FE modeling 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 program carried out at the Technical University of Timisoara 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 in order 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. The paper summarizes the experimental work and the results of the tested joints with (1) web bolts only and (2) both on web and flange bolts, in order to calibrate FE models. Describes the calibration procedure and parameters that were used to obtain the numerical models that accurately follow the real behaviour of the joints. These models form the basis of an extensive parametric study.

Rezumat

Studiile precendente indică rezultatele obţinute în urma unui extins program experimental derulat în cadrul Universităţii "Politehnica" Timişoara cu scopul evaluării performanţelor structurale a îmbinărilor cu şuruburi a cadrelor portal realizate din bare cu secţiune dublu C dispuse spate în spate prin încercări monotone asupra a trei configuraţii geomentrice de noduri atât de coamă cât şi de streaşină. A fost observat comportamentul şi mecanismul de cedare al acestora în scopul evaluării rigidităţii, rezistenţei şi ductilităţii. În faza a doua datorită performanţelor scăzute a nodurilor cu îmbinări doar la nivelul inimii, au fost utilizate doar noduri cu îmbinări cu şuruburi atât pe inimă cât şi pe tălpi pentru încercări ale ansamblului global realizat din două câte două cadre supus la (1) încărcări orizontale, şi (2) orizontale şi gravitaţionale. [2]. Scopul încercărilor la scară reală a cadrelor a fost evaluarea performanţei nodurilor la încărcări laterale, punând accentul în special pe acţiunile seismice. Lucrarea sintetizează activitatea experimentală şi rezultatele obţinute asupra nodurilor cu şuruburi pe inimă cât şi respectiv cu şuruburi pe tălpi şi inimă în scopul calibrării unor modele cu element finit. Este descrisă procedura de calibrare şi parametriii utilizaţi în urma căreia s-au obţinut modele numerice care urmăresc fidel comportamentul nodurilor. Aceste modele stau la baza unui extins program parametric de analiză.

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1. Introduction

The specific behaviour of portal frame structures realized of cold forms profiles mostly is due to the structural response of the nodes composition.

The joint configuration with metric bolts of the portal frame tried different geometric conformation made by M.F Wong & K.F. Chung [11], J.B.P Lim & D.A. Nethercot [3] [12] [13], Dundu & Kemp [4], A.Hazlan & M.Mahen [7], F. Santos and L. Simões da Silva [5]. In all these trials for the portal subassemblies and assemblies revealed that the nodes have a semi rigid character witch influenced crucially on the global response of the frame being partial resistant in almost all cases.

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 Technical University of Timisoara [1]. Detailed experimental results on joint behaviour are reported by Dubina et al. [1]. 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 [2].

Based on the obtain results in order to prepare an intensive parametric study, different numerical models were calibrated. An early version of the calibration process [14] exposed a good correlation between force-displacement curves but for the bending-rotation curves didn't overlapped closely over the reference specimens tested in the laboratory.

The paper summarizes the results of the finite element analyses (FEA), using recent studies to obtain a better calibration for the FE models of the tested joints.

The numerical results will be compared with the standard experimental specimens by overlapping the response curves.

2. Summary of testing program on joints

In order to be able 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 angle 10°. This frame was subjected to loads common in the Romanian design practice. These loads were totalling approximately 10 kN/m 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 Fig. 1 and Fig. 2), using welded bracket elements (S235: f_y=235N/mm^2).
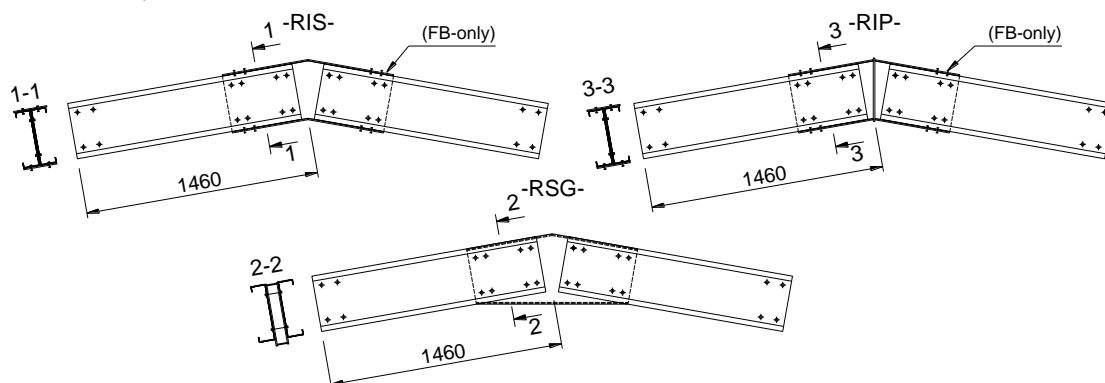

Figure 1. Configurations of ridge joints

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 Fig. 3 and Fig. 4).
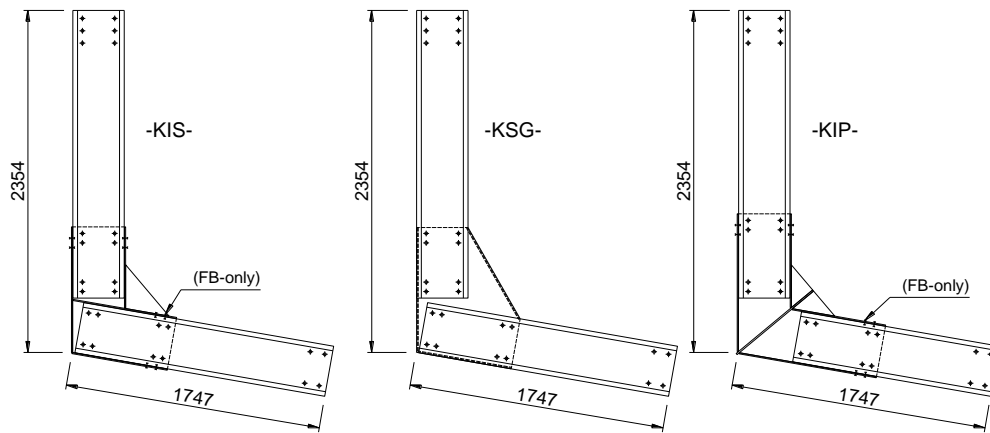

Figure 2. Configurations of knee joints

Figure 3. Failure of ridge specimens RIP-M (a) and RIS-FB-M (b)

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. Also was noted for the composed section double C that have unequal camber of the gaps, on the external row presented more pronounced deflection compared to the inside ones. Specimens of bolts installed both on the flanges and webs of connected members are nearly full resistant, but still remain semi-rigid.
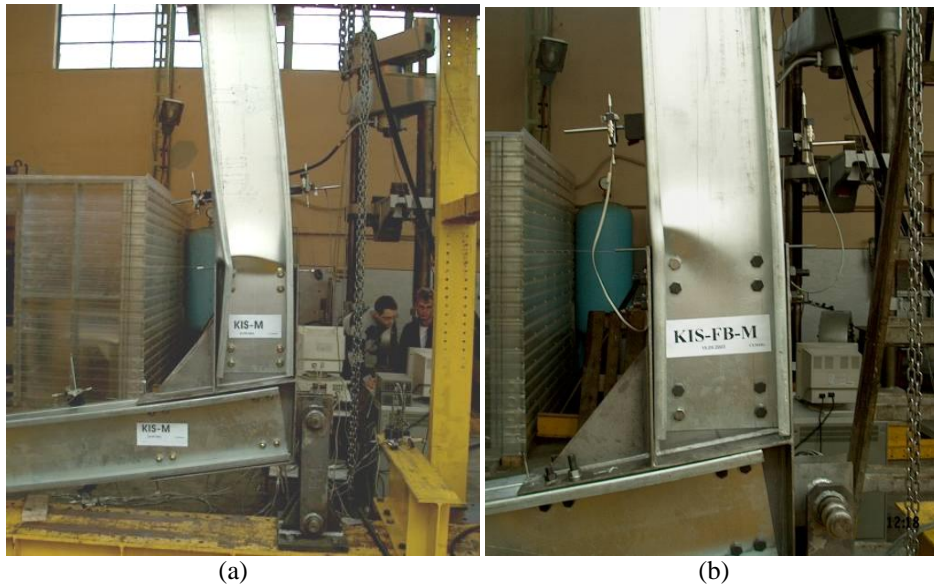
(a)   (b)

Figure 4. Failure of knee specimens KIS-M (a) and KIS-FB-M (b)

Comparative experimental curves for ridge and knee connections are presented in Fig. 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. Due to the simplicity of the assembly process these nodes typology reaches intense interest in different researches institutions.
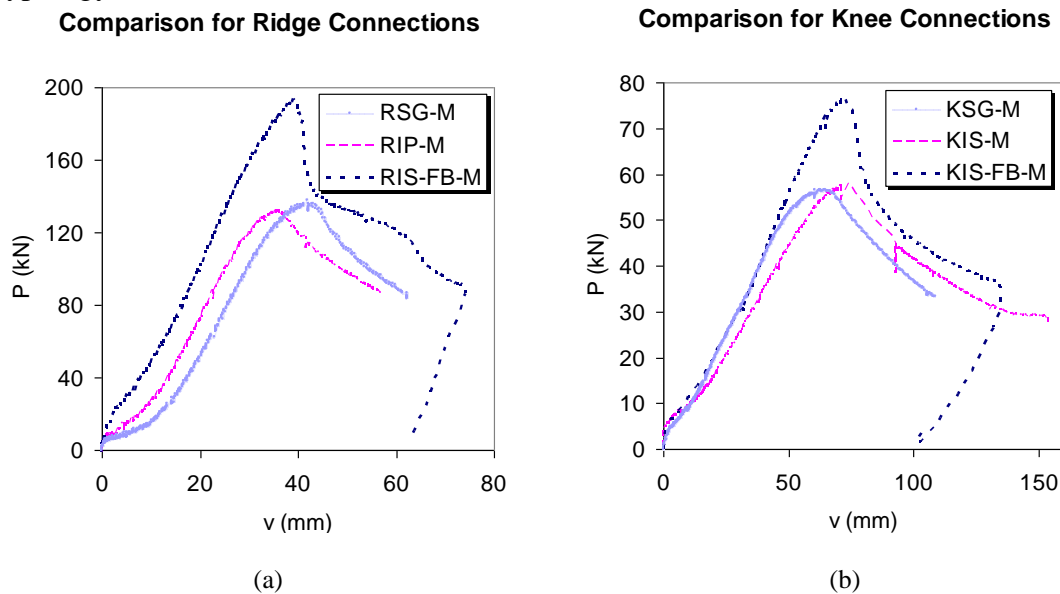

(a)   (b)

Figure 5. Comparative results from monotonic tests for ridge (a) and knee (b) joints

3. Finite element analysis of ridge and knee joints

Finite element models have been developed to study the structural behaviour of the three types of joints (RIS-FB-M; RIS-M; KIS-M), presented in Fig. 1 and Fig.2. Numerical models for the tested specimens were created using the finite element analysis software ABAQUS v.6.10 [9].

The numerical model was made for a set of components that includes lipped channels, brackets, bolts and washers. The components design was with the laboratory measured dimensions, the gaps

being designed with one diameter higher than the one from the bolt rod according to the reference specimen. Finite element was used for the gusset, bolts and nuts type: 8-noded 3D solid elements (C3D4) and back-to-back cold-formed lipped channels were designed standard quadratic, reduced integration, homogeneous shell element (S4R).

The interaction between parts was considered as hard contact behavior for normal direction and for tangential direction was chosen tangential friction coefficient of 0.25 as penalty.

The bolt ensemble was modeled in two components: bolt and nut to capture the effect of bolt tightening. The first step of the analysis consist the introduction of small forces of pre-tensioning.

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. For the C-profiles have adopted variable mesh for reduce the analysis time, with a mesh size of 8 mm × 8 mm for the joint adjacent portion and for the remaining element 12 mm x12 mm. The gusset mesh dimension was 12 mm x12 mm and for the bolts and nut was of 3 mm x 3 mm, this dimensions are providing enough accuracy. The wall openings, the rod bolts and the nut had an equal number of elements to provide an increased accuracy for the obtained results. The boundary conditions were considered as in the experimental tests, were blocked by the rotation and the translation, local buckling and vertical translation and clear for the horizontal translation and plane rotation.

The material behavior used for numerical modeling of C-sections was in accordance with the recorded curves from tensile tests (multi-linear isotropic model) and for the remaining elements the bilinear involve curves elastic-plastics with the characteristic value offered by Eurocode3 and Young's modulus $E=210000N/mm^2$ and 0,3 is the Poisson factor.

Nonlinear analyses were performed, using dynamic explicit steps as quasi-static, obtained by choosing time parameter and displacement amplitude so that the inertial effects on the system to be insignificant. The displacement amplitude was applied in the numerical program Abaqus using Smout-step curve with constant acceleration.

For the ridge specimen, the boundary conditions and the applied load are symmetrical and the failure mode was unsymmetrical. In this sense for the specimens was made a study of imperfection sensitivity. Geometrical imperfection was analysed of the pieces and the ensemble according to EN 1090-2:2008 presented in Figure 6. The analysis results had showed that for each imperfect element applied: the failure mode, the strength tensile, the initial stiffness doesn't modify from the initial element but the failure mechanism is translated to one of the branches.
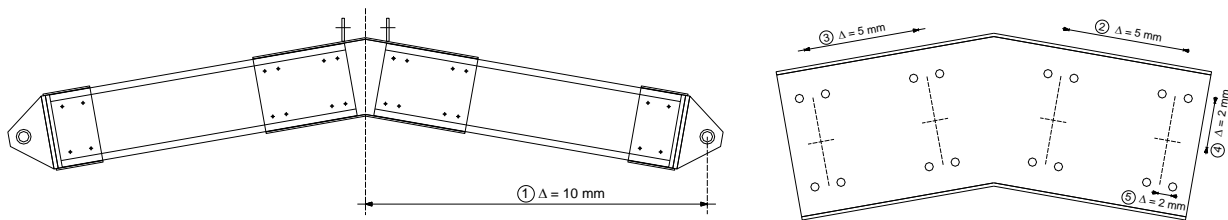

Figure 6. Geometrical limit of variation used for the sensitivity study of imperfections

The calibrated FE models reproduce the same failure mechanism for all the specimens (see Figure 7, Figure 8) as obtained experimentally. In case of ridge specimens (RIS-FB-M, see Figure 8), due to the imperfection implementation of the element, the failure is distinguish unsymmetrical.
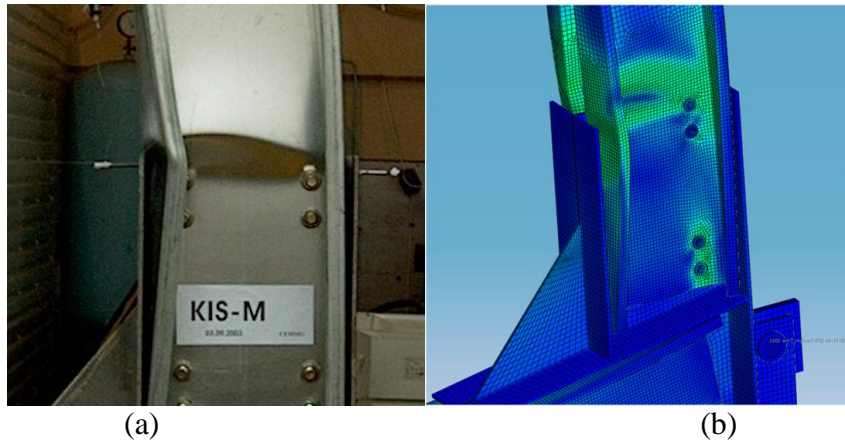
(a)   (b)

Figure 7. Knee specimen (KIS-FB-M) experimental test (a) and ABAQUS (b)

(a)

(b)

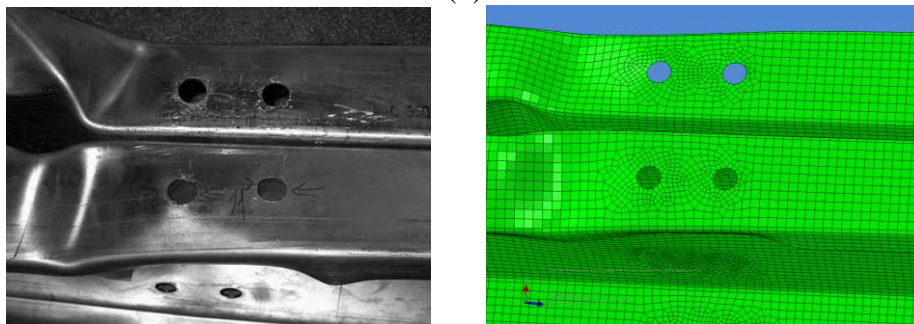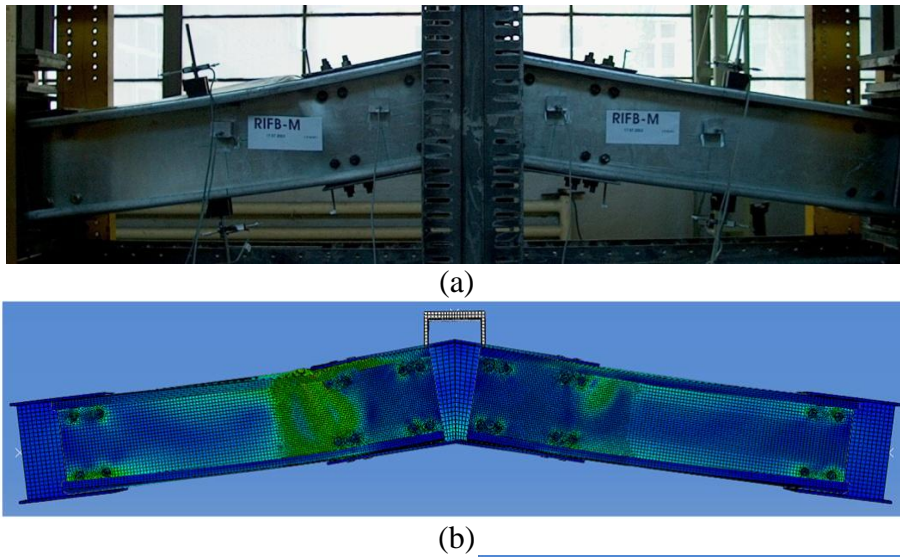

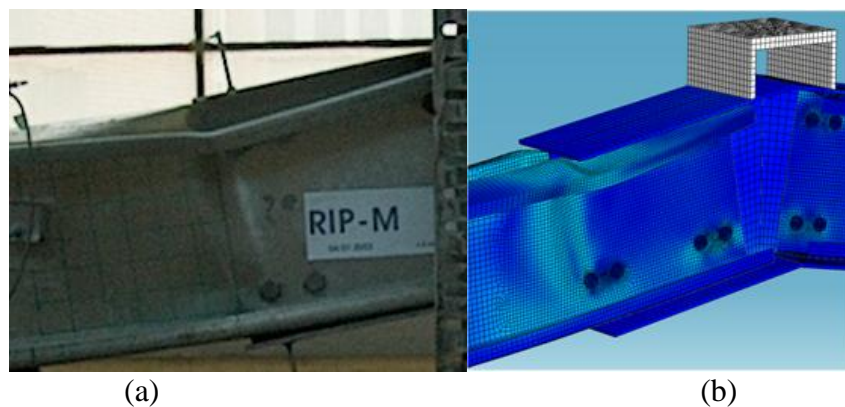Figure 8. Ridge specimen (RIS-FB-M) experimental test (a) and ABAQUS (b)

(a)   (b)

Figure 9. Ridge specimen (RIS-M) experimental test (a) and ABAQUS (b)

Table.1 Comparative results

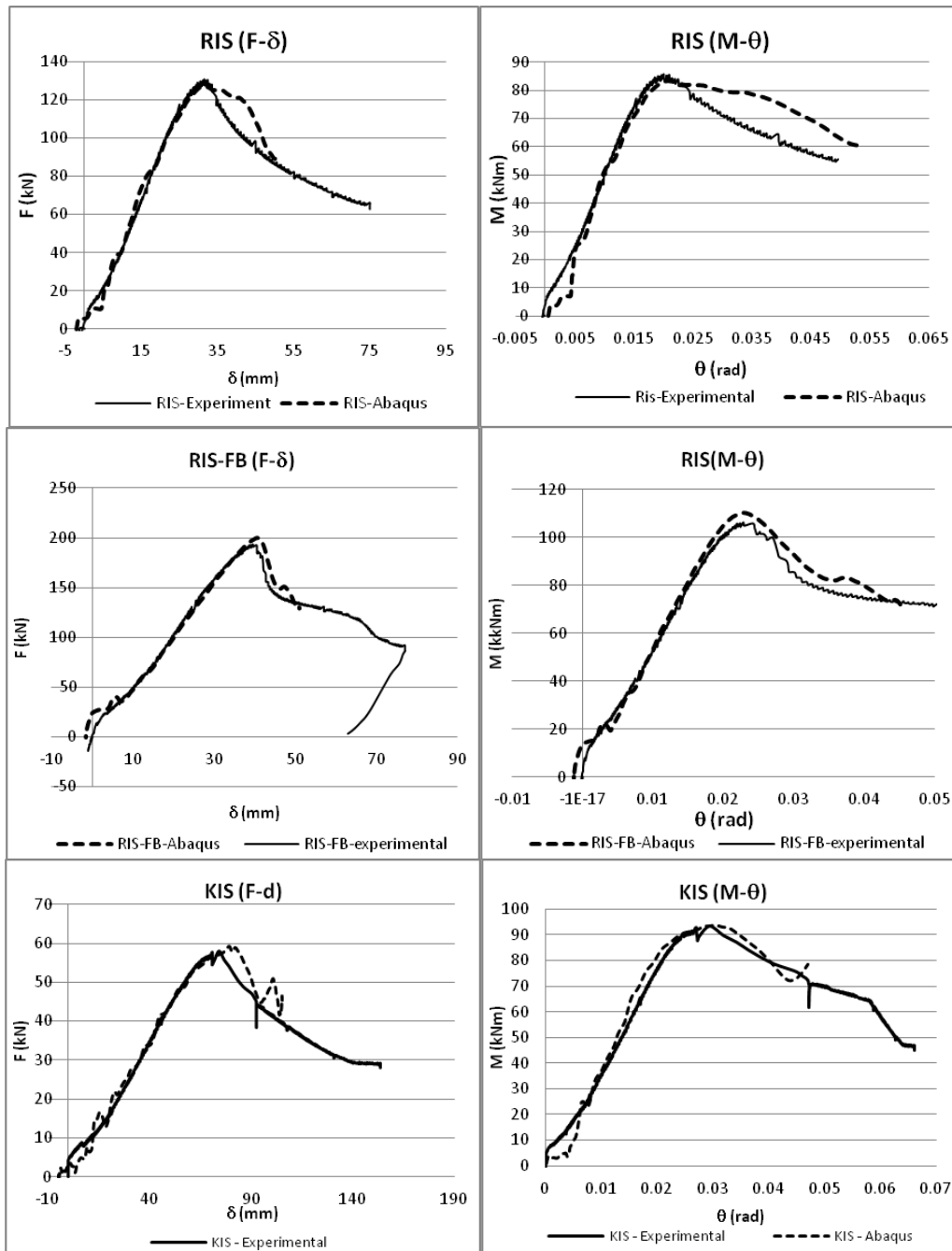| RIS					
Test type	F_{max}	d_{max}	M_{max}	q_u	$S_{j.ini}$
Experimental	130	31.5	87.03	0.02	5071
Abaqus	126.7	31.7	84.56	0.021	5020
RIS-FB					
Test type	F_{max}	d_{max}	M_{max}	q_u	$S_{j.ini}$
Experimental	193	39.1	106.3	0.023	6259
Abaqus	200	40.6	110.2	0.023	6697
KIS					
Test type	F_{max}	d_{max}	M_{max}	q_u	$S_{j.ini}$
Experimental	58.1	74.2	92.6	0.029	4301
Abaqus	59.3	79.1	93.23	0.0283	4843

Figure 10. Knee and ridge specimens –experimental and numerical curves

Comparative results are showed in Table.1 are presented of the numerical model and for the reference specimens are showed in Figure.10 can be noticed the force-displacement curves obtained both experimentally and numerically, for all types of nodes. Good agreement can be observed between groups of curves.

In the experimental cases of M-θ, rotation was computed through the displacement captors placed at the two flanges level, in the FEM model was determined through the gusset and the beam by determining the difference from the absolute rotation.

It can be observed that the force and the bending moment capacity of the joints are quite well fitted with the test results, the maximum variation is 3.6%.For the node RIS-M the stiffness is elastic bearing is well fitted but the post failure presents variations between the two types of test. For the RIS-FB-M and KIS-M elements the bearing post failure is better fitted, and the elastic zone presents small difference of stiffness between the pairs of curves with a 7% variation, and respectively 12%. In the stiffness terms, to this value, the difference appeared does not have a significant influence to the structural behavior of the specimens.

4. Conclusions

This study aimed to obtain a functional elements FE of joints back-to-back cold-formed steel lipped channel bolted, that can form the basis of a future experimental numerical program parametric that influences the forming of the plastic mechanism.

Elements were modelled in FE Abaqus program for the ridge specimens and eaves specimens that was calibrated based on the experimental results realized in the laboratory of steel structures of Timisoara.

The obtain elements by a refined analysis method the components discretisation and by sensitive studies to imperfection lead to more exact results than for the previous studies [14]. It can be remarked that the failure is unsymmetrical of the eaves models and in the laboratory trials.

The calibrated knee and ridge models are able to reproduce the same failure mechanism as obtained experimentally, offering a fair agreement between force-displacement curves and also moment-rotation. The variation between efforts of the curves is maximum 3.6%. It can be observed also a fitted correlation of the initial stiffness with a variation between 7-12%.

The nodes can be considered that are properly calibrated and 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 with parametric studies in order to achieve analytical procedure to evaluate the stiffness and strength, for specimens having web bolts only.

5. References

[1] Dubina D, Stratan A, Ciutina A, Fulop L, Nagy Zs. "Performance of ridge and eaves joints in cold-formed steel portal frames". *Proc. of the 17th International Specialty Conference*, Orlando, Florida, USA, 727-742, 2004.
[2] Dubina D, Stratan A, Nagy Zs. "Full – scale testing of cold-formed steel pitched-roof portal frames of back-to-back channel sections and bolted joints". *Proc. of the Sixth International Conference on Steel and Aluminum Structures*, Oxford, UK, 931-939, 2007.
[3] Lim JBP. "Joint effects in cold-formed steel portal frames". University of Nottingham, Ph.D. Thesis, 2001.
[4] Dundu M, Kemp AR. "Strength requirements of single cold formed channels connected back-to-back". *J. of Constructional Steel Research*, Vol. 62, Issue 3: 250-261, 2006.
[5] Filipe Santos and Luis Simões da Silva - Connections Of Cold Formed Profiles In Industrial Buildings ; *The 6th International Conference on Thin Walled Structures 2011,*

Timisoara,Romania
[6]   EN1993-1-8. Eurocode 3: "Design of steel structures - Part 1-8: Design of joints". European Committee for Standardization,Brussels, 2005.
[7]   Ahamed, HazlanșiMahendran, Mahen - Bolted beam-column moment connections between cold-formed steel members. In 21st Australasian Conference on the Mechanics of Structures and Materials (ACMSM 21), Victoria University, Melbourne
[8]   EN1993-1-3. Eurocode 3 – Part 1-3: "Supplementary rules for cold-formed thin gauge members and sheeting". European Committee for Standardization, Brussels, 2006.
[9]   ABAQUS. "Theory manual", Hibbit, Karlson and Sorenson, Inc, 2007.
[10]   M F Wong and K F Chung - *Experimental investigation of cold-ormed steel beam-column sub-frames pilot study;* Fifteenth International Specialty Conference on Cold-Formed Steel Structures ; St. Louis, Missouri U.S.A., October 19-20, 2000
[11]   M.F. Wong, K.F. Chung- Structural behaviour of bolted moment connections in coldformed steel beam-column sub-frames*;* Journal of Constructional Steel Research 58 (2002) 253–274
[12]   J.B.P. Lim , D.A. Nethercot- Design and development of a general cold-formed steel portal framing system; The Structural Engineer, 5 November 2002
[13]   J.B.P. Lim , D.A. Nethercot- Stiffness prediction for bolted moment connections between cold-formed steel members; Journal of Constructional Steel Research 60 (2004) 85–107
[14]   Paul Pernes and Zsolt Nagy „Calibration of a finite element model for evaluation of cold-formed steel bolted joints in pitch-roof portal frames" Proceedings of the International Conference on Thin-Walled stuctures 5-7 September 2011, Timisoara , Romania, Volume 1, Session 4 "Connection in thin-walled structures" pag 537-544, ISBN 978-92-9147-102-7