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TWO SIMPLIFIED MODELS OF COLUMN WEB PANEL IN SHEAR

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ABSTRACT

The paper presents behaviour of the column web panel in shear. The finite element method is employed to examine the resistance of a joint with an extended end plate connection. The FE model is validated to results of physical tests and verified by an analytical solution based on existing formulas. Finally, a parametric study is performed. The studied parameters include web panel thickness, bolt position and the column dimensions.

KEYWORDS

Web panel in shear, Finite element model, Parametric study, Steel joint, Component method

INTRODUCTION

Numerous studies were executed in last 20 years to analyse the bending resistance of beam-to-column joints. The component method described in EN1993-1-8:2005 [1] was developed for the beam-to-column steel joints between H and I-profiles and loaded in bending and shear. The resistances are determined for separated components. The compression side resistance of the joint is limited by the shear panel resistance. The influence of the size of the web panel to the quality of prediction by simplified models were studied by Standig [2] and Brandonisio [3]. In reality and in general description of component method the web panel and connections are separated. A simplified procedure is offered in EN 1993-1-8:2005, which is condensing the shear panel and connections into one spring. The shear resistance is described in 6.2.6.1.(2) and the acting forces in 5.3.(3). The distribution of deformations/forces to springs is modelled by β factor, which depends on loading conditions. The web panel and the connection are separated in the proposal of the second draft of EN1993-1-8:2017 [4]. Both concepts, separated and integrated, may have an influence on the joint resistance. A comparison of the results of the proposal of the second draft of EN1993-1-8:2017 with the separated solution according to EN1993-1-8:2005 with the condensed solution to the component based FEM (CBFEM) on the bending moment resistance is prepared for an unsymmetrical joint with the extended end plate connection.

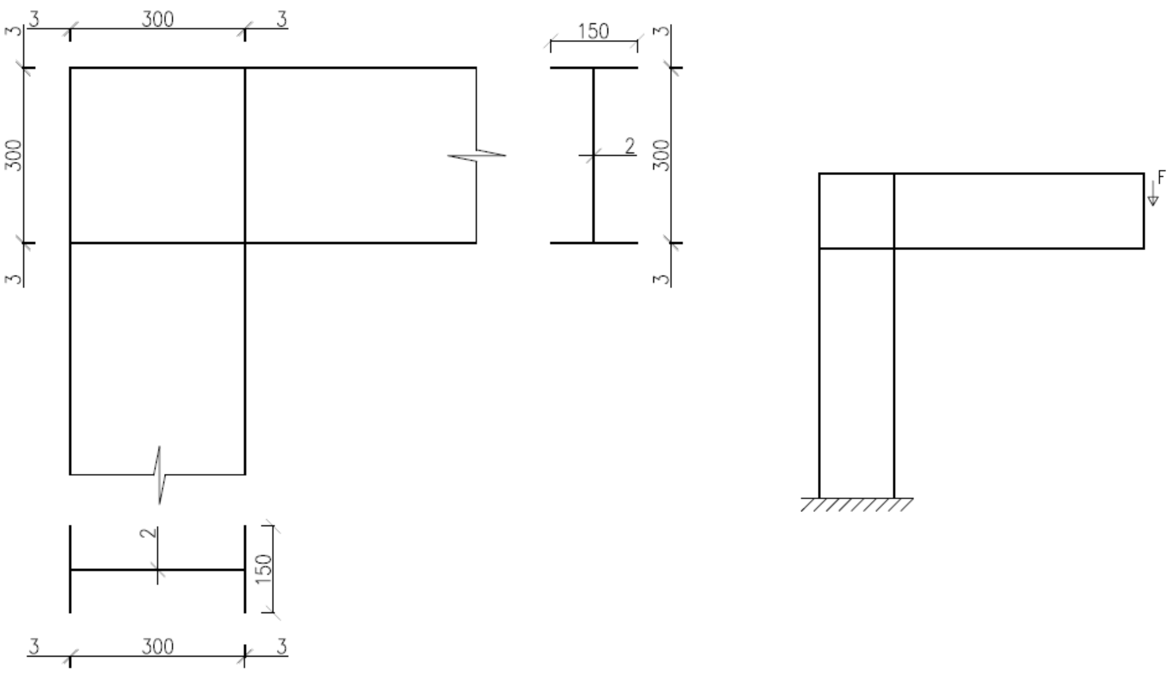
EXPERIMENTAL STUDY

Several experiments were examined to obtain the resistance of the column web panel and to verify the joint’s resistance. The influence of a high axial force is investigated in [5]. An experimental study of the web panels fabricated from high-strength steel is presented in [6] and [7].

An experimental programme done by Sheer et al. [8] is introduced. A tested specimen from the experimental programme is chosen and used in the numerical study hereafter.

Test details

An eaves moment joint loaded by concentrated force as shown in Figure 1a) was tested to study the behaviour of the column web panel in shear. The results are summarised in [8]. The specimen is made of two welded I section with the dimensions shown in Figure 1b). The height of the section is 306 mm, the column web thickness is 2 mm. The width of flanges is 150 mm and the thickness is 3 mm. The yield stress of the column web f_{yw} = 310 MPa and the yield stress of the beam and column flanges f_{yr} = f_{yc} = 232 MPa were obtained from the tensile test.



a) b)

Fig. 1 - a) Dimensions of tested joint b) Tested scheme

VALIDATION

For further investigations, RFEA is developed, see [9] and the design resistance is compared with the experiment. The numerical analyses are completed by the RFEM 5.0 (Dlubal RFEM) [10] finite element program system. The purpose of the validation is to verify the behaviour of the column web panel. A similar study referring to a design of haunches is published by authors in [11].

The materials are endowed with non-linear properties. The ideal plastic model with strain hardening is shown in Figure 2. The geometric details of the model in FEA are taken from the experimental measurement and set according to Figure 1. The welds between plates are neglected in order to simplify the FE model. The numerical model is shown in Figure 3 a). The column is fixed at the end while the beam is loaded by a concentrated force at the free edge. The beam is laterally supported to avoid the lateral torsional buckling.

In the numerical model, 4-node quadrilateral shell elements with nodes at its corners are applied with a maximum side length of 10 mm. Six degrees of freedom are in every node: 3 translations (u_x, u_y, u_z) and 3 rotations (φ_x, φ_y, φ_z). Material and geometric nonlinear analysis with imperfections (GMNIA) is applied. Equivalent geometric imperfections are derived from the first buckling mode and the amplitude is set according to Annex C EN1993-1-5:2006 [12]. Large deformation analysis is used and the Newton-Raphson method for solving systems of equations is chosen. The number of loading steps is set to 50, the convergence criteria for tolerance to 1.0% and the maximum number of iterations to 50. The first buckling mode of the model is shown in Figure 3 b).

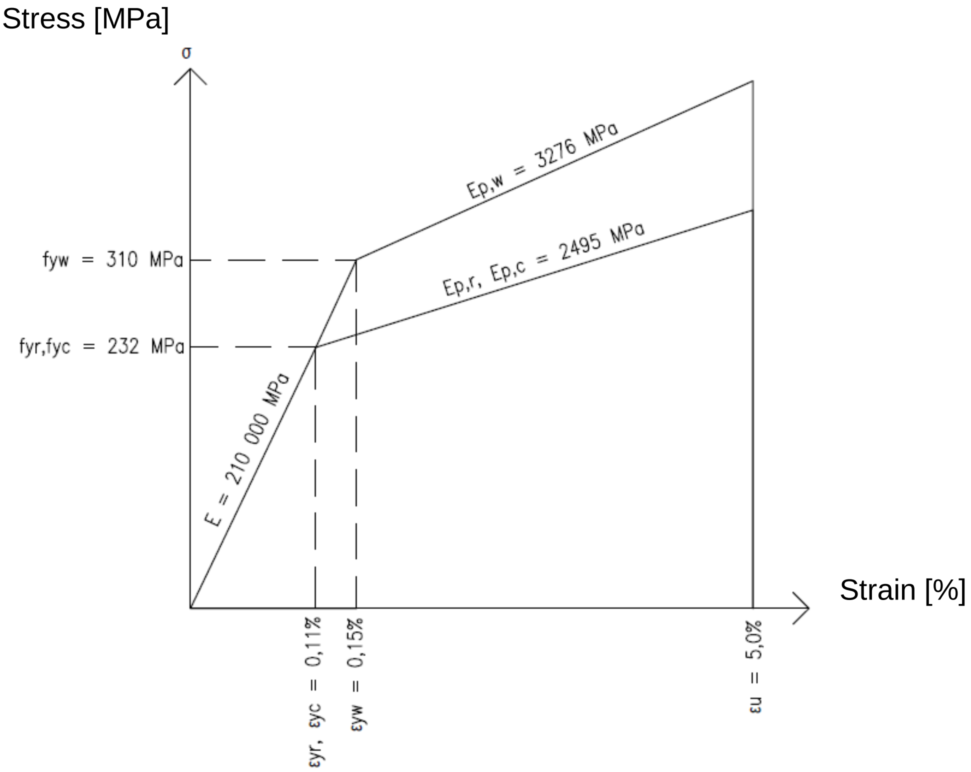
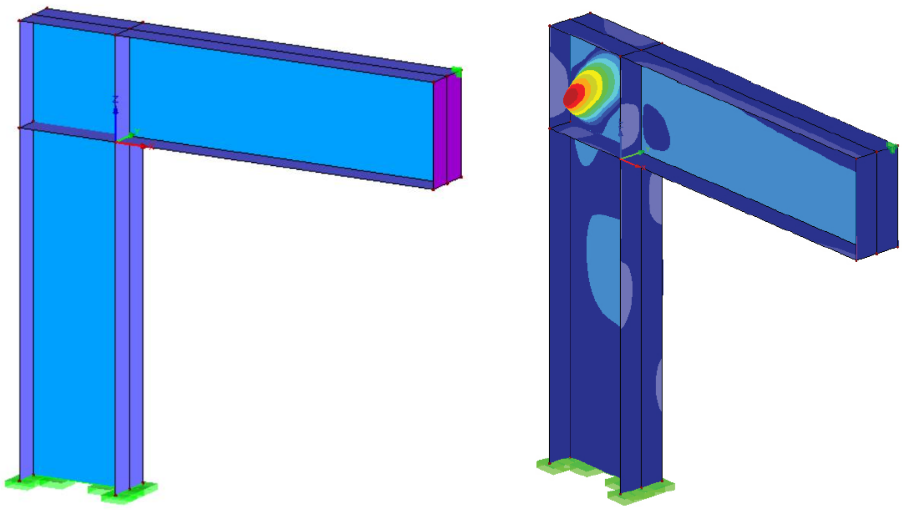


Fig. 2 - Material model used in the numerical model



a) *b)*

Fig. 3 - a) Numerical model in RFEM b) First buckling mode

The design resistance obtained in the numerical model is $M_{u,Rd}$ = 23,75 kNm. The peak load of the specimen obtained from numerical modelling is validated against the experimental results. The resistance measured in the experiment is M_{exp} = 25,6 kNm. The difference in the load is 7%, which is less than 10% and is considered as a good result. The stresses in the panel at the design resistance are shown in Figure 4.

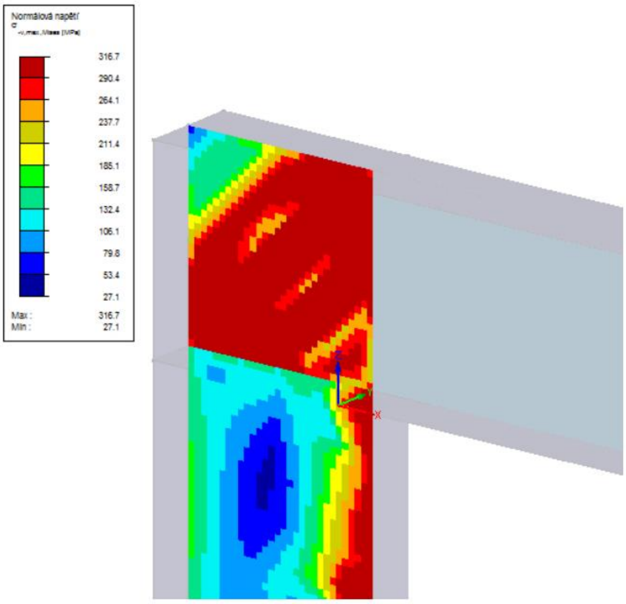


Fig. 4 - Equivalent stress in column web panel at reaching the design resistance

VERIFICATION

The design procedure for slender plates is described below. The design procedure is verified on the comparison of CBFEM and RFEM models. The buckling analysis is implemented in the software IDEA Statica Connection [13]. The design procedure for class 4 cross-sections according to reduced stress method is described in Annex B of EN1993-1-5:2006. It allows predicting the post buckling resistance of the joints. Critical buckling modes are determined by material linear and geometric nonlinear analysis. In the first step the minimum load amplifier for the design loads to reach the characteristic value of the resistance of the most critical point coefficient $\alpha_{ult,k}$ is obtained. Ultimate limit state is reached by 5 % plastic strain. The critical buckling factor α_{cr} is determined and stands for the load amplifier to reach the elastic critical load under complex stress field. The load amplifiers are related to the non-dimensional plate slenderness, which is determined as follows:

$$\bar{\lambda} = \sqrt{\frac{\alpha_{ult}}{\alpha_{cr}}} \tag{1}$$

The reduction buckling factor ρ is calculated according to EN1993-1-5:2006 Annex B. Conservatively, the lowest value from longitudinal, transverse and shear stress is taken. The verification of the plate is based on the von-Mises yield criterion and the reduced stress method. The buckling resistance is assessed as:

$$\frac{\alpha_{ult} \cdot \rho}{\gamma_{M1}} \geq 1 \tag{2}$$

where γ_{M1} is the partial safety factor.

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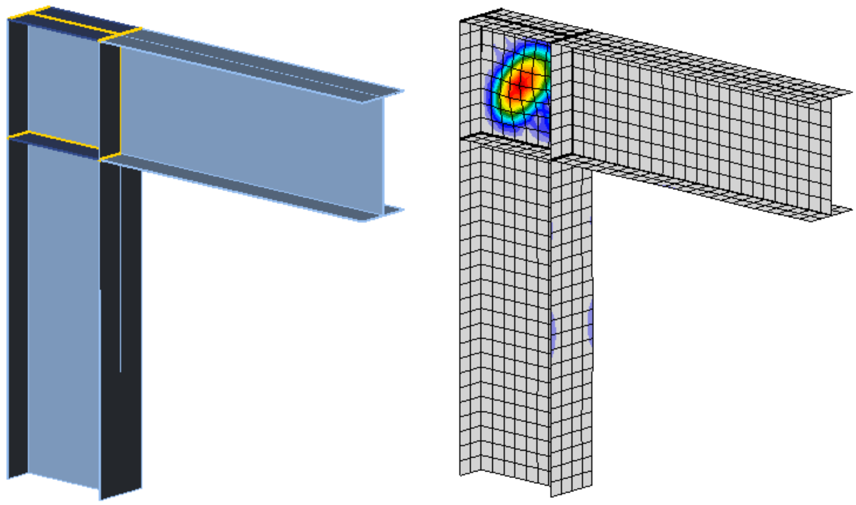
The RFEM model is compared to the CBFEM model to verify the design procedure. Compared to validated model a simplified material model is used with the yield stress f_y = 235 MPa, the tensile strength f_u = 360 MPa and Young's Modulus E = 210 GPa. The resistance of the RFEM model is M_{RFEM} = 21,9 kNm while the resistance of the CBFEM model is calculated as M_{CBFEM} = 21,0 kNm. The results show good agreement in the design resistance. The numerical model is shown in Figure 5a) and the first buckling mode is in Figure 5b).



a) *b)*

Fig. 5 - a) Numerical model in Idea Statica Connection b) First buckling mode

Sensitivity to the column size

A comparison of results of the proposal of the second draft of EN1993-1-8:2017 with the separated solution according to EN1993-1-8:2005 with the condensed solution to the component based FEM (CBFEM) on the bending moment resistance is prepared for an unsymmetrical joint with the extended end plate connection. The sum of tensile forces limited by a component (beam flange in compression, web panel in shear, column web in compression) with the lowest design resistance according to EN1993-1-8:2005 while the column web panel in shear is not considered directly in the calculation of tensile forces in the second draft and is checked separately. The geometry of the connection is taken from Example C.4 of P398 (2013) but transferred from imperial to continental cross sections IPE and HEB. Geometry of the end plate is in Figure 6, Table 1 and chapter 9.2 in [14].

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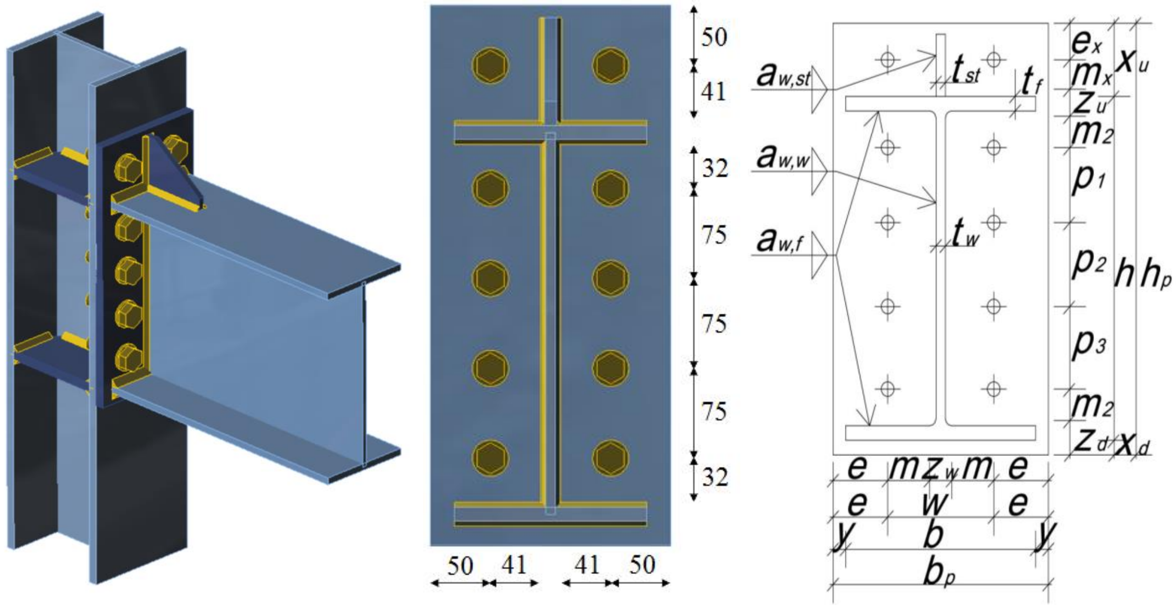

Fig. 6 - Geometry of the extended end plate joint

Tab. 1 - Geometry of the end plate

	Bolts:	M24	8.8	k_2=	0,9		
	End-plate thickness				t_p =	15	mm
1. bolt row	$a_{w,w}$ =	5	mm		z_u =	30	mm
	$a_{w,f}$ =	8	mm		p_1 =	75	mm
2. bolt row	$a_{w,st}$ =	5	mm		p_2 =	75	mm
	h_p =	450	mm				
3. bolt row	b_p =	200	mm		p_3 =	75	mm
	h =	330	mm		m_2 =	32	mm
	x_u =	100	mm		z_d =	41	mm
4. bolt row	x_d =	20	mm		m =	41	mm
	b =	160	mm		z_w =	19	mm
	y =	20	mm		t_f =	12	mm
	w =	100	mm		t_w =	8	mm
	e =	50	mm		t_{st} =	10	mm
	e_x =	50	mm		h_{st} =	90	mm
	m_x =	41	mm		n =	50	mm

The results of the extended end plate connection loaded by the bending moment are summarised in Table 2 and for loading by a proportional bending moment and shear in Table 3. The failure modes are included. The bending moment is created by the shear force on a lever arm of 1 m.

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Tab. 2: Comparison for joint loaded by bending moment

Beam IPE 330 Loaded by bending moment								
Column	CM			Difference EN:2017/ EN:2005	CBFEM		Difference CM/CBFEM	
	Resistance		Failure mode		Resistance	Failure mode		
	EN:2017	EN:2005					EN:2017	EN:2005
	kNm	kNm		%	kNm		%	%
HEB 200	110	122	CWP Sh.	9,8	110	CWP Sh.	0	-11
HEB 220	124	134	CWP Sh.	7,5	125	CWP Sh.	1	-7
HEB 240	147	153	CWP Sh.	3,9	141	CWP Sh.	-4	-9
HEB 260	164	167	CWP Sh.	1,8	150	CWP Sh.	-9	-11
HEB 280	180	180	CWP Sh.	0,0	165	CWP Sh.	-9	-9
HEB 300	189	189	BF Com.	0,0	180	BS Ten.	-5	-5
HEB 320	189	189	BF Com.	0,0	189	BF Com.	0	0
HEB 340	189	189	BF Com.	0,0	189	BF Com..	0	0
HEB 360	189	189	BF Com.	0,0	189	BF Com.	0	0
HEB 400	189	189	BF Com.	0,0	189	BF Com.	0	0
HEB 450	189	189	BF Com.	0,0	189	BF Com.	0	0
HEB 500	189	189	BF Com.	0,0	189	BF Com..	0	0

Failure modes: Column web in shear - CWP Sh., Beam flange in compression - BF Com. and Beam stiffener in tension – BS Ten.

Tab. 3 - Comparison of resistances for joint loaded by bending moment and shear

Beam IPE 330 Loaded by $M + V$								
Column	CM			Difference EN:2017/ EN:2005	CBFEM		Difference CM/CBFEM	
	Resistance		Failure mode		Resistance	Failure mode		
	EN:2017	EN:2005					EN:2017	EN:2005
	kNm	kNm		%	kNm		%	%
HEB 200	110	122	CWP Sh.	9,8	120	CWP Sh.	8	-2
HEB 220	124	134	CWP Sh.	7,5	138	CWP Sh.	10	3
HEB 240	147	153	CWP Sh.	3,9	157	BF Com.	6	3
HEB 260	164	167	CWP Sh.	1,8	171	BF Com.	4	2
HEB 280	180	180	CWP Sh.	0	190	BF Com.	5	5
HEB 300	208	208	BF Com..	0	210	BF Com.	1	1
HEB 320	223	223	BF Com..	0	228	BF Com.	2	2
HEB 340	226	226	BF Com.	0	243	BF Com.	7	7
HEB 360	229	229	BF Com..	0	250	BF Com..	8	8
HEB 400	234	234	BF Com.	0	255	BF Com.	8	8
HEB 450	241	241	BF Com..	0	263	BF Com.	8	8
HEB 500	248	248	BF Com.	0	271	BF Com.	8	8

Failure modes: Column web in shear - CWP Sh., Beam flange in compression - BF Com.

Sensitivity to the column web thickness

A parametric study of the column web thickness is presented. A welded cross-section is used for the column and the web panel thickness is changing from 5 to 20 mm. The end-plate and column geometry are summarised in Table 4. The difference from the previous example is highlighted.

Tab. 4 - The end-plate and column geometry

	Bolts	M24	8.8	k_2=	0,9			Column		
1. bolt row	End-plate thickness				t_p =	16	mm			
	$a_{w,w}$ =	5	mm		z_u =	30	mm	h =	260	mm
	$a_{w,f}$ =	8	mm		p_1 =	75	mm	b_{fl} =	260	mm
2. bolt row	$a_{w,st}$ =	5	mm		p_2 =	75	mm	t_{fl} =	16	mm
	h_p =	450	mm					t_w =	5 - 20	mm
3. bolt row	b_p =	200	mm		p_3 =	75	mm	S235		
	h =	330	mm		m_2 =	32	mm			
	x_u =	100	mm		z_d =	41	mm			
4. bolt row	x_d =	20	mm		m =	41	mm			
	b =	160	mm		z_w =	19	mm			
	y =	20	mm		t_f =	12	mm			
	w =	100	mm		t_w =	8	mm			
	e =	50	mm		t_{st} =	12	mm			
	e_x =	50	mm		h_{st} =	90	mm			
	m_x =	41	mm		n =	50	mm			

The results for the extended end plate connection loaded by proportional bending moment and shear are summarised in Table 5. Higher resistance is obtained for a column with a thin web up to 12 mm in CBFEM and in EN1993-1-8:2005. The resistance according to the second draft is underestimated. The distribution of plastic strain in the column web panel with the thickness of 8 mm in CBFEM model is shown in Figure 7.

Tab. 5 - Comparison of resistances for joint loaded by bending moment and shear

Beam IPE 330 Column stiffened thickness 5 to 20 mm (as web) Loaded by *M+V*								
Column web thickness	CM			Difference EN:2017/ EN:2005	CBFEM		Difference	
	Resistance		Failure mode		Resistance	Failure mode	CM/CBFEM	
	EN:2017	EN:2005					EN:2017	EN:2005
	kNm	kNm		%	kNm		%	%
5	79	93	CWP Sh.	15,1	98	CWP Sh.	19	5
6	90	105	CWP Sh.	14,3	110	CWP Sh.	18	5
8	111	123	CWP Sh.	9,8	133	CWP Sh.	17	8
10	131	140	CWP Sh.	6,4	152	CWP Sh.	14	8
12	152	158	CWP Sh.	3,8	168	CWP Sh.	10	6
15	183	183	CWP Sh.	0,0	189	BF Com.	3	3
16	185	185	BF Com.	0,0	189	BF Com..	2	2
20	185	185	BF Com.	0,0	189	BF Com.	2	2

Failure modes: Column web in shear CWP Sh., Beam flange in compression - BF Com.

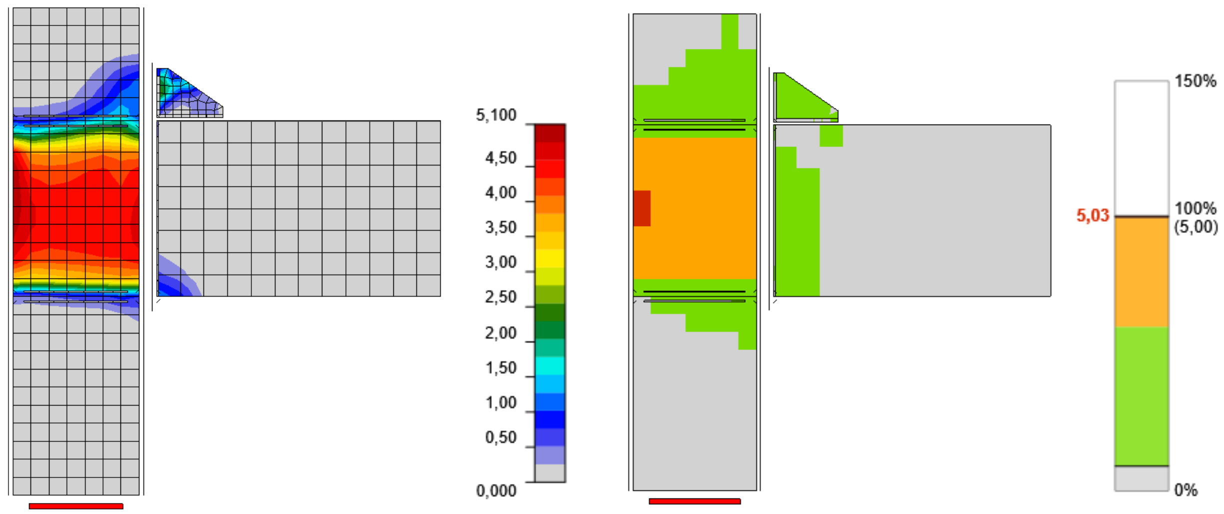

Fig. 7 - Shear distribution in shell model of FEA analysis and the strain check

CONCLUSION

The procedure to design slender plates in structural steel joints is proposed. It is possible to use material nonlinear analysis without imperfections and linear buckling analysis to design slender plates in FEM models with a complex geometry. It is proved that the results of RFEM are in good accordance with the experimental results, therefore, it can be used to predict the actual behaviour of the column web panel in structural steel joints. The proposed design procedure is verified on the RFEM.

Differences of results between prediction of bending resistance by proposal of second draft of EN1993-1-8:2017 and EN1993-1-8:2005 are in all cases up to 10 %. Higher differences are observed for the combination of bending moment and shear, especially between the second draft and CBFEM model.

The FEA with MITC4 2D shell elements (4 notes, 6 degree of freedom, Mindlin hypotheses) may be expected in this case for the column web panel loaded in shear the most accurate solution. The higher differences in prediction are in case out of a practical application. From the results is clear, that the both analytical models give different results to both sides and oscillate round the FEA results.

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