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INFLUENCE OF MEMBER COMPONENTS ON THE STRCUTURAL PERFORMANCE OF BEAM-TO-COLUMN JOINTS OF PITCHED ROOF PORTAL FRAMES WITH CLASS 3 AND 4 SECTIONS

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Abstract. *Pitched roof portal frames, largely used for industrial steel buildings are usually made of slender welded sections, characterized as low dissipative. Frame members are of variable cross-section in accordance with the stress and stiffness demand and Class 3 and/or Class 4 web section may be obtained. An extensive parametrical investigation on a significant number of beam-to-column joints for pitched roof portal frames with tapered column and hunched rafter is presented in order to establish their sensitivity in relation to the variation of different components of the joint. Different values are used for flange and web thickness in order to obtain sections of Class 3 and/or Class 4, or for the end plate (t_p=15 mm, 20 mm, 25 mm). Different steel grades are also used e.g. S235, S355 or S460. The sensitivity is analysed through main characteristics of the joint (i.e. moment capacity and stiffness). The parametric study is performed by FEM non-linear elastic-plastic analysis. The models are calibrated with experimental results. Final results concerning the joint characteristics will be compared with the results obtained through the Component method of EN 1993-1-8.*

1 INTRODUCTION

Modern industrial halls are made of steel pitched roof portal frames with slender sections of Class 3 and 4. The structural elements of the transverse frame have variable sections (e.g. tapered column and hunched rafters) in accordance with the stress and stiffness demand in component elements.

Since important axial compressive stresses develop in the rafter, an increased sensitivity to lateral-torsional instability characterizes the behavior of these members. If there are no lateral restrains, their lateral-torsional buckling strength is generally poor. However, the lateral restraining provided by the secondary structure and diaphragm effect of the envelope, significantly improve their response against buckling.

Due to the non-rectangular shape of the web of connected members, the knee joint detail is very specific. Usually bolted connection with extended end plate on the top or at the face of the column are used. Hereafter the case of the top connection will be examined.

A large parametrical investigation on a significant number of beam-to-column joints for pitched roof portal frames with tapered column and hunched rafter is presented in order to establish their sensitivity due to the variation of different components of the joint. Different grades of steel and thickness are used for the flange and in order to obtain sections of Class 3 and/or Class 4.

Moment capacity and stiffness of the joints are monitored by parametric study and advanced FEM non-linear elastic-plastic analysis is applied. The models are calibrated with experimental results. Final results concerning the joint characteristics are compared with results obtained through the component method of EN 1993-1-8.

2 TESTING PROGRAM

2.1 Specimens for the testing program

In order to define realistic specimen configurations, a simple pitched-roof portal frame, as the one in Figure 1, was firstly designed: span 18 m, bay 6 m, height 5 m and roof angle $\alpha=8^0$. Common load cases in the Romanian design practice were taken into consideration i.e: dead load of roof cladding 0.25 kN/m^2 (γ_{ULS}=1.35); technological load 0.20 kN/m^2 (γ_{ULS}=1.35); snow load 2.0 kN/m^2 (γ_{ULS}=1.5). S355 steel frames were analyzed and designed according to the current EN 1993-1-1 rules. Finally a number of 3 frames were obtained with different cross-section classes. The thickness, width and height of the cross-section elements were changed to obtain approximately similar stiffness and stress distribution in the frame.

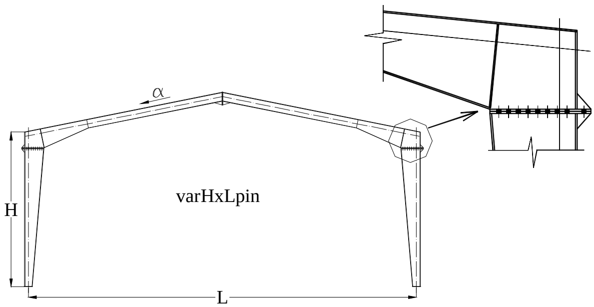


Fig. 1. Reference frame

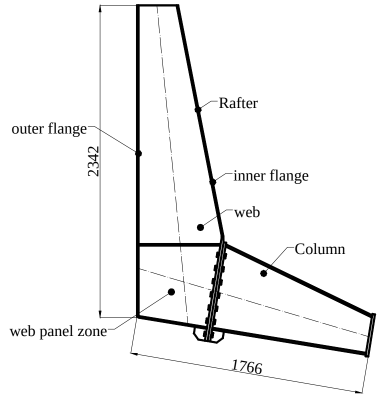


Fig. 2. Top rafter-to column joint

The three different joint configurations are: J2-3 (rafter and column of class 2 flanges and class 3 webs); J2-4 (rafter and column of class 2 flanges and class 4 webs); J3-4 (rafter and column of class 3 flanges and class 4 webs).

From design, the following joint dimensions and configurations were obtained (Table 1):

Table 1. Joint dimensions and characteristics

Joint	Column ($H*B*t_f*t_w$)	Rafter ($H*B*t_f*t_w$)
J2-3	650*240*15*8	650*200*12*8
J2-4	700*240*15*6	700*200*12*6
J3-4	700*280*12*6	700*230*10*6

where H = depth pf the section; B = width of the rafter; t_f = thickness of the flange; and t_w = thickness of the web.

Design of the joints was made using the component method in EN1993-1-8, adopted to account for significant axial force in the rafter [5]. M20-10.9 bolts and 20 mm end plates were used in all specimens.

A particular aspect of this type of joint is location of the zone of the web panel, working in shear at the end of the rafter that is bolted on the top of the column (Fig. 2).

2.2 Test set-up

Two specimens of each configuration were tested, one under monotonic and the other under cyclic loading. Figure 3 shows the loading scheme and test instrumentation.

The tests have been conducted in displacement control procedure. Lateral restraints were applied at the points indicated in Figure 3, to avoid out of plane displacement due to inherent imperfection. Load was applied quasi-statically with a displacement velocity of 3.33 mm/min.

In order to identify the material behaviour tensile test was performed on the specimens, extracted from the tested joints. The results of the tensile tests lead to the conclusion that S275 steel grade was used by fabricator instead of S355. Therefore, from this moment on, S275 steel grade was considered.

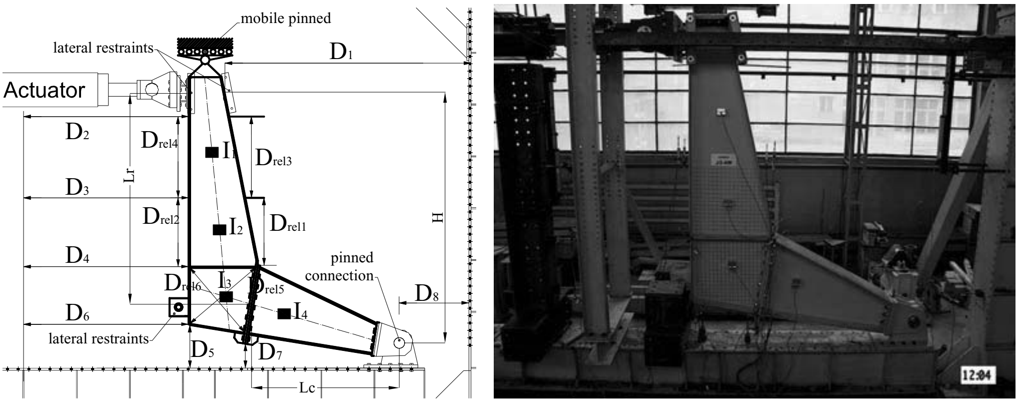


Figure 3. Loading scheme and instrumentation, where D_i = measured displacement; D_{reli} = measured relative displacement; I_i = inclinometers.

2.3 Results of the testing program

Comparative moment-rotation experimental curves for the tested specimens, under monotonic loading, are presented in Figure 4. As it can be seen, in all cases, the values of initial stiffness of the joints are very close. The failure mode is characterized by distortion of the compressed flange coupled with local buckling of the rafter web and is presented in Figure 5 for the 3 (three) different joint configurations [4].

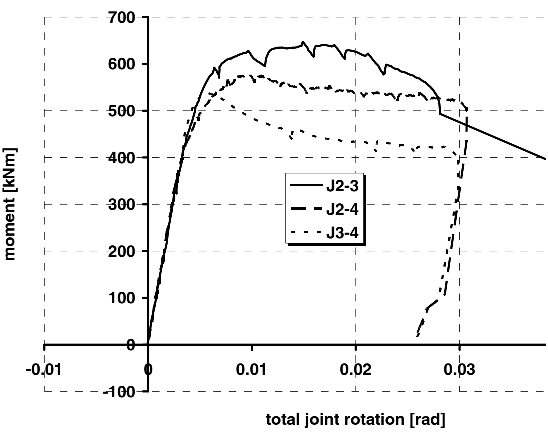


Figure 4. Comparative results from monotonic tests

Figure 5a. Failure mode of specimen J2-3m – monotonic loading

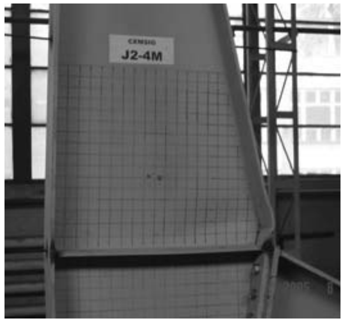


Figure 5b. Failure mode of specimen J2-4m – monotonic loading

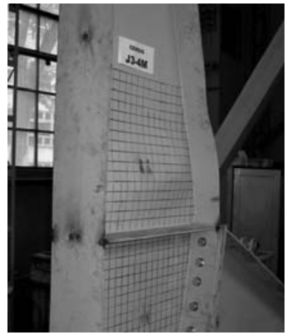


Figure 5c. Failure mode of specimen J3-4m – monotonic loading

3 NUMERICAL SIMULATION PROGRAM

3.1 FEM modeling

An advanced non-linear elastic plastic FEM model has been calibrated by using test results. With this purpose was applied Ansys computer program, using Shell 43 elements enabling for large strain plastic analysis. The initial geometrical imperfections were considered as the first eigen buckling mode of the connected elements. The material behavior was introduced by a bilinear elastic-perfectly plastic model, with a yielding limit of 275 N/mm^2. Between the end plates of the column and rafter, were used contact elements (Fig. 6a).

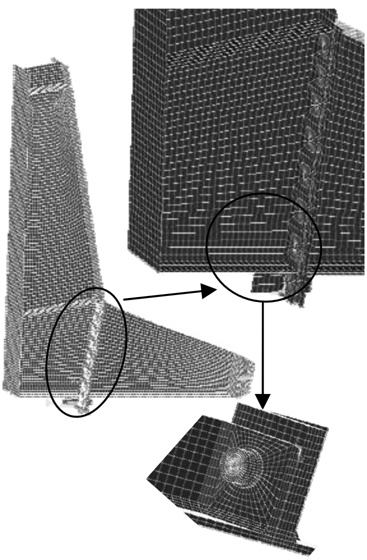


Figure 6a. FEM model

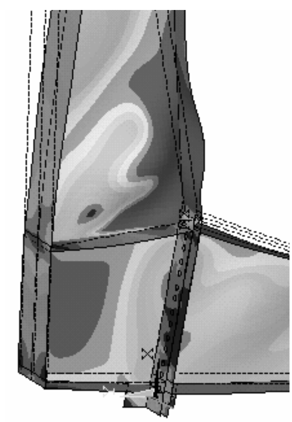
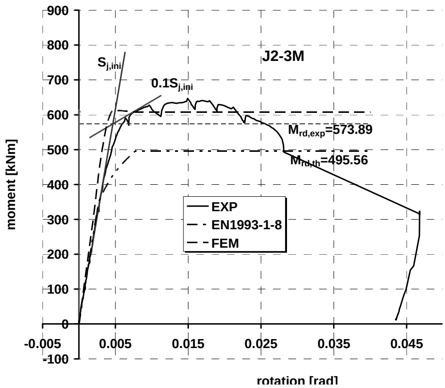


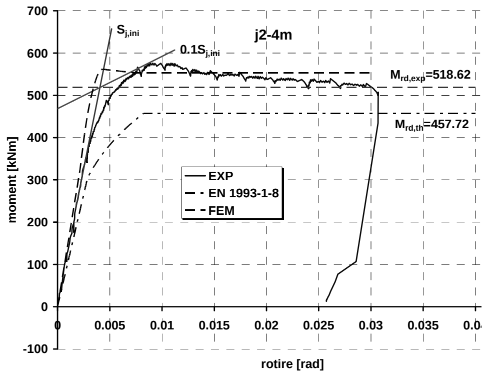
Figure 6b. Failure mode of the joint - FEM simulation

Figure 6c. Failure mode of the joint – experimental test

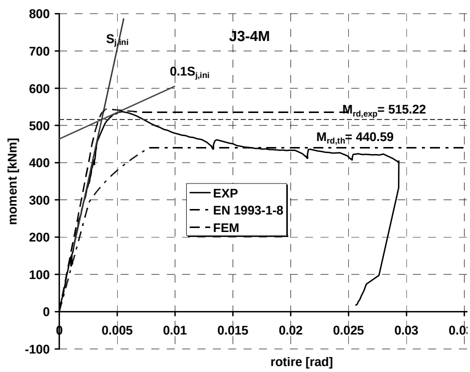
The FEM analysis qualitatively shows the location of the stress concentration and the failure modes characterizing the different joint configurations. The same failure mode of the joint was identified with FEM simulation and in case of experimental tests (Fig. 5) i.e.: distortion of the compressed flange coupled with local buckling of the rafter web. A comparison between the moment-rotation curves of experimental tests, component method [5] and FEM is illustrated in Figure 7. From Figure 7 a good similarity between experimental curves and FEM simulation can be observed, regarding the capacity of the joint and its initial stiffness.



a) J2-3M



b) J2-4M



c) J3-4M

Figure 7. Comparison between experimental curves, EN 1993-1-8 and FEM curves

3.2 Parametric study

It is very well known that experimental test, especially when dealing with big specimens, are time and labor consuming. An alternative to experimental test is represented by numerical simulations, where all the effects that might appear during a test should be taken into consideration.

Further on numerical simulation were made in order to determine the joint characteristic (moment and rotation capacity) and its behavior. The numerical simulations were made through nonlinear elastic-plastic analysis, using the same joint configurations as the ones for the experimental test (see Figs. 2, 3), but different steel grade, end plate thickness and different web thicknesses (6 or 8 mm) were used. The steel grades were S235, S355 and S460. Three different dimensions of the end plates were used i.e.: 15 mm, 20 mm and 25 mm, resulting finally a number of 36 analyzed cases. The dimensions of joint components used in the numerical simulation, for a chosen thickness of the end plate, are presented in Table 3.

In the numerical analysis was used the same static scheme as for the experimental test (see Fig. 3).

Table 3. Main dimensions of the analysed joints

Steel grade	Joint name	Column			Rafter		
		Dimension $H*b*t_f*t_w$ [mm]	Section class		Dimension $H*b*t_f*t_w$ [mm]	Section class	
			flange	web		flange	web
S235	S235_650-1	650*240*15*8	2	3	650*200*12*8	2	3
	S235_650-2	650*240*15*6	2	3	650*200*12*6	2	3
	S235_700-1	700*240*15*8	2	3	700*200*12*8	2	3
	S235_700-2	700*240*15*6	2	4	700*200*12*6	2	4
S355	S355_650-1	650*240*15*8	2	3	650*200*12*8	2	3
	S355_650-2	650*240*15*6	2	3	650*200*12*6	2	3
	S355_700-1	700*240*15*8	2	3	700*200*12*8	2	3
	S355_700-2	700*240*15*6	2	4	700*200*12*6	2	4
S460	S460_650-1	650*240*15*8	2	3	650*200*12*8	2	3
	S460_650-2	650*240*15*6	2	3	650*200*12*6	2	3
	S460_700-1	700*240*15*8	2	3	700*200*12*8	2	3
	S460_700-2	700*240*15*6	2	4	700*200*12*6	2	4

3.2 Comparative analysis

Determination of the moment capacity of the considered connections was made according to the ECCS recommended procedure [6], as the one from Figure 8a. The obtained results via FEM and numerical analysis, expressed in terms of, moment capacity ($M_{Rk,FEM}$) and moment capacity ($M_{Rk,th}$) of the joint evaluated by means of component method, are presented in Figure 9 (a, b,c) for the three material types (i.e. S235, S355, S460). If the values of moment capacity obtained by FEM simulation, $M_{Rk,FEM}$, are taken as reference, one concludes that in almost all cases the results obtained with the component method are on the safe side.

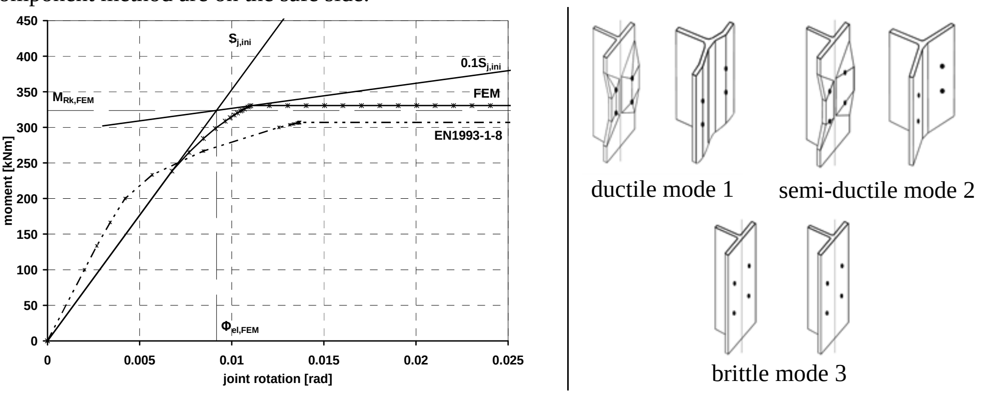
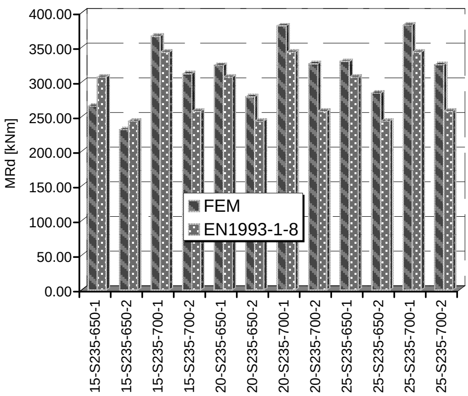
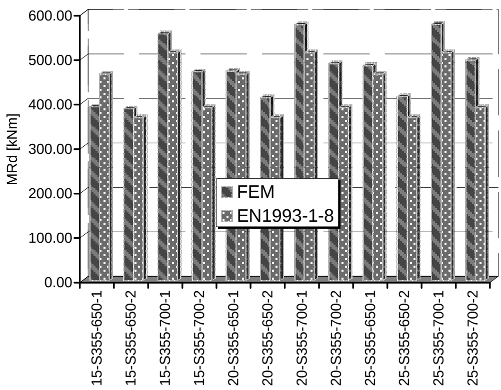


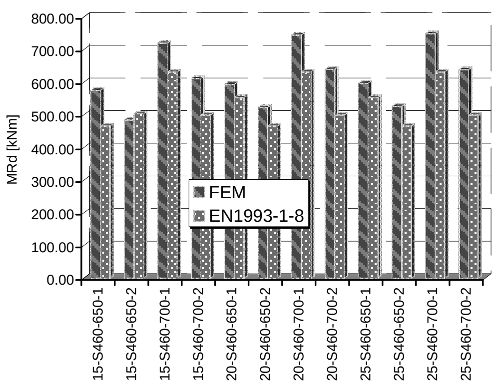
Figure 8. a) ECCS procedure for determining of $M_{Rk,FEM}$ and $\Phi_{el,FEM}$ (case of 20_S235_650-1); b) theoretical failure modes of actual components and equivalent T-stub [2]



a) S=235 N/mm^2



b) S=355 N/mm^2



c) S=460 N/mm^2

Figure 9. Comparative results $M_{Rk,FEM}$ and $M_{Rk,th}$ for different steel grades. a) S235; b) S355; c) S460

Comparative results between moment capacity evaluated with FEM ($M_{Rk,FEM}$) and the theoretical one ($M_{Rd,th}$) of the joints having a 20 mm thickness end plate, are illustrated in Figure 10.

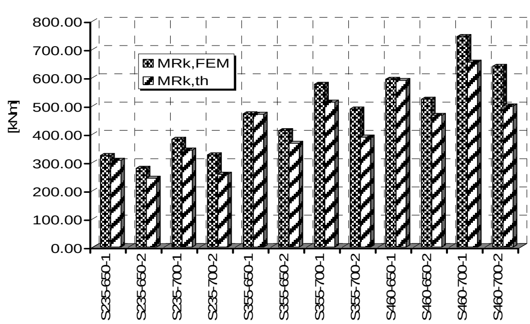


Figure 10. Comparative results (t_p=20 mm) $M_{Rk,FEM}$ and $M_{Rk,th}$

4 CONCLUSIONS

A large number of rafter-to-column joints were analysed in order to determine the influence of the changing of one of the following parameters: steel grade, web thickness, end plate thickness and height of the cross-section, all joints are of full strength and perfectly rigid. Finite element models were calibrated with experimental test carried out at the CEMSIG [4] (http://cemsig.ct.upt.ro) research centre of the Politehnica University of Timisoara. The results obtained analytically were compared, thereafter, with the ones obtained via Component Method of EN1993-1-8 [2].

The same failure mode was obtained in all cases, i. e. distortion of the compressed flange coupled with local buckling of the rafter web (see Fig. 5), even changes in height of the cross-section and thickness of the web was performed. The ductile failure mode of type 1 (Figure 8b. [2]), with the plastic bending of end plate was recorded for the 15 mm thick end plate, while semi-ductile mode 2 and the brittle one mode 3, was recorded for 20 mm and 25 mm thick end plates, respectively. However the change of the end plate thickness from 20 to 25 mm in the procedure of EN1993-1-8, doesn't influence the final capacity of the joint. This might be explained by the identified failure mode that takes place in the connected elements. For this particular detail of the joint, case of the end plate extended towards interior for head-to-head welding preparation with the rafter flange, there must be paid attention for the variation of the thickness of the end plate. The obtained results confirmed the previous observation; lower moment capacity in case of lower thickness was obtained.

For the same steel grade, the height of the cross sections has the most significant influence on the moment capacity of the joint. The initial stiffness of the joint is not significantly influenced by the chosen parameters (steel grade, end plate thickness, and cross section height and web thickness). The difference in the moment capacity of the joint, between elements of class 3 and class 4, increases by increasing the steel grade.

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