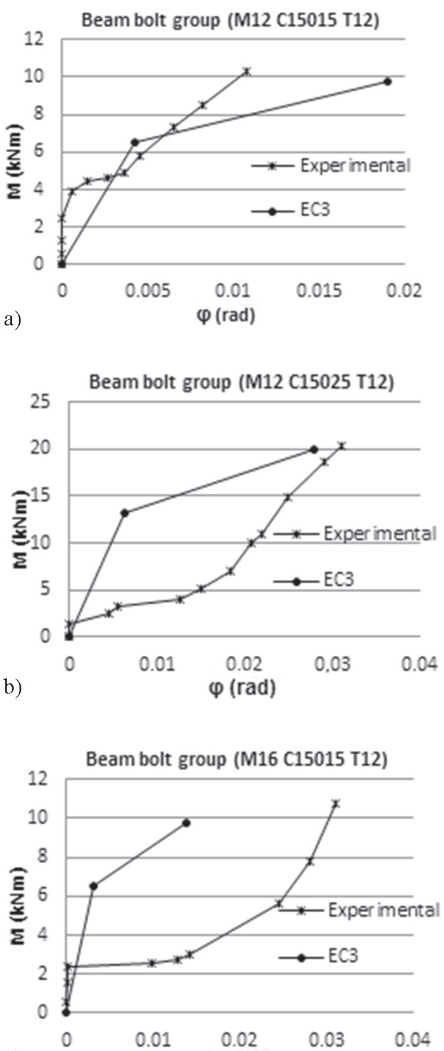


Fig. 12. *M*-φ curves of the beam bolt group of the specimens: a) M12 C15015 T12; b) M12 C15015 T12; c) M16 C15015 T12

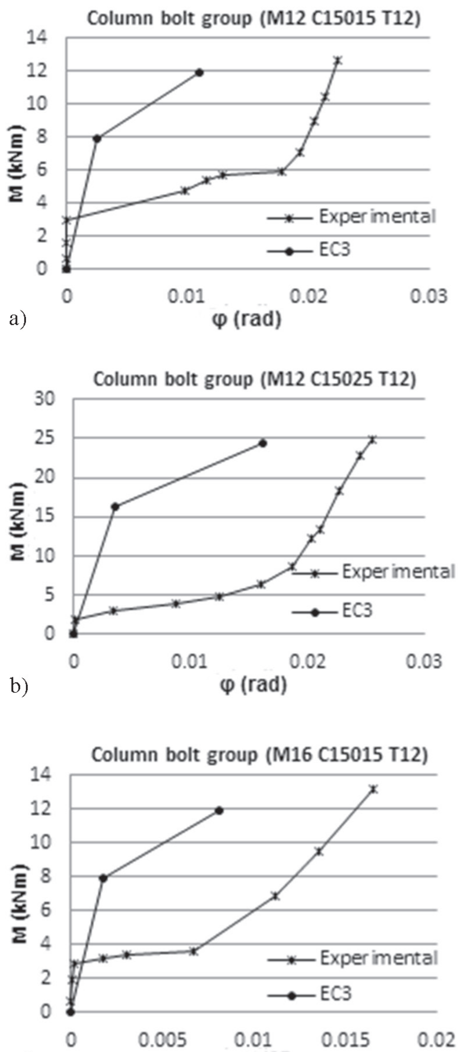


Fig. 13. *M*-φ curves of the column bolt group of the specimens: a) M12 C15015 T12; b) M12 C15015 T12; c) M16 C15015 T12

The data from experiments and analytical calculations on *M*-φ curves of the gusset plate are presented in Figures 14a, b, c. Both laboratory test data and the presented method showed good agreement. More work is needed to test that in different a gusset-plate geometry combination as thickness and length.

Figures 15a, b, c give the rotation of the column bolt group, the beam bolt group and the gusset plate according analytical calculations of each specimen. The bending moment was calculated taking the lever arm from the load to the centre of the column in order to compare the stiffness of springs. The biggest impact is of the beam bolt group. The rotation of the column bolt group and the gusset plate is similar. As a result, the presented data suggests that the gusset-plate rotation should be evaluated.

3.2. Strength of the joints

Here, the strength capacity calculated analytically (according to the technique described in Part 1.4 of this paper) and experimentally measured values are presented. The same steel strength properties for component strength calculations were taken as for the stiffness calculation.

According to Eurocode 3 (EN 1993-1-8 2005; EN 1993-1-1 2005; EN 1993-1-3 2006; EN 1993-1-5 2006) calculations, the specimen M12 C15015 T12 should collapse when the load reaches the value of 9.68 kN (Fig. 16a) with the failure mode of local buckling in the beam. It showed a 5% reserve (Fig. 16d) compared with the experimental data and the same failure mode. The second specimen M12 C15025 T12 should collapse when the load reaches the value of 19.74 kN (Fig. 16b), and the failure mode should be local buckling in the beam. Compared with the experimental test, it showed a 3% reserve and a different failure mode (analytical – beam section in bearing, experimental – bolt collapse due to shear). The third specimen M16 C15015 T12 should collapse when the load reaches the value of 9.68 kN value (Fig. 16c) with the same failure mode as the first specimen. It showed a 10% reserve.

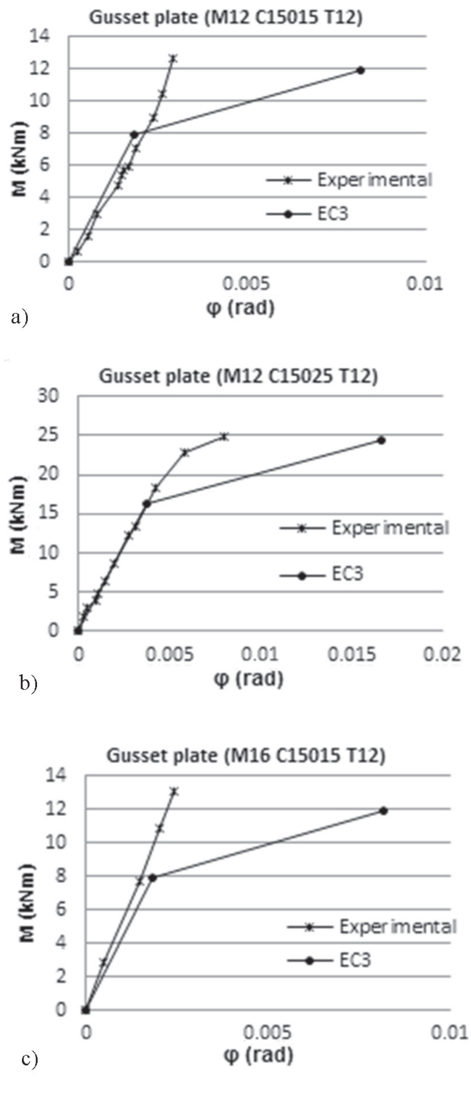


Fig. 14. *M*-φ curves of the gusset plate of the specimens: a) M12 C15015 T12; b) M12 C15015 T12; c) M16 C15015 T12

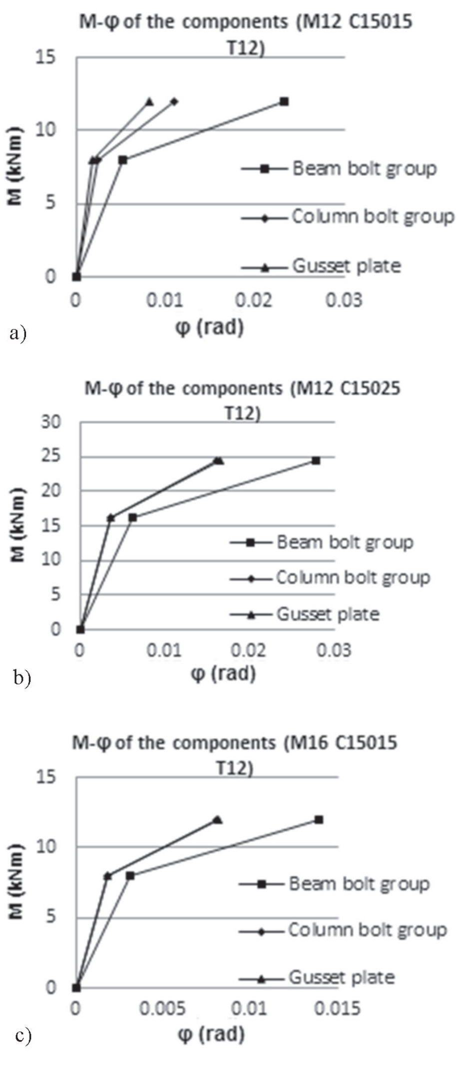


Fig. 15. *M*-φ curves of the springs of the specimens: a) M12 C15015 T12; b) M12 C15015 T12; c) M16 C15015 T12

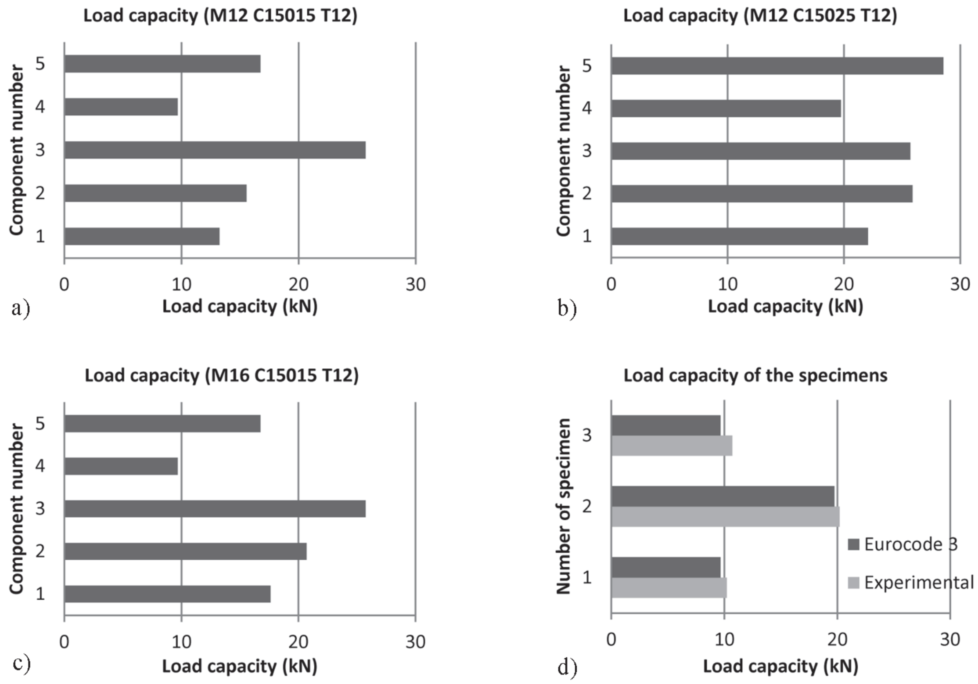


Fig. 16. The strength capacity of the specimen components and specimens: a) specimen M12 C15015 T12; b) specimen M12 C15015 T12; c) specimen M16 C15015 T12; d) strength capacity of the specimens

Conclusions

The analytical analysis and experimental results of the cold-formed steel beam-to-column bolted gusset-plate joints allow making the following conclusions:

1. The mechanical model of a bolt group that was created using Eurocode 3 Part 1–8 stiffness (EN 1993-1-8 2005) formulations (originally suitable for standard joints with elements that are 4 mm and thicker) showed satisfactory results compared with experimental test results.
2. The formula for the stiffness calculation of the gusset plate was presented and showed good agreement with laboratory test data.
3. The three-spring model for the stiffness calculation was suggested.
4. The strength capacity shows similar values in analytical calculations and experimental results. The safety margin was from 3% to 10%.
5. Calculating the strength of a gusset plate in bending, we suggest taking into account plastic section modulus because this component is between the cold-formed sections that prevent form buckling. Laboratory tests confirmed that plastic deformations occurred in the gusset plate.

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