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

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PARAMETRIC STUDY OF COLD-FORMED STEEL BOLTED JOINTS IN PITCH-ROOF PORTAL FRAMES

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Abstract: Previous studies reported the results of a large experimental and numerical program carried out at the “Politehnica” University of Timisoara and Technical University of Cluj, in order to evaluate the performance of three different configuration of ridge (apex) and eaves knee joints of pitched roof cold-formed steel portal frames of back-to-back lipped channel sections and bolted joints under monotonic and cyclic loading [1]. The behaviour and failure mechanisms of joints were observed, to evaluate their rigidity, strength and ductility. In a second phase, considering the poor performance of joints with web bolts only, joint configurations with both web and flange bolts were used to test two full-scale double frame units under: (1) horizontal load, and (2) horizontal and gravity loadings [2]. The objective of the full-scale tests was to assess performance of pitched-roof cold-formed portal frames with moment-resisting joints under lateral loading, with particular emphasis on earthquake loading. Numerical analysis of the tested joints was performed, obtaining calibrated FE models.
A parametric study consisted of 8 ridge joint configurations with flange bolts and 6 ridge joint configurations with web bolts only was conducted using the calibrated numerical model, in order to identify the weakness of these types of joints. The failure modes of material yielding and local and distortional buckling of the joint components due to stress concentration were found in this study. The experimental joint strengths and numerical results predicted by the parametric study were compared with the design strengths calculated using the principles of component method proposed by current European specifications for steel joints.
The paper summarizes the experimental work; the numerical results of the tested ridge joints and the parametric study on the developed joint typologies are presented to determine the mechanical characteristics of the analyzed connections.

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

I. 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 [1]. 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 is by 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 determination of connections characteristics.

II. 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 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 N/mm^2).

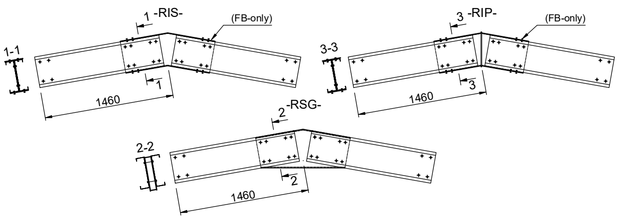


Figure 1. Configurations of ridge joints.

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 denoted 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 testes. 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.*).

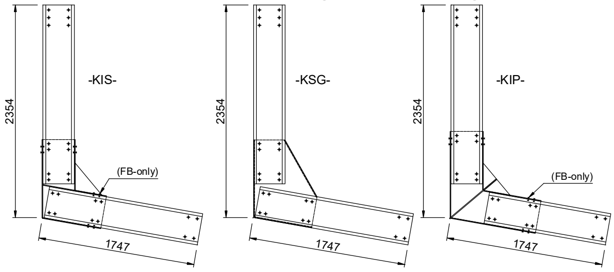
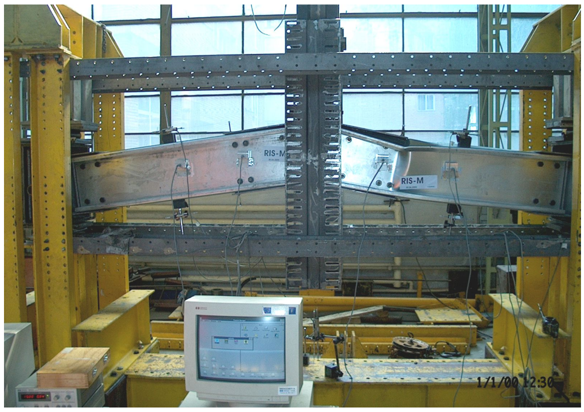
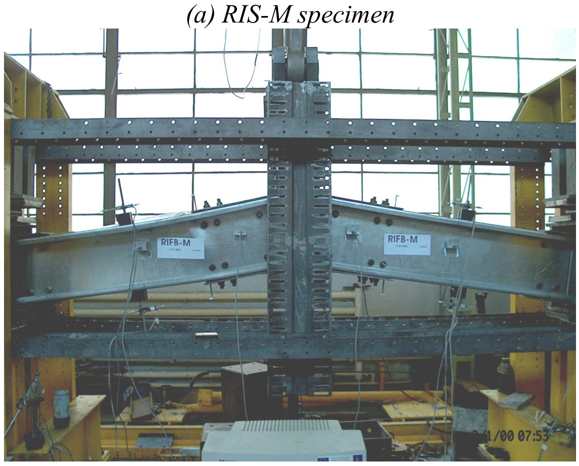


Figure 2: Configurations of knee joints



(a) RIS-M specimen

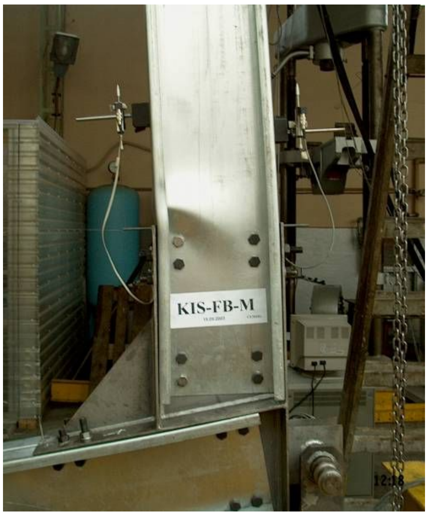


(b) RIS-FB-M specimen

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 bucking 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.

(a) KIS-M knee specimen

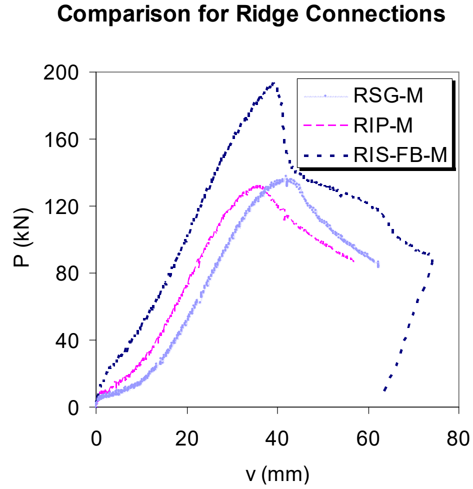


(b) KIS-FB-M knee specimen

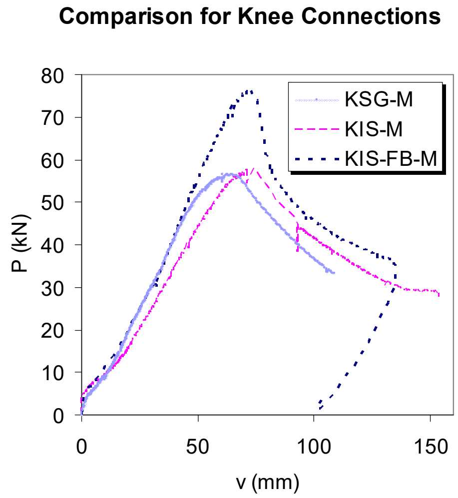
Figure 4: Failure of knee specimens

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).

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.



(a) ridge joints



(b)knee joints

Figure 5: Comparative results from monotonic tests

III. FINITE ELEMENT ANALYSIS OF RIDGE AND KNEE JOINTS – CALIBRATION PROCESS

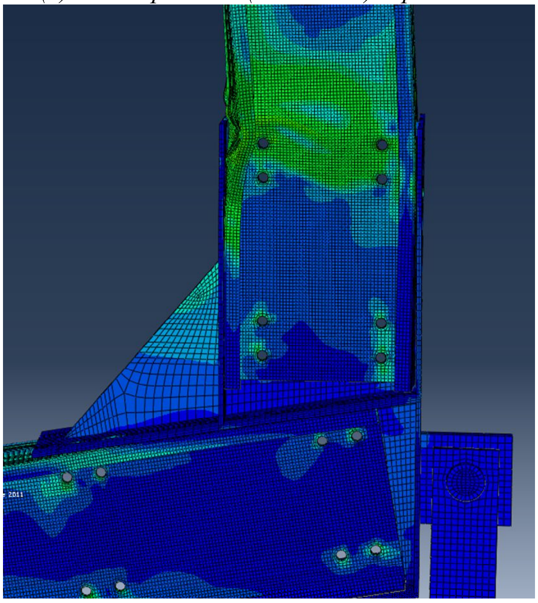
To study the structural behaviour of the joints (RIS-FB-M; KIS-FB-M; RIS-M; KIS-M), finite element models have been developed, they are shown in *Figure 6.* and *Figure 7*. The numerical models for the tested specimens were created using the finite element analysis software package ABAQUS/CAE v.6.10 [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 × 8 mm was considered for C-profiles and 12mm × 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/mm^2, ultimate tensile strength 553N/mm^2, Young's modulus E=210000N/mm^2 and a measured thickness minus zinc coating of 2.90mm.

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).

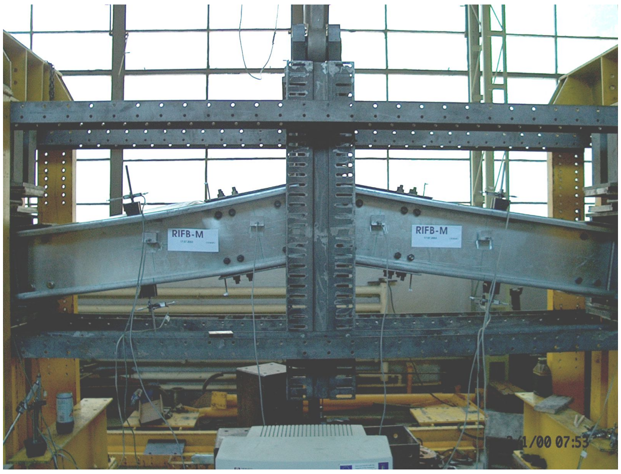
The calibrated FE models reproduce the same failure mechanism in case of knee specimens (KIS-FB-M, see *Figure 6.*) as obtained experimentally. In case of ridge specimens (RIS-FB-M, see *Figure 7.*), due to the symmetry of the model, the failure was distributed between the symmetric branches, obtaining two local failure mechanisms (see *Figure 7b.*) instead of one obtained experimentally. Due to this aspect, in case of ridge specimens, the total rotation of left and right branches was considered. *Figure 8.* presents the force-displacement curves obtained both experimentally and numerically, for all types of joints. Good agreement can be observed between groups of curves.

(a) Knee specimen (KIS-FB-M) experiment

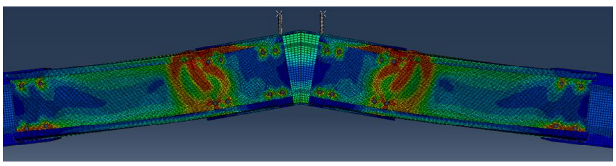


(b) Knee specimen (KIS-FB-M) FEM model

Figure 6: Knee experimental test vs. ABAQUS results



(a) Ridge specimen (RIS-FB-M) experiment



(b) Ridge specimen (RIS-FB-M) FEM model

Figure 7: Ridge experimental test vs. ABAQUS results

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.

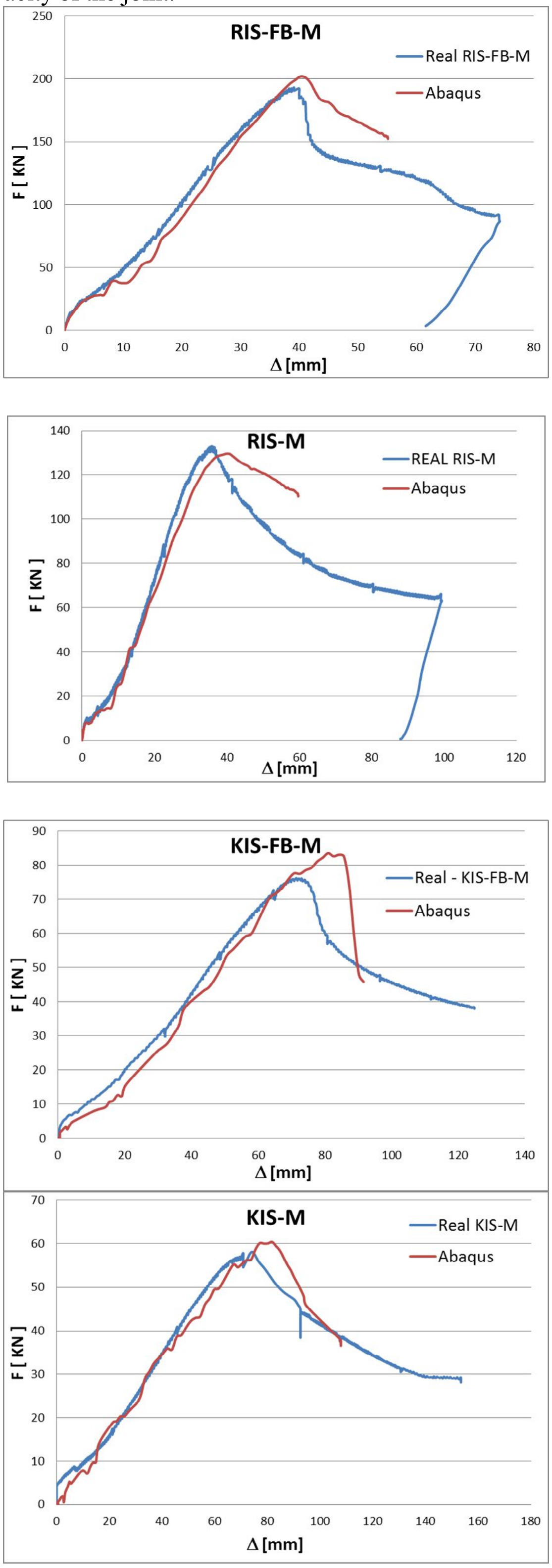


Figure 8. Knee and ridge specimens: F-Δ experimental and numerical curves

IV. PARAMETRIC STUDY OF 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. Six geometric configurations were studied, as it is presented in *Figure 9*. Configuration 1 to 4 has the same bolt numbers, in case of type 5 and 6 the number of bolts was reduced with two respectively with four bolts. The 'yield' displacement and the corresponding conventional elastic capacity were determined according to the ECCS procedure (*Figure 10*).

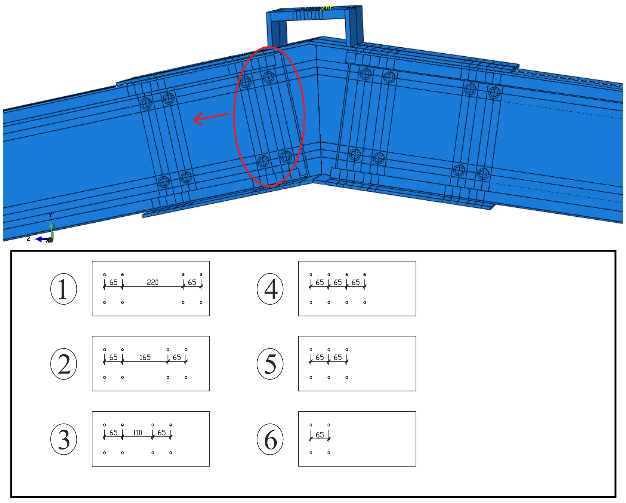


Figure 9. RIS-M ridge specimens used in parametric study

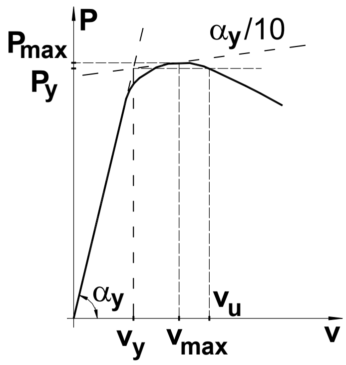


Figure 10. ECCS procedure to define the yield displacement

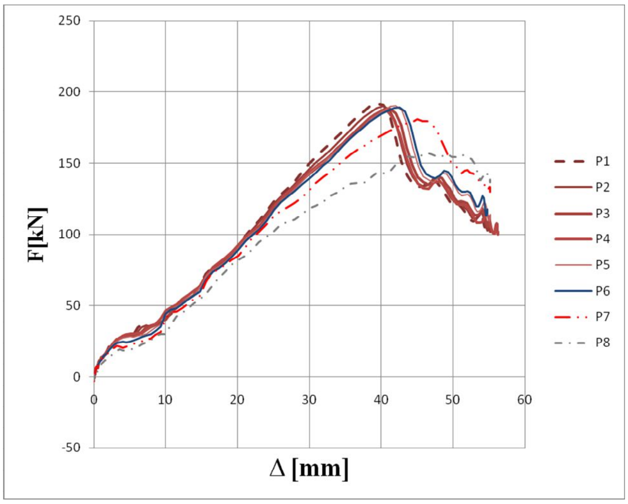
Model P1 corresponds to the tested specimens (RIS-FB-M; RIS-M). *Table 1* shows the variation of the computed maximum force P_{max} and the corresponding displacement v_{max} in the parametric study for the ridge joints (RIS-FB-M; RIS-M). According to the ECCS procedure, the yielding force P_e and the corresponding yielding displacement v_e was also computed. As it can be observed, in case of the specimen with flange bolts, there are insignificant differences between the specimens involved in the parametric study, since the flange bolts are present in all specimens. 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 5 and 6 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.

Model	**P_e**	**v_e**	**P_{max}**	**v_{max}**
RIS-FB-M	**[kN]**	**[mm]**	**[kN]**	**[mm]**
Test	***190.40***	***34.10***	***193.10***	***39.10***
P1	190.67	37.08	191.94	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

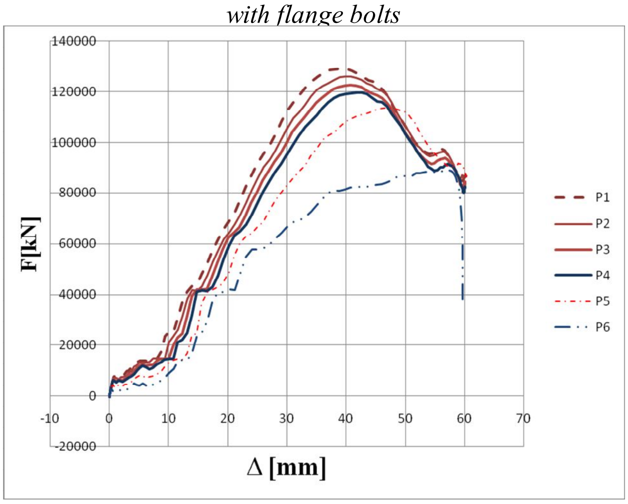
Model	**P_e**	**v_e**	**P_{max}**	**v_{max}**
RIS-M	**[kN]**	**[mm]**	**[kN]**	**[mm]**
Test	***127.30***	***26.70***	***130.50***	***31.50***
P1	127.70	35.83	129.21	40.02
P2	125.10	37.00	126.07	39.83
P3	121.70	37.57	122.74	40.70
P4	118.20	38.28	119.66	42.63
P5	112.10	41.76	113.66	47.07
P6	87.56	48.06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

Table 1. Parametric study results – ridge specimens

It is also important to mention that in case of model 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 11*.



Force – displacement graphs, ridge specimens RIS-FB-M with flange bolts



Force – displacement graphs, ridge specimens RIS-M with web bolts only

Figure 11. Graphs obtained in the parametric study

V. DISCUSSIONS

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 P1 up to P6 graphs. 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 P1 up to P4 graphs. 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 center 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.

Further studies based on these test results will focus on the moment-rotation curves and using the component method in order to characterize the behaviour of such type of joints for frame analysis.

VI. 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: 8 ridge configurations with flange and web bolts and 6 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 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 center 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 behavior.

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 obtained results and to extend the proposed analytical procedure using the component method to evaluate the stiffness and strength, for specimens having web bolts only.

ACKNOWLEDGMENT

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