

A Parametric Study of Cold-Formed Steel Bolted Joints in Pitch-Roof Portal Frames

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Abstract

Based on a large experimental program on eaves and knee bolted joints [1] used in pitched roof cold-formed steel portal frames of back-to-back lipped channel sections [2], numerical analysis of the tested joints was performed, obtaining calibrated finite element models. Using the developed finite element models, a parametric study has been carried out, consisting of eight ridge joint configurations with flange bolts and six ridge joint configurations with web bolts only. The experimental joint strengths and numerical results predicted by the parametric study are compared with the design strengths calculated using the principles of component method, proposed by current European specifications for steel joints.

Keywords: cold formed steel joints, back-to-back lipped channel sections, full scale tests, finite element analysis, parametric study, virtual testing.

1 Introduction

The global behaviour of cold-formed steel portal frames of bolted joints were studied experimentally by Lim [3], Dundu & Kemp [4], Kwon et al. [5] and Ahamed, Hazlan & Mahendran [6]. All these studies provided evidence of the crucial importance of joint performance on the global response of frames, which are semi-rigid and in almost all cases with partial strength [3].

An extensive experimental program on ridge and eaves joints, with three alternative joint configurations, using welded bracket elements and bolts installed either on webs only or both on webs and flanges was carried out at the “Politehnica” University of Timisoara. Detailed experimental results on joint behaviour are reported by Dubina et al. [1]. Based on experimental results, a calculation procedure using the component method of EN1993-1-8 [7] was adapted for cold-formed steel joints [8]. Joint stiffness and moment capacity, obtained using the component method, were used to develop a joint model for global structural analysis. In a

second phase, two full-scale tests on cold-formed pitched-roof portal frames with bolted joints were performed, with the primary objective to assess their performance under horizontal (seismic) loading.

Experimental tests emphasize the bearing work of bolts associated with elastic-plastic elongation of bolt-holes, proven to be far the most important component controlling the stiffness and capacity of such type of connections [8]. The contribution of other components, such as flanges in tension and compression due to bending action, and the web in shear due to transverse action is significantly lower.

The paper summarizes the results of the finite element analyses (FEA) done to calibrate FE models of the tested joints [9], and brings supplementary results using the parametric study on ridge joints. The comparison of the experimental and numerical results will be presented together with the ones obtained from the parametric study, for the evaluation of connections characteristics.

2 Summary of testing program on joints

To define realistic specimen configurations, a simple pitched roof portal frame was first designed with the following configuration: span 12 m; bay 5 m; eaves height 4 m and roof slope of 10° . This frame was subjected to loads common in the Romanian design practice. These loads were evaluated at 10 kN/m, as uniformly distributed load on the frame. The frame was analysed and designed according to EN 1993-1-3 [10]. The size of knee and ridge specimens and testing setup were chosen to obtain in the connected members a distribution of bending moment similar to the one observed in the designed structure.

Elements of the portal frame resulted back-to-back built-up sections made of Lindab C350/3.0 profiles (nominal yield strength $f_y=350 \text{ N/mm}^2$). Using these cross-section dimensions, three alternative joint configurations were designed (see Figure 1. and Figure 2.), using welded bracket elements (S235: $f_y=235 \text{ N/mm}^2$).

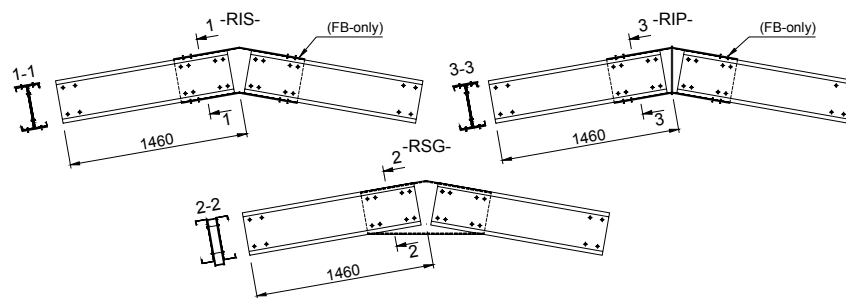


Figure 1: Configurations of ridge joints.

One group of specimens (KSG and RSG) used spaced built-up gussets. In this case, bolts were provided only on the web of the C350 profile. In the other cases, where two different details were used for the connecting bracket – i.e. welded I sections only (KIS and RIS), and welded I section with plate bisector (KIP and RIP), respectively - bolts were provided on the web only, or both on the web and the flanges. Joints where bolts were provided on the web and on flanges were marked by FB letters.

Monotonic and cyclic tests were performed for each specimen typology, all specimens being tested statically [1]. This paper will discuss only the results of the monotonic tests. The monotonic tests identified failure modes of the different joint typologies. All specimens had a failure due to local buckling of the cold-formed profiles; however two distinctive modes were identified for specimens with flange bolts and those without (see Figure 3. and Figure 4.).

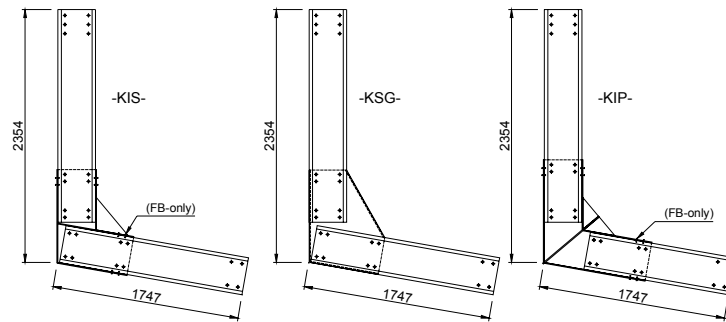


Figure 2: Configurations of knee joints

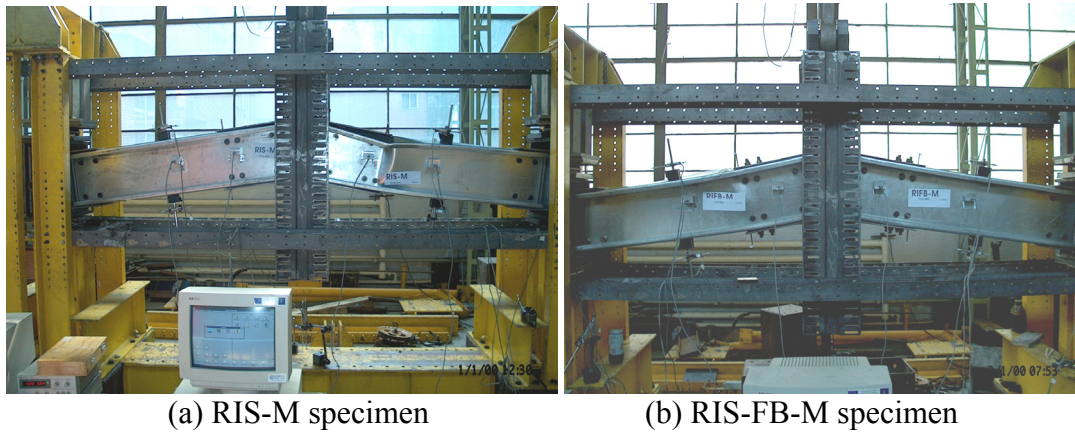


Figure 3: Failure of ridge specimens

Test on joints have shown their failure occurs always at the edge of lap between connecting bracket and cold-formed sections. In case of specimens with bolts on webs only, the failure starts early by local buckling of the web, caused by the high concentration of compression stresses around bolt holes, and subsequently is extended on the flanges, to form at the end a local plastic mechanism. Specimens of bolts installed both on the flanges and webs of connected members are nearly full resistant, but still remain semi-rigid. Comparative experimental curves for ridge and knee connections are presented in Figure 5. There are no significant differences among the specimens without flange bolts (RSG-M, RIP-M, and KSG-M, KIS-M). This could be explained by the higher stiffness and capacity of the connecting bolts compared to the other components of the joint. On the other hand, there is an important gain in load bearing capacity when bolts are installed also on the flanges, although this joint type is more difficult to fabricate (RIS-FB-M and KIS-FB-M).

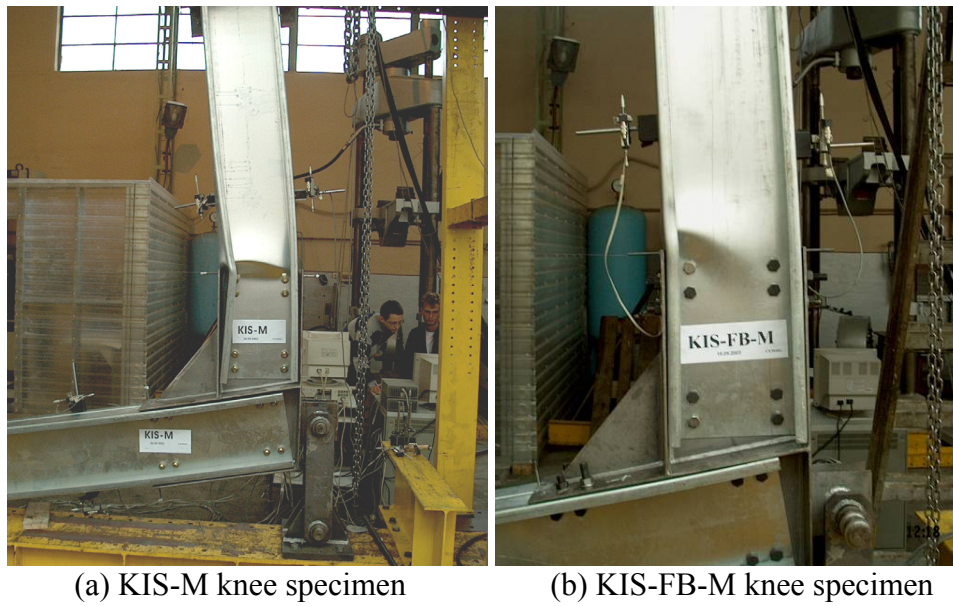


Figure 4: Failure of knee specimens

Obviously, the specimens with unbolted flanges that failed prematurely by web buckling due to stress concentration around the outer bolt rows, would be the weakest part of portal frames. Consequently, this joint typology is not recommended to be used in practice.

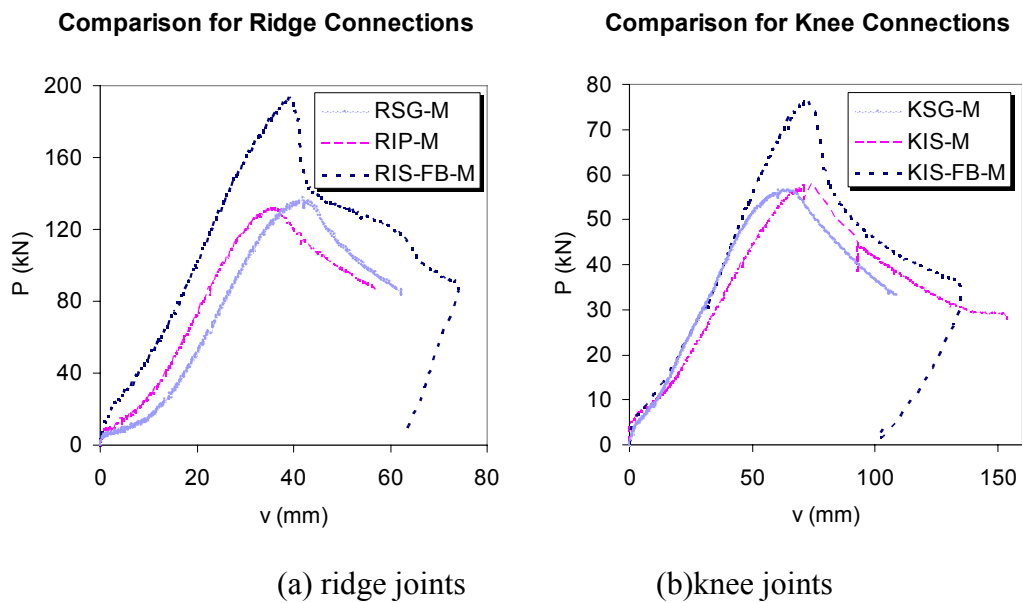


Figure 5: Comparative results from monotonic tests

3 Finite element analysis (FEA) of ridge and knee joints

3.1 FEA Calibration process

In order to study the structural behaviour of the joints (RIS-FB-M; KIS-FB-M; RIS-M; KIS-M), finite element models have been developed, they are shown in Figure 6 and Figure 7. The numerical models for the tested specimens were created using the finite element analysis software package ABAQUS/CAE v.6.10 [11].

Some features about the FE model: (1) finite element type: 8-noded standard quadratic, reduced integration, homogeneous shell element (S4R) to model the cold-formed members; (2) 3D solid elements (C3D4) to model the brackets at the beam-column joints; (3) 3D elements to model the bolts between the back-to-back cold-formed lipped channels and brackets; (4) the interaction between parts was considered as hard contact behaviour for normal direction and for tangential direction was chosen tangential friction coefficient of 0.3 as penalty; (5) pre-tensioning force to model the effect of bolt tightening; (6) different mesh refinements were studied in order to find the optimum number of elements from the point of view of ultimate force accuracy and analysis time. Finally, a mesh size of 8 mm $\times$ 8 mm was considered for C-profiles and 12mm $\times$ 12mm for the bracket providing enough accuracy; (7) load and end conditions were considered as in experimental tests; (8) the material behaviour used for numerical modeling was in accordance with the recorded curves from tensile tests (multi-linear isotropic model). The material properties for thin-walled cold-formed elements, determined from coupon tests, are: yield strength of 486N/mm², ultimate tensile strength 553N/mm², Young's modulus $E=210000\text{N/mm}^2$ and a measured thickness minus zinc coating of 2.90mm.

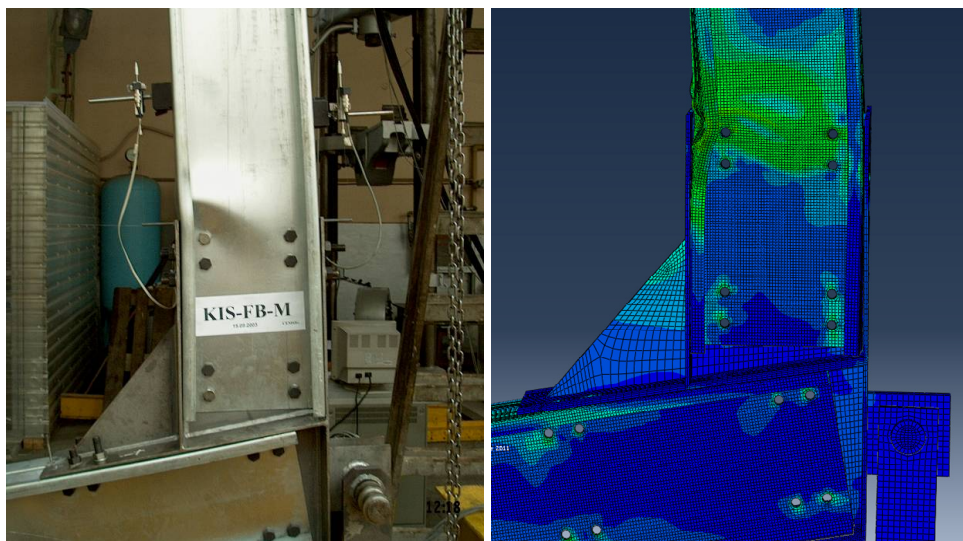


Figure 6: Knee (KIS-FB-M) experimental test vs. FEM results

Nonlinear analyses were performed, using dynamic explicit steps as quasi-static, choosing time parameter so that the inertial effects on the system to be insignificant (takes into account stiffness loss due to local buckling).

The calibrated FE models reproduce the same failure mechanism in case of knee specimens (KIS-FB-M, see Figure 6.) as obtained experimentally. In case of ridge specimens (RIS-FB-M, see Figure 7.), due to the symmetry of the model, the failure was distributed between the symmetric branches, obtaining two local failure mechanisms (see Figure 7.) instead of one obtained experimentally.

Due to this aspect, in case of ridge specimens, load introduction imperfection was considered, obtaining similar failure mechanism as experimentally. Figure 8 and 9 presents the force-displacement curves obtained both experimentally and numerically, for all types of joints. Good agreement can be observed between groups of curves.

It is interesting to mention that increasing the distance between the cold formed C profile flanges and the stiff bracket on the calibrated numerical model, there is no significant differences in results in terms of stiffness neither load carrying capacity of the joint.

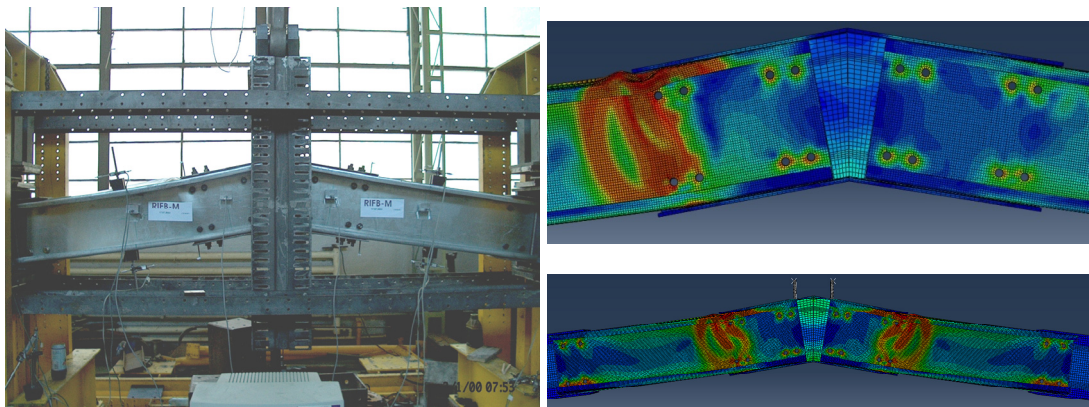


Figure 7: Ridge (RIS-FB-M) experimental test vs. FEM results

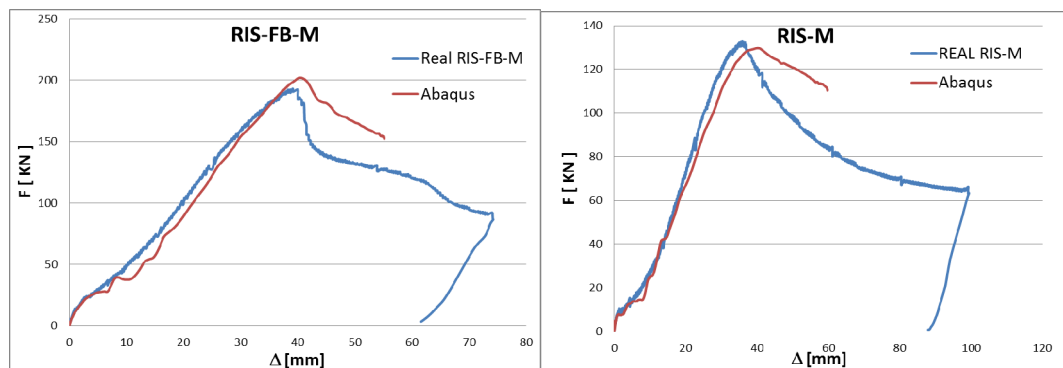


Figure 8: Ridge specimens: F- Δ experimental and numerical curves

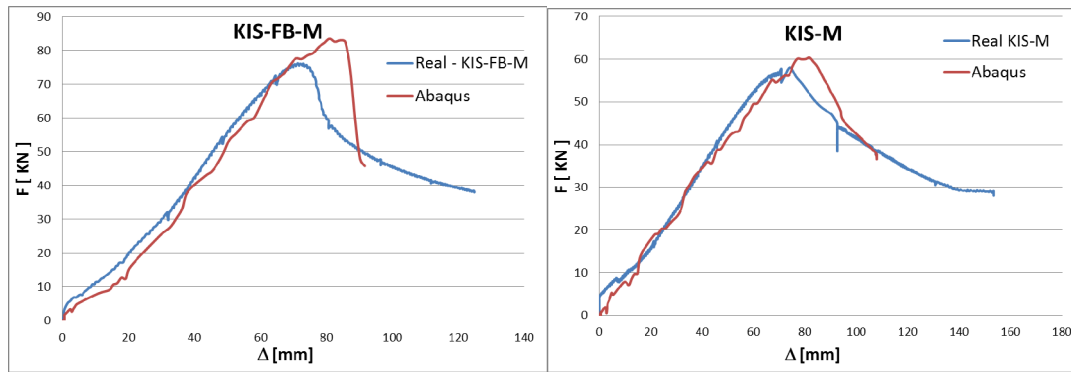


Figure 9: Knee specimens: F- Δ experimental and numerical curves

3.2 Parametric study of the ridge joints

The calibrated FE model described in previous chapter was used in the parametric study of the ridge joints (RIS-FB-M; RIS-M). First, the influence of the distance between the two bolt groups over global mechanical characteristics was monitored. The outer bolt group was kept on the position; the inner bolt group distance was gradually reduced, together with the length of the C profile, keeping the original end distances of the inner bolt group, till the inner and outer bolt group overlaps. Eight geometric configurations were studied, as it is presented in Figure 10. Configuration P1 to P4 has the same bolt numbers, in case of type P5 and P6 the number of bolts was reduced with two respectively with four bolts. P7 and P8 are applicable only for the specimens with flange bolts. The ‘yield’ displacement and the corresponding conventional elastic capacity (Figure 11) were determined according to the ECCS procedure [12]. Model P1 corresponds to the tested specimens (RIS-FB-M; RIS-M).

Table 1 shows the variation of the computed maximum force P_{max} and the corresponding displacement v_{max} in the parametric study for the ridge joints (RIS-FB-M; RIS-M). According to the ECCS procedure, the yielding force P_e and the corresponding yielding displacement v_e was also computed.

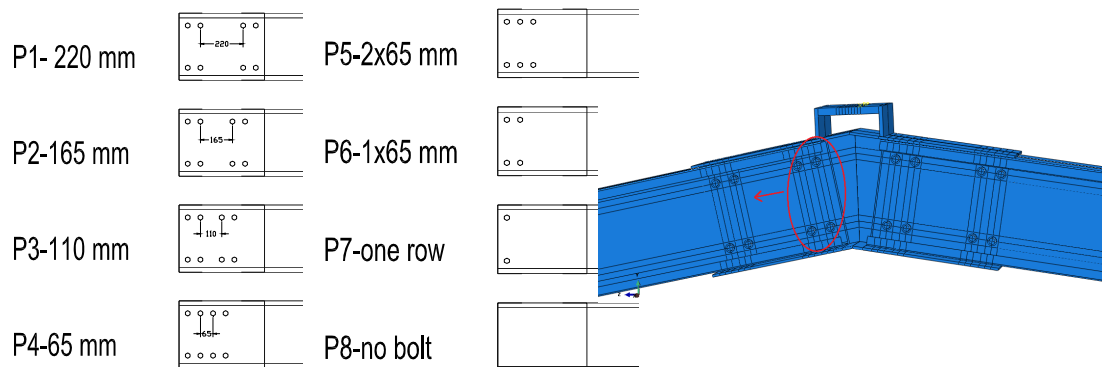


Figure 10: Ridge specimens used in parametric study

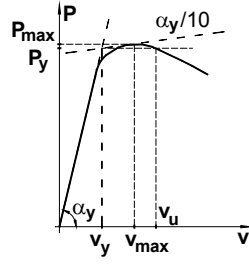
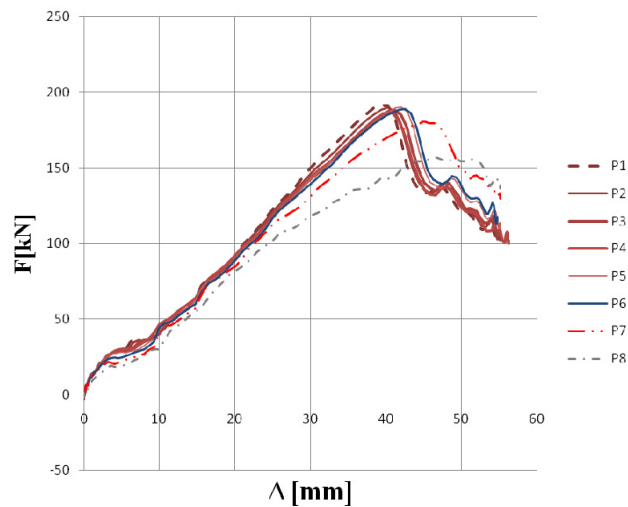
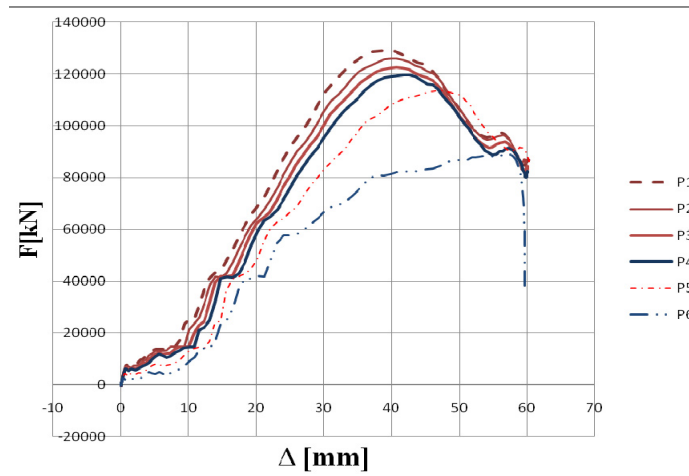


Figure 11: ECCS procedure [12] to define the yield displacement

As it can be observed in Figure 12, in case of the specimen with flange bolts, P1 to P6 models are almost insensitive to the reduction of the bolt distance between the inner and outer bolt groups. In case of specimens with web bolts only, there is an important capacity difference compared with the specimen with also flange bolts (in case of specimen with web bolts only P_e is 35% lower), but until no any decrease in total bolts number, there is no any significant decrease in load carrying capacity of the specimen. Only in case of specimen P5 and P6 in the load carrying capacity can be observed moderate (reducing the number of web bolts with 2) or significant decrease (reducing the number of web bolts with 4), simultaneously with moderate or significant increase in deformations of the ridge specimen. In case of the specimens with flange bolts, a P7 and P8 model also were analyzed. This corresponds to the situation with only two bolts on web (P7) or no any bolts on the web (P8). Moderate (P7) or significant decrease (P8) in load carrying capacity of the model with flange bolts was observed but only with moderate increase in deformations of the ridge specimen. All the computed graphs from FE analysis for RIS-FB-M and RIS-M specimens are shown in Figure 12 (load-displacement) and Figure 13 (moment-rotation). Associated labels (Table 1,2,3,4) includes the mechanical characteristics evaluated according to the ECCS procedure [12] based on FEM results.

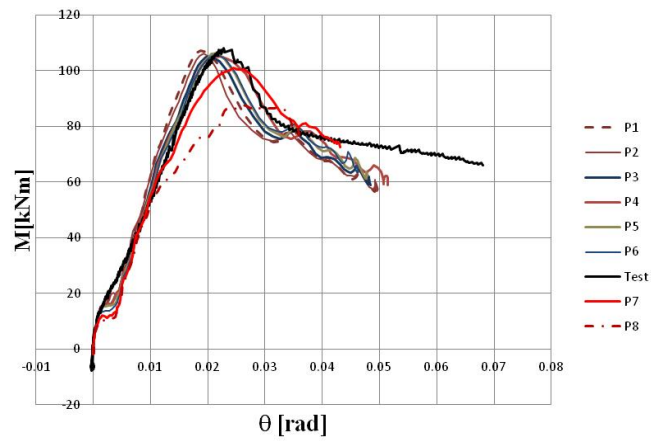


a. Ridge specimens RIS-FB-M with flange bolts

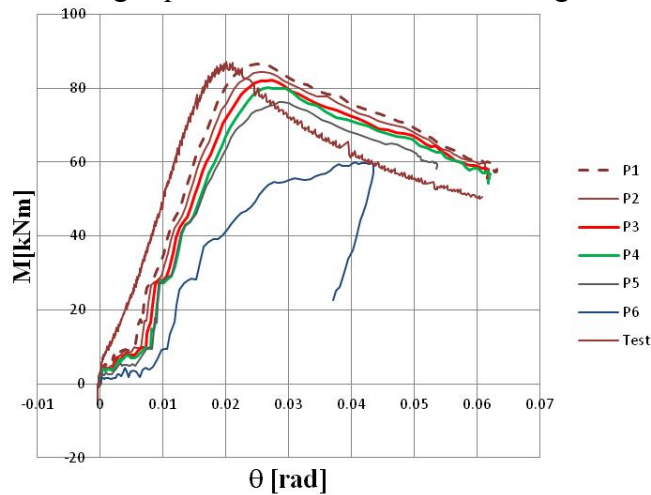


b. Ridge specimens RIS-M with web bolts only

Figure 12: Load – displacement graphs obtained in the parametric study



a. Ridge specimens RIS-FB-M with flange bolts



b. Ridge specimens RIS-M with web bolts only

Figure 13: Moment – rotation graphs obtained in the parametric study

Model RIS-FB-M	P_e [kN]	v_e [mm]	P_{max} [kN]	v_{max} [mm]
Test P1	190.40	34.10	193.10	39.10
P1	190.67	37.08	91.94	39.42
P2	188.44	37.63	189.84	40.29
P3	186.56	37.99	188.19	41.15
P4	184.44	37.39	186.36	40.90
P5	188.11	38.29	190.06	42.05
P6	187.11	38.85	189.07	42.55
P7	177.56	38.48	180.74	45.06
P8	152.78	36.84	157.00	46.41

Table 1: Parametric study results – Specimens with flange bolts

Model RIS-M	P_e [kN]	v_e [mm]	P_{max} [kN]	v_{max} [mm]
Test P1	127.30	26.70	130.50	31.50
P1	127.70	35.83	129.21	40.02
P2	125.10	37.00	126.07	39.83
P3	121.70	37.57	122.74	40.70
P4	118.20	38.28	119.66	42.63
P5	112.10	41.76	113.66	47.07
P6	87.56	48.06	89.22	57.16

Table 2: Parametric study results – Specimens with web bolts only

Specimen RIS-FB-M	S_{j,ini} [kNm/rad]	θ_y [rad]	θ_u [rad]	μ	M_{max} [kNm]
Test P1	6011.10	0.01700	0.02500	1.40	108.00
P1	5600.00	0.01746	0.01992	1.141	107.28
P2	5350.00	0.01859	0.02021	1.087	106.12
P3	5310.00	0.01841	0.02065	1.121	105.20
P4	5200.00	0.01891	0.02236	1.182	105.20
P5	5250.00	0.01911	0.02157	1.129	106.24
P6	5250.00	0.01915	0.02189	1.143	105.69
P7	4880.00	0.01935	0.02448	1.265	101.04
P8	4650.00	0.01780	0.02612	1.467	87.76

Table 3: Evaluated stiffness – Specimens with flange bolts

Specimen RIS-M	$S_{j,ini}$ [KNm/rad]	θ_y [rad]	θ_u [rad]	μ	M_{max} [KNm]
<i>Test P1</i>	5806.8	0.018	0.028	1.600	74.30
P1	4724.2	0.020	0.029	1.450	86.44
P2	4462.5	0.022	0.030	1.364	84.40
P3	4208.3	0.022	0.029	1.318	82.15
P4	3999.3	0.024	0.031	1.292	80.01
P5	3667.8	0.024	0.032	1.333	76.25
P6	1923.7	0.031	0.044	1.419	59.84

Table 4: Evaluated stiffness – Specimens with web bolts only

4 Application of the component method

The component method is a general procedure for design of strength and stiffness of joints in building frames, and is implemented in EN1993-1.8 [7]. The procedure is primarily intended for heavy-gauged construction. Its application to joints connecting light-gauge members was investigated and found appropriate with minimum set of adjustments [8]. Based on the conclusions of experimental programme, only joints with both web and flange bolts (RIS-FB-M, KIS-FB-M, and KIP-FB-M) were investigated.

Due to member flexibility and local buckling, the connected members do not behave as rigid bodies, and the centre of rotation of web bolts does not coincide with the centroid of web bolts. The centre of rotation of the connection is shifted towards the outer bolt rows, whose corresponding force is an order of magnitude higher than the force in the inner bolts. Inner bolt group has proven to be unefficient in the analysed joints.

The following components were identified and used to model the connection stiffness and strength: (1) Cold-formed member flange and web in compression. Only strength of this component was considered, while stiffness was considered infinite; (2) Bolts in shear; (3) Bolts in bearing on the cold-formed member; (4) Bolts in bearing on the bracket. Details related to the application of component method are presented in [8]. Based on the reduced influence of the inner bolt group over the mechanical characteristics of the whole joint, only the outer group of bolts were considered in the component method.

The configuration of the outer group of bolts being the same in the case of all specimens with web and flange bolts (RIS-FB-M, specimen P1 to P6), a single set of analytical connection properties were determined. On Figure 13 it can be seen that no any important difference between specimens P1 to P6 in terms of moment resistance and stiffness. The situation without web bolts (P8 specimen) was also computed with the component method. A comparison of experimental vs. analytical characteristics of the analysed connections using the component method and the computed FEM results (stiffness and moment resistance) are presented in Table 5. Generally a fair agreement between experimental and analytical stiffness of the connection can be observed. Stiffness of the connection is considerably lower than

the EN1993-1-8 limits for classification of joints as rigid ($25EI_b/L_b$), which amounts to 25256 kNm/rad (considering the beam span L_b equal to frame span and using gross moment of inertia I_b). Therefore, these types of connections are semi-rigid, and their characteristics need to be taken into account in the global design of frame.

For the evaluation of the joint stiffness, the proposed simplified method should be used. The results obtained analytically, applying the component method are shown in Table 5, where FEM results and experimental results are also included. Figure 14 shows good agreement between the methodologies applied.

Specimen RIS-FB-M	Initial stiffness, K_{iniC} [kNm/rad]			Moment resistance, M_C [kNm]		
	experimental	analytical	FEM	experimental	analytical	FEM
P1	6011	5224	5600	108.0	117.8	107.3
P8	-	5043	4650	-	89.9	87.7

Table 5: Experimental, analytical and FEM connection characteristics

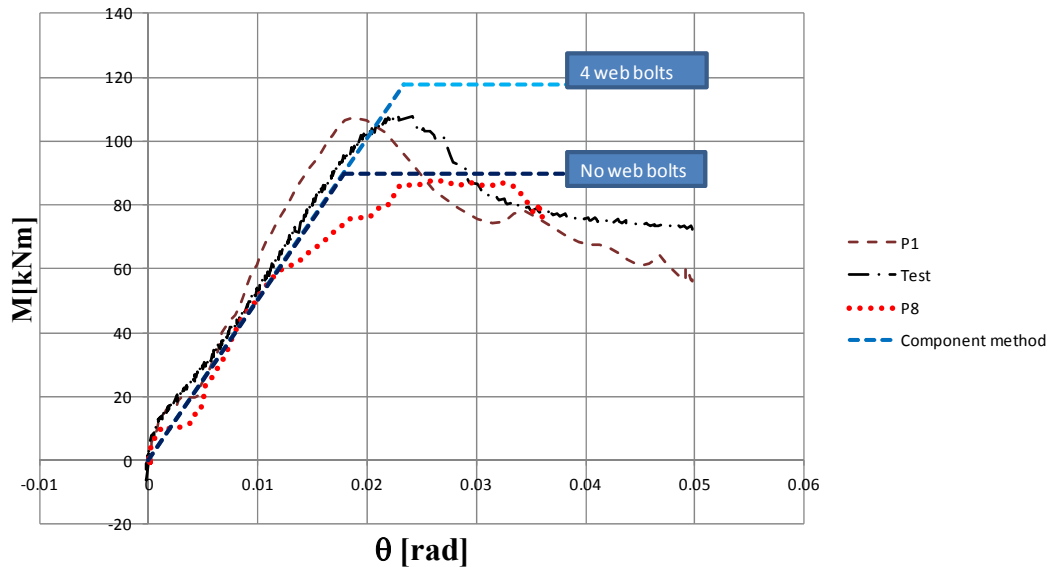


Figure 14: Experimental, analytical and FEM results

In case of the P1 specimen, moment resistance of the bolted connection $M_{bC,Rd}$ determined by the component method amounted to 193.9 kNm, which was larger than the moment resistance of the cold-formed member $M_{beam,Rd}$, amounting 117.8 kNm. Therefore, this type of connection is a full-strength one. This was demonstrated also by the experimental results, failure mode being local buckling of the cold-formed member. In case of the P8 specimen, moment resistance of the bolted connection $M_{bC,Rd}$ determined by the component method amounted to 89.9 kNm, which is less than the moment resistance of the cold-formed member $M_{beam,Rd}$, amounting 117.8 kNm. Therefore, this type of connection is partial-strength. According to FEM analysis results, failure mode being local buckling of the compressed flange of cold-formed member due to stress concentration around the flange bolts. In the component method, the bearing resistance on the cold-formed profile flange define the moment resistance of the P8 joint.

5 Discussion

In case of the specimen with flange bolts (RIS-FB-M), the inner web bolt group contribution to the resistance and stiffness of the joints is negligible, since there are no any important differences between model P1 up to P6 (see Figure 13a). Reducing the number of bolts in outer group, there is a limited impact in the resistance and stiffness of the joint, which is under 25 % compared with the resistance of the tested specimens.

In case of the specimen with web bolts only (RIS-M), the inner web bolt group contribution to the resistance and stiffness of the joints is also reduced, since there are no any important differences between model P1 up to P4 (see Figure 13b). Reducing the number of web bolts, gives a more significant impact in the resistance and stiffness of the joint. In terms of resistance the decrease is around 30 %, compared with the resistance of the tested specimens.

The calculation model for the connection in both cases, based on the linear distribution of the force on each bolt is applicable only if the centre of gravity of the whole connection is considered on the symmetry axis of the analyzed specimen. In this particular case, the calculation model only for the bracket to C profile connection, based on the linear distribution of the force on each bolt is not correct. The force distribution is unequal due to the flexibility of the connected members in the joint. In both cases, the force is an order of magnitude bigger in the outer bolt rows compared to inner one. All connections failed by local buckling of the compression web (in case of specimens with web bolts only) or flange (in case of specimens with flange bolts) closest to the connection bracket itself. Material yielding and local and distortional buckling of the joint components due to stress concentration were observed both in tests and developed FE models.

The computed moment-rotation curves in the parametric study, confirm the methodology based on the component method in order to characterize the behaviour of such type of joints and use as input for frame analysis.

6 Conclusions

The goal of the paper was the parametric study using the previous calibrated FE model of back-to-back cold-formed steel lipped channel bolted joints. A FE model has been calibrated for ridge and knee specimens based on laboratory tests results. Using the calibrated FE model, fourteen types of different joint configurations were analyzed: Eight ridge configurations with flange and web bolts and six with web bolts only. According to the parametric FE analysis results, the following conclusion can be stated:

1. The calibrated knee and ridge models are able to reproduce the same failure mechanism as obtained experimentally, offering a fair force-displacement and moment-rotation agreement between test and FE results;
2. Using the calibrated models, a quite good evaluation of the load carrying capacity of the analyzed joints can be obtained;

3. The calculation model for the connection in both cases, based on the linear distribution of the force on each bolt is applicable only if the centre of gravity of the whole connection is considered on the symmetry axis of the analyzed specimen; this imposes to account for the joint as a whole;
4. Increasing the distance between the cold formed C profile flanges and the stiff bracket on the calibrated numerical model, there is insignificant differences in results in terms of stiffness and load carrying capacity of the joint. This leads to conclude that no any significant influence of the bracket-cold formed C flange contact in the joint global behaviour.
5. The methodology based on the component method in order to characterize the behaviour of such type of joints is relatively simple to apply; the generated results can be used as input for linear elastic frame analysis.

The results and conclusions of this paper have to be limited to the joint typologies and range of section dimensions used in the simulation program. The research will continue to refine the results obtained and to extend the proposed analytical procedure using the component method, to be able to evaluate the stiffness and strength with reasonable differences compared with experimental results, for all the tested specimens.

Acknowledgements

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