COMPONENT BASED FINITE ELEMENT METHOD FOR STEEL JOINTS AT AMBIENT AND ELEVATED TEMPERATURES

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ABSTRACT: This paper introduces Component Based Finite Element Model (CBFEM) which is a numerical design model to analyze and design connections of steel structures with features demonstrated here on a portal frame eaves bolted connection. The connection in CBFEM procedure is analyzed by Finite Element Method FEM. The correct behavior of components is treated by introducing components that well represent its behavior in terms of initial stiffness, ultimate strength and deformation capacity, of bolts, welds etc. As for another FEM design procedures a special care is given to the validation and verification procedures which is demonstrated here in this contribution on example of portal frame eaves welded connection. In this paper, CBFEM is applied to analyze steel beams and columns also at elevated temperature. The main objective of this study is to verify the CBFEM for predicting the resistance of steel members and connections at elevated temperatures.

KEYWORDS: steel structures, connection design, component method, finite element method, validation and verification.

1. INTRODUCTION

The only way to prove the accuracy of finite element analysis (FEA) simulations is through a methodical system response quantity process. Without this validation, FEA results are meaningless and cannot be used in the design process to make decisions. The system response quantity consists of validation, which compares the numerical solution with the experimental data, verification, which compares computational solutions with highly accurate analytical or numerical solution, and benchmark cases, the examples for checking the software and its user on approved and simplified input and output.

In the eighties, FEA of structural connections was treated by some researchers as a non-scientific matter. Two decades later, it was already a necessary extension of experimental and analytical work. Today, computational analysis, especially computational mechanics and fluid dynamics, is widely used as a catalyst of many research fields and as an indispensable design tool. The recommendation for design by advanced modeling in structural steel is ready for use in Chapter 5 and Annex C of EN 1993-1-5:2005 [1]. The development of modern general-purpose software and the decreasing cost of computational resources facilitate this trend. FEA of structural connections is the next step in structural steel design. As the computational tools become more readily available and easier to use even by relatively inexperienced engineers, the proper procedure should be employed when judging the results of computational analysis.

This paper describes the system response quantity for Component Based Finite Element Model (CBFEM), which is a multi-stage FEA method for analyzing and designing connections of steel structures, see [2]. The steel plates in the joint are analyzed by FEA in CBFEM procedure. The correct behavior of the components is treated by introducing the components which well represent their behavior in terms of initial stiffness, ultimate strength and deformation capacity, of bolts, welds etc. To help this process, a paper is prepared that summarizes the history of achievements of FEA application in structural connections. The article shows current trends in advanced modeling of connection components and differences between numerical simulation and numerical calculation. Special attention is paid to the design calculation of generally loaded end plates.

Experimental evidence and curve fitting procedures have been and are used for safe and economical design of connections. Based on analytical models of the resistance of joints, such as welds, bolts and plates, and the estimated lever arm of internal forces, the resistance of the joint is predicted. Zoetemeijer [3] was the first to equip this model with an estimation of stiffness and deformation capacity. The elastic stiffness was improved in the work of Steenhius, see [4]. The basic description of the behavior of components in major steel connections was prepared by Jaspart for beam-column connections [5] and by Wald et al. for column bases [6]. The method implemented in the current European structural standard for steel and composite connections, see [7] and [8], is used in the majority of structural steel software used in Europe. The idea was generalized by da Silva [9] for 3D behavior including nonlinear parts of the behavior. The procedure starts with the decomposition of a connection into components, followed by their description in terms of normal/shear force deformation behavior. The components are then grouped to study the joint moment-torsional behavior and classification/representation in a spring/shear model and application in global analyses. The advantage of this often called Component Method (CM) is the integration of current experimental and analytical knowledge of the behavior of fasteners, bolts, welds and plates. This provides a very accurate prediction of the behavior in elastic and ultimate loading levels. Verification of the model is possible through simplified calculation. The disadvantage of CM is that experimental evaluation of internal force distribution is available only for a limited number of open section joint configurations. In temporary scientific papers, description of atypical components is either not present or has low validity and description of background materials. The CM's are not developed for manual calculation, but as a method for preparation of design tables or software tools. Models of hollow section connections are described in Chapter 7 of EN1993-1-8 [7] by means of curve fitting procedures based on mechanical and numerical experiments. Their component representation is prepared according to the curve fitting procedures available in 2012 [10], based on the selection of appropriate level arms and effective widths.

The global analysis of steel structures today is carried out by FEA and all the traditional procedures are not used any more (such as force method, three moment equation, Cremon's pattern, the cross method or the method of distribution moments). In the current rapid development of software capability connections ready to be designed by FEA and thousands of experiments the validation process is available. In such situation the verification process performed by benchmark tests gains crucial importance. The source and extent of such benchmark tests for the field of structural connections is yet to be established. To achieve this goal, a set of small benchmark tests has been developed that can be used as a reference in the verification process of simulations [11].

In fire, the stiffness and strength of steel members are significantly reduced at elevated temperatures, resulting in a reduction of the ultimate load capacity. EN 1993-1-2 [24] suggests that the design of isolated steel members exposed to fire, assuming a uniform temperature in the section, can be analyzed using simplified analytical methods, taking into account the mechanical properties of steel at elevated temperatures. When designing a steel member, ambient temperature actions with reduction factors for the mechanical properties of structural steel at elevated temperatures can be used to consider the influence of fire action [25]. Experimental investigation is the most accurate way to calculate the fire resistance of steel members. However, it is not applicable due to the cost and size limitations of currently used furnaces. Many models can be used to calculate the fire moment resistance of steel beams. They are analytical, numerical and CBFEM models. The solid models can provide high accuracy compared to experimental results. On the other hand, the main disadvantage of solid models is the computation time. Therefore, shell elements are a very good alternative to solid models. The CBFEM simulates the behavior of steel plates using shell element and evaluates the design parameters based on different design specifications. CBFEM is the most widely used method to analyze and design connections of steel structures. It is the combination of analytical component method and numerical finite element method (FEM). FEM is used to solve the distribution of internal forces. The plates are modeled using 4-node quadrilateral shell elements.

In this paper, CBFEM is used to analyze steel beams and columns at elevated temperatures. EN 1993-1-2 [24] suggests that the design of isolated steel members exposed to fire, assuming a uniform temperature in the section and considering the mechanical properties of steel at elevated temperatures, can be analyzed using simplified analytical methods. Simple calculation models are used to easily design individual members under conservative assumptions. In this paper, a benchmark study for steel beams and columns exposed to fire is prepared using simple models to provide safe design values for designers. The results obtained from the analytical model are used to verify the results of CBFEM. Parametric studies are performed by changing some parameters in CBFEM to provide benchmark studies. Length, section type, boundary conditions and loading conditions are the changing parameters in the study.

2. VALIDATION AND VERIFICATION

FEA for connections has been used since the 70s of the last century as a numerical simulation. Its ability to express real behavior of connections makes numerical experiments a valid alternative to testing and source of additional information about local stresses. Validation and verification (V&V) process of models is an integral part of the procedure, see e.g. [12], and the studied FEA are based on the researcher's own experiments. During the preparation of the Component Method (CM) for EN1998-1-3:2006, all basic components were modeled in detail, see [13]. Special attention was paid to the modeling of the T-junction, which represents the connections between the end plates of the beams and the columns, the connections between the beams and the bases of the columns [14]. The latest generation of FEA models of joints is used in studies focused on the application of high strength steel in joints [15] and bolts [16]. Prediction of hollow section connections is based on experimental evidence confirmed by numerical FEA experiments, see e.g. [17]. Due to the large variety of geometries, some types have been studied only numerically, see [18].

V&V of FEA numerical simulation of steel connections design is native part of its preparation, see [19]. The detailed procedure for verification of CBFEM and its application in the design tool has been prepared, see [11]. The procedure consists in preparation of benchmark studies for used components, e.g. bolts, welds, slender plates in compression, anchor bolts and concrete block in compression. Three different types of welded connections were selected for benchmark studies: connections loaded in shear, connections loaded in flexure, and connections welded to a flexible plate. For bolted connections, benchmark studies are prepared for T-joints in tension, the joints in shear, and the generally loaded end plate connection, see Figure 1. For slender plate in compression, the triangular haunch in compression, the slender stiffener of the column web, and the plate in compression between bolts are studied. For hollow section connections, the welded connections between CHS or RHS members and RHS/CHS diagonals welded to open section chords in the form of T, K and TT connections are studied. For column bases, verifications are prepared for T-joints in tension and compression and for generally loaded columns of open and hollow sections. Benchmark study consists of description of selected connection, results of CBFEM and CM, differences described in terms of global behavior on force-deformation/rotation curve and verification of initial stiffness, resistance, deformation capacity. At the end of each benchmark study, a benchmark case is prepared to allow the user to check his results. In some cases, the CBFEM method gives higher resistance, initial stiffness or deformation capacity. In these cases, an advanced FEM model of the brick element validated on own or from literature experiments is used to obtain proper results. CBFEM is approved by this procedure.

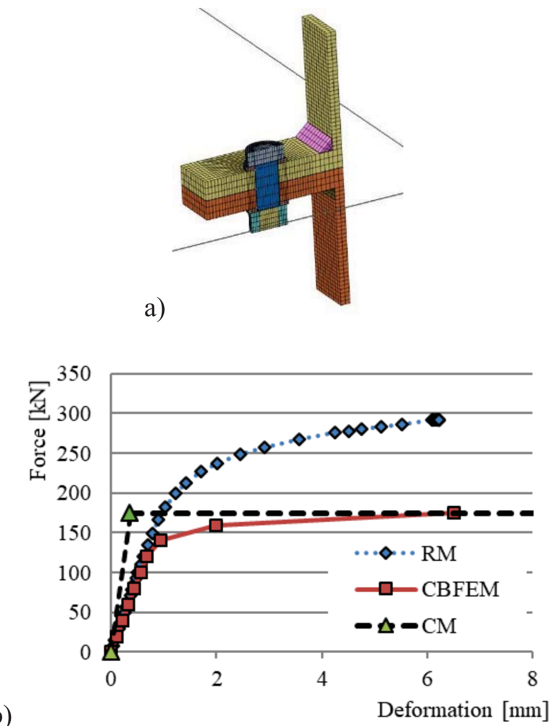

Figure 1. The bolted T stub in tension b) mesh for numerical simulation a) force-deformation diagram

The numerical simulation of the bolted T stubs in tension was prepared Midas FEA code, see Figure 1a, and validated on experiments [20]. T-stub of steel S235, with flange thickness t_f = 20 mm, web thickness t_w = 20 mm, flange width b_f = 300 mm, length b = 100 mm, double fillet weld a_w = 10 mm, bolts 2 x M24 8.8 with pitch w = 165 mm was modelled and selected for sensitivity study. The numerical simulation using the true stress true stain material diagram represents the experimental behaviour was used for verification of the CBFEM model of T stub, see Figure 1b. For thin plates gives CM unrealistic low value due to neglecting of membrane action of end plates, see Figure *2*.

For regular plates predict CM higher resistance by neglecting the shear and bending interaction on deformed endplate. For very thick plates is for CM calculated the limit of deformation capacity separately and its estimation may not fit into shear and tension interaction in bolt. The sensitivity study of flange thickness width and material quality, bolt size, pitch and material quality, shows good prediction resistance by CBFEM on asked design level. Summary of verification of CBFEM to CM for the bolted T stub in tension is presented in Figure *3,* where are recapitulated the studies for bolt size, material and pitches, the flange thickness and width. The results show that the difference of the two calculation methods is mostly up to 10 %. In cases with CBFEM/CM > 1.1 accuracy of CBFEM was verified by the results of RM which gives highest resistance in all selected cases.

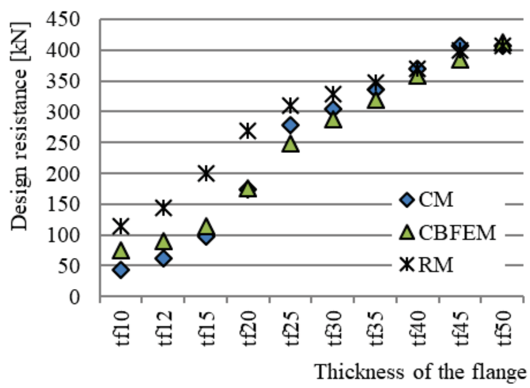

Figure 2. Sensitivity study of flange thickness

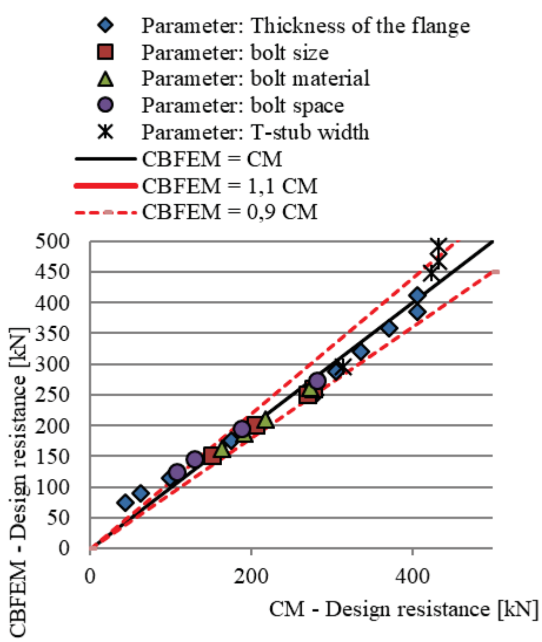

Figure 3. Summary of verification of CBFEM to CM for the bolted T stub in tension

Experimental investigation of three samples of the generally loaded end plate joints was performed [21]. End plates were welded on two RHS 250x150x16 beams of different lengths 2000 mm and 1000 mm. The beams and plates were designed from S355, with measured values of $f_{y,m}$ = 410 MPa and $f_{u\,m}$ = 582 MPa. The end plates P10 – 400 x 300 were connected by M20 8.8 bolts, with the vertical distances 35 – 230 – 100 - 35 mm and horizontal ones 30 – 240 – 30 mm. The beam with connection 500 mm from its centre was loaded in its centre through P20 by hydraulic jack, see Figure 4. The configuration creates in the connection shear forces and bending moments. The results of the contact imprints on paper placed between the end plates is included on right side of the Figure 1b, see [12]. The inclination of the specimens varied from 0°; 30° till 45°. The test set up with 0° inclination is documented at Figure 5.

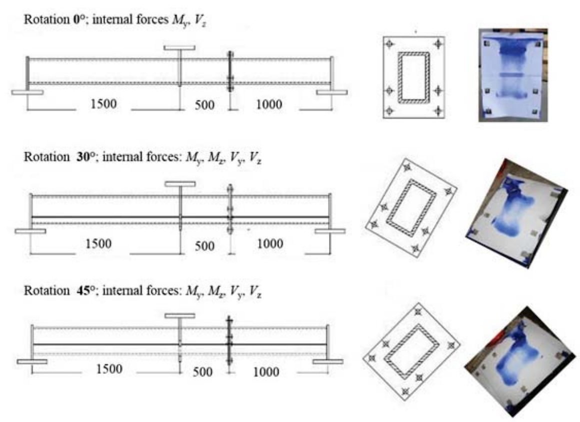

Figure 4. Position of the beam splice joins on beam, inclination and contact imprints.

Figure 5. The test sample with 0° inclination

Connections were designed according to EN 1993-1-8:2006 [7]. Four components are guiding the behaviour the fillet welds, the beam flange in compression and in tension, the end plate in bending and the bolts in tension. Effective lengths for circular and noncircular failures are considered according to EN 1993-1-8:2006 cl. 6.2.6. Three modes of collapse according to EN 1993-1-8:2006 cl. 6.2.4.1 are considered. Bolts are designed according to cl. 3.6.1 in EN1993-1-8:2006. Design resistance considers punching shear resistance and rupture of the bolt. For component method is in EN1993-1-8:2006 recommended a linear interaction. The quadratic interaction curve according to [22] is included in verification study.

Samples 30° and 45° with strong axis bending moment were chosen to present of the global behaviour described by moment-rotation diagram, see Figs 6 and 7. CM with quadratic interaction gives higher initial stiffness compared to CBFEM and experimental data. In all cases are resistances by CM and CBFEM similar and corresponds to asked characteristic design level. Experimentally measured resistance is higher including hardening of the materials after reaching yield strength. Resistance calculated by CBFEM was compared with the results of CM and experimental results, see Figure 8. CM with linear interaction gives conservative values of resistance. CM with quadratic interaction gives the highest resistances, which are to experimental results still rather conservative. CBFEM gives similar results as CM with quadratic interaction. The verification of the prediction of the resistance of the CBFEM to CM for inclination of°0° and changing the end plate thickness is presented in Figure 9 and Table 1. The results shows good agreement between both models. The verification of the prediction of the resistance of the CBFEM to CM for inclination of°0° and changing the bolt material is presented in Table 2. The results presents good agreement between both models.

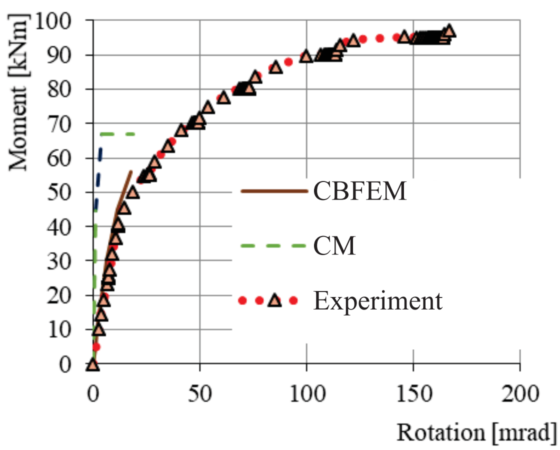

Figure 6. Validation of moment rotational curve of numerical model (CBFEM) and analytical model (CM) to experiments for inclination 30°

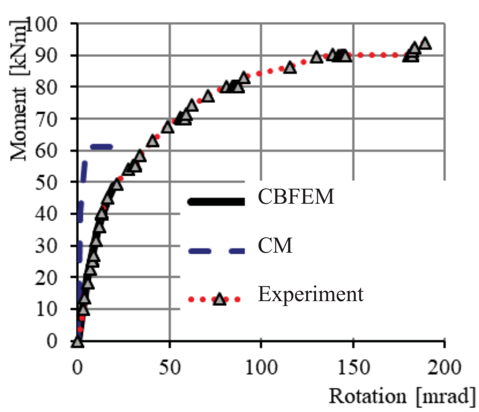

Figure 7. Validation of moment rotational curve of numerical model (CBFEM) and analytical model (CM) to experiments for inclination 45°

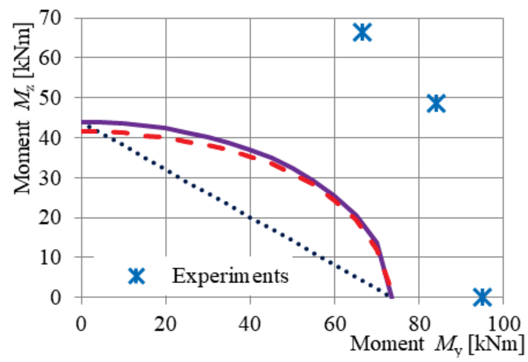

Figure 8. Validation of resistances for numerical model (CBFEM) and analytical model (CM) to experiments

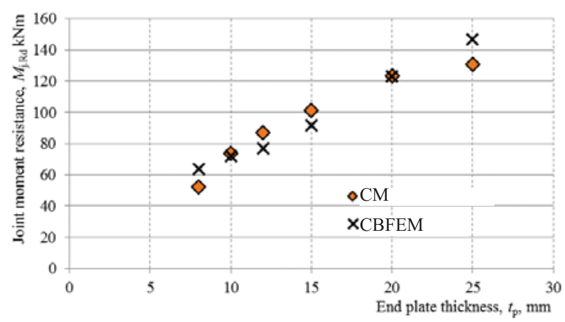

Figure 9. Verification of numerical model (CBFEM) to analytical model (CM) for the end plate thickness

Table 1. Resistance values and failure modes of CBFEM by CM by changing plate thickness

t_p, mm	CBFEM Res. kNm	CBFEM Failure	CM Res. kNm	CM Failure	CBFEM/ CM
8	64	End pl.	52	End pl.	1,22
10	72	End pl.	74	End pl.	0,98
12	77	End pl.	87	End pl.	0,88
15	92	End pl.	101	End pl.	0,91
20	123	End pl.	124	End pl.	1,00
25	147	Bolts	131	Bolts	1,12

Table 2. Resistance values and failure modes of CBFEM by CM by changing bolts 8.8

Bolt.	CBFEM		CM		CBFEM/ CM
	Res. kNm	Failure	Res. kNm	Failure	
4.8	49	End pl.	48	End pl.	1,01
5.8	50	End pl.	56	End pl.	0,89
6.8	55	End pl.	64	End pl.	0,86
8.8	72	End pl.	74	End pl.	0,98
10.9	75	End pl.	79	End pl.	0,94

3. STRAIN FOR LIMITATION OF PLATE RESISTANCE

Strain is in cl. C.8(1) EN 1993-1-5 [1] recommended to limit to 5 % to reach the design resistance based on studies of the plated structures resistance. The studies of the limiting strain of plates in structural steel connections, see [11, 23], shows similar results and acceptable 5 % limit. By CM is expected linear behaviour till 2/3 of the joint design resistance, based on the plate elastic and plastic resistance, see [2]. In structural connections analysed by FEA reaches the steel elastic behaviour at about 30 % till 50 % of resistance in case of the estimated ideal elastic and plastic steel material model. The moment rotation/force deformation curve, which represents the behaviour begins to curve at 70 % of resistance with about 1 % strain. From strain 3 % till 8 % starts the almost horizontal part of the curve and the differences in resistance are coming negligible. The theory of small deflections used in FEA analyses gives according to number of integrations.

The example of limiting 5 % stain is presented for complex industrial joint form steel S235, see Figure 11a), where is column of cross-section HEB600. The horizontal beams HEA180 are loaded at design resistance in tension 8,9 kN, 106,7 kN, and 114,3 kN, the inclined front beams HEB240 are loaded in tension 1016,0 kN, 323,9 kN, and 53,3 kN, and the inclined rear beam HEB200 in tension 53,3 kN. Development of plastic stains load in the column web related to applied is presented in Figure 12, where in Figure 11b) is 0,2 % stain at 47 % of design resistance, in Figure 11c) 0,9 % at 79 %; in Figure 11d) 4,0 % at 96 %; in Figure 11e) 5,0 % at 100 %, and in Figure 11f) 6,0 % at 102 %.

4. CONNECTION FIRE DESIGN

4.1. Analytical Model

There is no formulation or guidance for the calculation of the elastic critical moment M_{cr} in

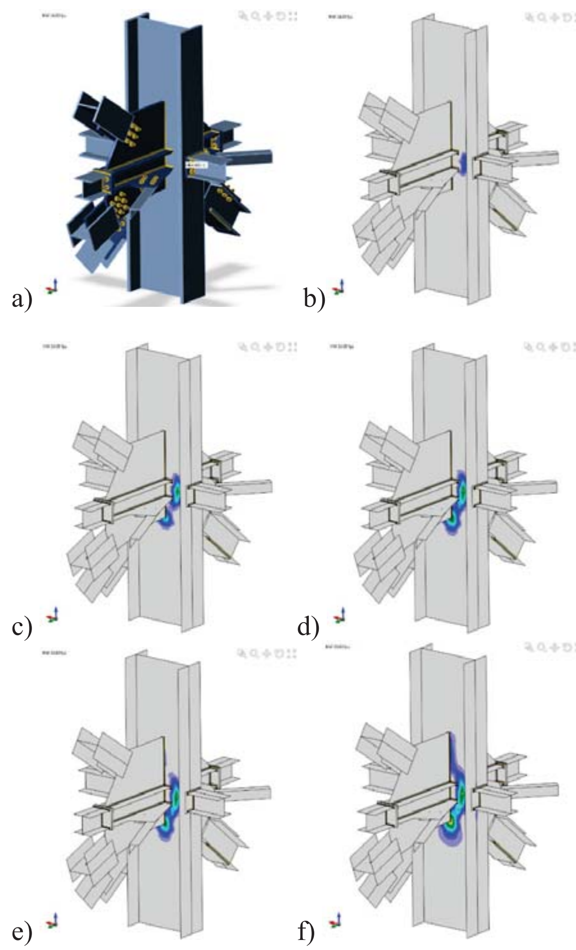

Figure 10. Development of plastic stains in the column web of the complex joint a) visualisation, b) 0,2 % stain at 47 % design resistance; c) 0,9 % at 79 %; d) 4,0 % at 96 %; e) 5,0 % at 100 %; f) 6,0 % at 102 %

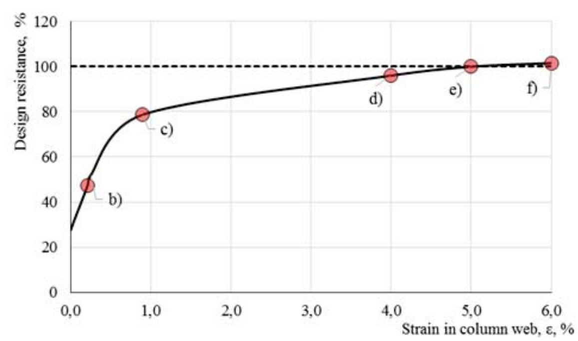

Figure 11. Percentage of design resistance and strain in the column web diagram with marked points shown at Figure 11

Clause 6.3.2.2(2) of EN 1993-1-1:2005 [26]. A general expression proposed by NCCI SN003 [27] is used to calculate the elastic critical moment for lateral-torsional buckling considering the shape of the bending moment diagram, different end restraint conditions, warping restraints, in-plane curvature before buckling, and the level at which the load is applied. The detailed calculation method is explained in the study [28].

According to clause 4.2.3.3 in EN 1993-1-2:2005 [26], the design lateral-torsional buckling resistance moment $M_{b,fi,t,Rd}$ at time t of a laterally unrestrained member with a Class 1 or Class 2 cross-section should be determined from:

$$M_{b,fi,t,Rd} = \chi_{LT,fi} W_{pl,y} \frac{k_{y,\theta com} f_y}{\gamma_{M1}} \quad (1)$$

where $W_{pl,y}$ is the plastic section modulus of cross-section, $k_{y,\theta,com}$ is the reduction factor for the yield strength of steel considering the maximum temperature in the compression flange θ_{com} reached at time t, and $\chi_{LT,fi}$ is the reduction factor for lateral-torsional buckling in the fire design situation, which is calculated using the following equation:

$$\chi_{LT,fi} = \frac{1}{\phi_{LT,\theta,com} + \sqrt{\left[\phi_{LT,\theta,com}\right]^2 - \left[\lambda_{LT,\theta,com}\right]^2}} \quad (2)$$

with

$$\phi_{LT,\theta,com} = \frac{1}{2}[1 + \alpha\bar{\lambda}_{LT,\theta,com} + \left(\bar{\lambda}_{LT,\theta com}\right)^2 \quad (3)$$

and the imperfection factor α is given by

$$\alpha = 0.65\sqrt{\frac{235}{f_y}} \quad (4)$$

and, for Class 1, 2 and 3 the non-dimensional elevated temperature LTB slenderness of a steel beam $\bar{\lambda}_{LT,\theta,com}$ is determined through the following expression

$$\bar{\lambda}_{LT,\theta,com} = \bar{\lambda}_{LT}\sqrt{k_{y,\theta,com} / k_{E,\theta,com}} \quad (5)$$

in which $k_{E,\theta,com}$ is the modulus of elasticity reduction factor at the maximum steel temperature in the compression flange,

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_y f_y}{M_{cr}}} \quad (6)$$

The design buckling resistance $N_{b,fi,t,Rd}$ at time t of a compression member with Class 1, Class 2 or Class 3 cross section and uniform temperature θ_a can be determined from clause 6.3.1 of EN1993-1-1:2005 [26] as

$$N_{b,fi,t,Rd} = \chi_{fi} A k_{y,} f_y /_{M,fi} \quad (7)$$

where χ_{fi} is the reduction factor for flexural buckling in the fire design situation; $k_{y,\theta}$ is the reduction factor for yield strength of steel at temperature θ_a. The minimum value of $\chi_{y,fi}$ and $\chi_{z,fi}$ can be taken as the value of χ_{fi}.

$$\chi_{\text{fi}} = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \quad \text{but } \chi \le 1.0 \quad (8)$$

with

$$\Phi = 0.5\left[1 + \alpha\bar{\lambda}_\theta + \bar{\lambda}^2\right] \quad (9)$$

and for all steel grades $\alpha = 0.65\sqrt{235 / f_y}$, where f_y is the yield strength of steel at ambient temperature. The relative slenderness $\bar{\bar{\lambda}}_\theta$ at temperature θ_a is calculated by

$$\bar{\lambda}_\theta = \bar{\lambda}\sqrt{k_{y,} / k_{E,}} \quad (10)$$

in which $k_{E,\theta}$ is the reduction factor for modulus of elasticity of steel at temperature θ_a and $\bar{\lambda}$ is the non-dimensional slenderness at room temperature given by the following equations using the buckling length in fire situation l_{fi}. The non-dimensional slenderness at room temperature $\bar{\lambda}$ is given by Eq. 11

$$\bar{\lambda} = \sqrt{\frac{Af_y}{N_{cr}}} \text{ for Class 1, 2 and 3 cross sections} \quad (11)$$

where N_{cr} is the elastic critical force for flexural buckling based on the gross cross-sectional properties and in the buckling length in fire situation l_{fi}.

$$N_{cr} = \frac{\pi^2 EI}{l_{fi}^2} \quad (12)$$

where E is the Young's modulus at room temperature, I is the second moment of area about y-y or x-x axis based on the gross cross-sectional properties and l_{fi} is the buckling length in fire situation.

4.2. Numerical Calculation

Four-node quadrangle shell elements with nodes at its corners are used to simulate plates. The material behaviour is based on Von Mises yield criterion. It is assumed to be elastic before reaching the yield strength . The value of 5 % plastic limit strain is recommended for predicting the resistance. The uniform temperature distribution is applied to each member in the study. The numerical model can predict the resistance at target temperature by user defined. Numerical calculation model [29] is used to prepare the numerical calculation.

4.3. Linear Buckling Analysis (LBA)

LBA checks buckling mode shapes directly in a 3D view. It displays critical load factors and amplitudes set the initial imperfection value. LBA can be used to determine critical load factor. The structure is considered perfect without any geometrical or material imperfections, and the material is elastic in this analysis type. Linear buckling analysis factor α_{cr} is the minimum amplifier for design loads to reach the elastic critical resistance of the structural component. The buckling mode shape also provides designer with information on whether the member fails in flexural buckling around weaker or stronger axis, torsional buckling (axially loaded columns) or lateral-torsional buckling, bent beams, or local buckling, members with slender plates.

4.3. Geometrically and Materially Nonlinear Analysis with Imperfections (GMNIA)

GMNIA provides designer geometrically and materially nonlinear analysis and checks steel structure with initial imperfection definitions. Analysis considers imperfections set in pre-vious step (LBA) together with material and geometrical nonlinear behaviour. Fire design of steel member takes into consideration the degradation of materials according to user-set temperatures during the analysis. Geometrically and materially nonlinear analysis with im-perfections is the most sophisticated analysis type for static loading. All the imperfections (varying thickness of plates, out-of-straightness, residual stresses, non-homogeneities in material, misalignment of supports...) are substituted by equivalent geometrical imperfections and can be set using buckling mode shapes calculated by LBA. Designer may select the maximum amplitude of the buckling mode shape used for imperfection.

4.4. Benchmark Study Steel Beam

The study includes a beam from the IPE300 section with a span of L = 6 m, which is loaded by concentrated force at the midspan at the top flange as shown in Fig 12. The beam is assumed that it has a uniform temperature along the cross-section and length of the beam. Elas-tic modulus and yield strength of steel beam at room temperature are taken as 210000 N/mm² and 355 N/mm², respectively. The analysed beam is designed with connection and related members. Therefore, the connection does not behave as full pinned support and contributes to the moment resistance slightly.

Figure 13 indicates the elastic critical moment values of the benchmark studied using analytical model, shell model generated in ABAQUS [7] and CBFEM. The CBFEM predicts the resistance similar to analytical model. At elevated temperature the resistance obtained from CBFEM is slightly higher than analytical model due to the connection model. Analytical model (AM) assumes the connection fully pinned support whereas the realistic connection model in the numerical calculation provides additional rotational stiffness.

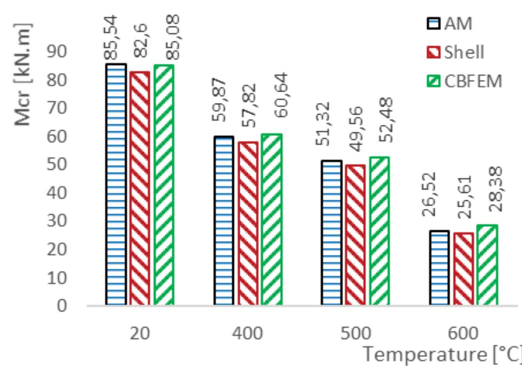

Figure 13. The elastic critical moment of the studied beam

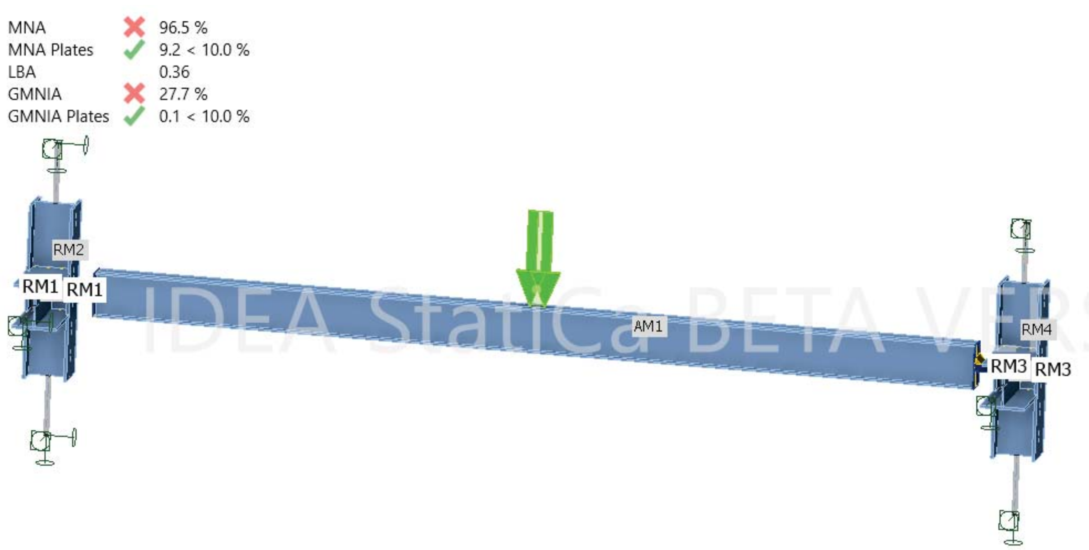

Figure 12. The configuration of the studied steel beam

Table 3. The lateral torsional buckling moment – Cross section

θ (°C)	IPE 300 - 6 m simply supported - point load		
	AM	Shell	CBFEM
20	85.54	82.6	85.08
400	59.87	57.82	60.64
500	51.32	49.56	52.48
600	26.52	25.61	28.38

Table 4. The lateral torsional buckling moment – boundary conditions

θ (°C)	IPE 300 - 6 m fix supported - point load		
	AM	Shell	CBFEM
20	138.68	124.37	124.39
400	97.08	87.06	87.49
500	83.21	74.62	75.11
600	42.99	38.55	39

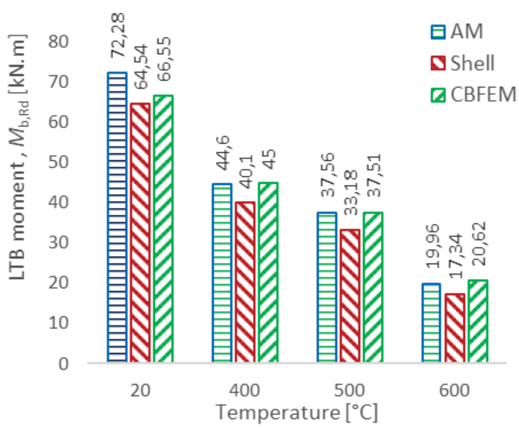

Figure 14. The lateral torsional buckling moment of the studied beam

Tables 1-3 presents the moment resistance of lateral torsional buckling (LTB) for steel beams at elevated temperature. The changing parameters are mentioned in tables. The LTB moment of the studied beam at ambient and elevated temperature from 400° to 600 °C can be seen in Figure 14. Generally, the CBFEM provides accurate resistance comparing to analytical model with a maximum 10 % difference.

6. SUMMARY

The numerical calculation of steel connections replaces the curve fitting and component design methods. For its proper use it is necessary to apply a good validation and verification procedure with well-defined hierarchy to allow a safe use and to prepare benchmark studies for its proper use.

The presented results show the good accuracy of the numerical simulation models verified to the analytical ones and to the numerical simulations/ experiments in cases where the numerical model gives higher stiffness, resistance, or deformation capacity, see [11].

The strain limits must be studied based on the safety of numerical applications. For even complex joint is presented here that 5% leads to good and safe prediction.

The comparison between the numerical calculation and the analytical model of members at elevated temperature during fire showed generally small deviations in the LBA and GMNIA results, with a maximum difference of 10% in individual cases. The verified numerical calculation can be used for structural fire engineering at the design level of members and their connections.

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