Strength of Fillet-welded Joint Connections: Comments on Correlation Between Classical and Particular Finite Element Approach

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The paper deals with strength of load-carrying fillet welds with application of two different approaches. First one is calculation by classical/scholar approach while second one is devoted to finite element analysis. The classical approach is concerned with national and European engineering practice. The finite element approach includes the application of particular tool which preserves main recommendations from modern postulations in design of joints. The object of interest is welded beam-to-column joint with different structural elements and the stresses are obtained for two models. It is investigated correlation of results of weld stresses from both the approaches. Direct matching of results was neither expected nor found but basic correlation is revealed within the joint behaviour under loadings. Considering finite element approach as prevailing, its advantage is clearly shown throughout inclusion of local effects of plates. However, classical approach is essential for proper understanding of joint behaviour and should be always the first step in structural analysis. The usage of at least two different approaches is one way of improving safety checks in engineering and stands for purpose of validation or verification of design.

Keywords: Joint, Fillet weld, Stress, Classical approach, FEA

1. INTRODUCTION

Welding is the joining method that creates one-piece member out of several components. The earliest form of welding has traces back to ancient times but the welding as we recognize it today was developed in the last decades of the 19th century. One may say that modern welding brought the revolution and freedom in design of large-scale structures.

The transfer of load and stiffness, by welded connections, can be introduced in a gradual continuous manner, instead of in step changes through bolted connections. Welded connections can be considered as very important for safety and durability of steel structure. However, ensuring weld quality and performance require perfect correlation between the design and fabrication. First step is dedicated to designer who are responsible for the selection of joint type, weld size, weld properties and calculation of design resistance. Fabrication is very important because it depends of parameters such as right choice of welding process, skills and experience of welders, working conditions, etc [1]. The final step in welding process is inspection which is dedicated to approval of welds with respect to the level of flaws.

Basic categories of welds are fillet and butt. The design resistance of a butt weld (full penetration) should be taken as equal to the design resistance of the weaker of the parts connected, provided that weld characteristics are not less than those specified for the parent metal. Butt welds are often subjection to inspection which gives the best insight in quality of fabrication process.

Hence, this paper deals with calculation of fillet weld connections due to the fact that they are widely present in structures and sometimes considered as the weakest link in structural strength. Also, they are essential for the corner joints (sometimes referred as T-configuration) at framed structures in mechanical engineering and are rarely subjected to inspection. Since their production is much cost effective then butt welds, the comprehensive design of fillet welds should be the cost to pay to ensure safety of the structure.

It is known that weldable structural steels should be preferred within this topic. Any other postulation is in relation to experimental investigations/tests [2]. Also, the problem of residual stress and deformation caused by welding are present in researches [3,4].

The aim of this paper is to check the correlation between the results of calculation of design resistance of fillet weld connections by two completely different approaches. The emphasis in this paper is given only on the strength of load-carrying welds. First approach is conventional way of calculation which implies application of knowledge from the theory of strength of materials and steel structures. Theoretical background may have deviations for welded or bolted connections and structural analysis in such locations requires special attention. Therefore, this approach will be based mostly on national engineering practice. It presents long-lasting tradition in engineering and even today is in everyday usage. It is accompanied with set of guidances due to issues of small sizes of welds when compared with sizes of beams which are usually subjected for joining by welding process.

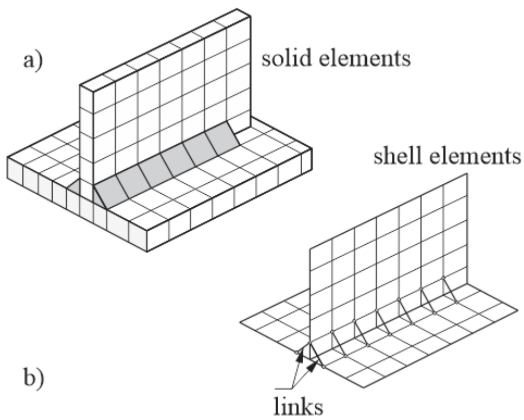

Figure 1:Modelling ways of welds with FEA

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Regarding classical approach one also have to consider requirements from Eurocode. Within the title topic, especially is important Eurocode 3: Design of steel structures-Part 1-8: Design of joints (EC 3-1-8) which introduced the concept of joint as a system of interconnected items, i.e. component method (CM), [5].

Second approach is numerical simulation of weld stresses which is often performed by the finite element analysis (FEA). Here, FEA belongs to the typical static analysis while other type like thermal analysis can be used for simulation of the welding process [6].

There are two characteristic ways of modelling by FEA. One is usage of solid elements (standard technique in engineering) where is preferred to model welds as separate bodies (Fig. 1.a). Sometimes, it requires the volume modelling of welds and is often challenging task because of weld shape to be analyzed which makes a crucial influence on the results. The second way is modelling with 2D elements and is common and allowed by most of the standards. For the cases where results are affected by local influences, the welds may be included by inclined elements having appropriate stiffness or by links to couple node displacements (Fig. 2.a). Along with big advantages with FEA, significant drawback comes from the fact that is necessary to find stresses and reorient them into the weld direction to perform the checks.

Here, the FEA will be concerned with the usage of shell elements for modelling of joint. In order to narrow down the field of FEA, it will be used software which preserves main recommendations from EC 3-1-8 (which traced the component based methods in calculation of joints). This software can be concerned as modern and particular tool for calculation of joints.

The experimental approach within this field is highly appreciated but expensive and reserved to simple cases of welded joints [7,8,9]. It had valuable importance in 20th century but now is common opinion that finite element approach can be used with confidence to predict failure loads of joint under various loadings [10].

2. DESIGN RESISTANCE OF THE FILLET WELD

When design resistance of the fillet weld is considered, an uniform distribution of stress is assumed on the throat section of the weld, leading to the normal stress and shear stresses shown in Fig. 2, as follows: $\sigma_\perp$ - the normal stress perpendicular to the throat, $\tau_\perp$ - the shear stress perpendicular to the length of the weld, $\tau_\parallel$ - the shear stress parallel to the length of the weld.

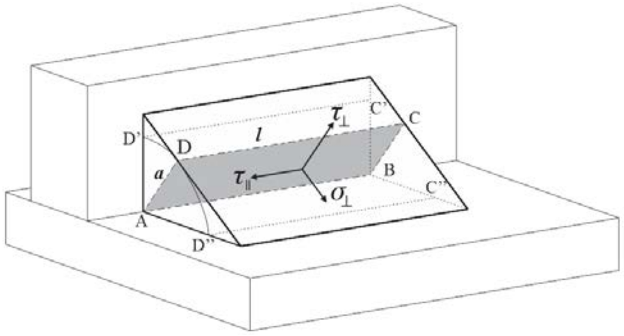


Figure 2: Stresses on the throat section of a fillet weld

2.1. Requirements of the EC3 standard

According to the EN 1993-1-8, the design resistance of the fillet weld should satisfy following expressions:

$$\sqrt{\sigma_\perp^2 + 3\left(\tau_\perp^2 + \tau_\parallel^2\right)} \le \frac{f_u}{\beta_w \gamma_{M2}} \quad (1)$$

$$\text{and } \sigma_\perp \le 0,9\frac{f_u}{\gamma_{M2}} \quad (2)$$

where: f_u - the nominal ultimate tensile strength of the weaker part joint; β_w - appropriate correlation factor (0,8 for S235 and 0,9 for S355); γ_{M2} - partial safety factor for joints (recommended 1,25).

It is common to understand the (1) in the following form:

$$\sigma_{w,eq} \le \sigma_{w,Rd} \quad (3)$$

where $\sigma_{w,eq}$ represents the weld equivalent stress (can be considered as von Mises stress and in further text denoted as σ_{eq}) while $\sigma_{w,Rd}$ represents design (permissible) stress. For the sake of clarity, the right side of (2) will be denoted as $\sigma_{w,Rd,1}$ which presents the limit value for normal stress.

In order to use previous relations, the objects of calculation should be made of weldable structural steels (e.g. yield strengths in the range 185-460 MPa). Also, required quality level of weld should be C (according to EN ISO 2518) if not otherwise specified.

2.2. Requirements of the national standard

Due to position (slope) of the throat section, determination of the stresses in throat section can be fairly complicated. According to Serbian engineering practice, the long-lasting usage of regulations of the Serbian standard JUS U.E7.150 (probably based on DIN) allows the calculation of stresses in horizontal/vertical neighbouring plane which share root line with the throat section (Fig.2). Obviously, this can be considered as needed but represents the initial approximation in calculation. This can simplify the process of calculation for the designer. In further text, this will be marked as classical (sometimes called conventional) approach.

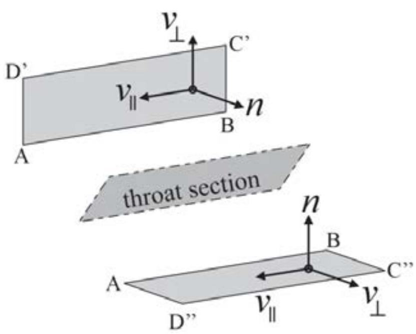


Figure 3: Stresses in neighboring planes

Hence, the calculation of stresses can be done in section ABC'D' or ABC''D'', according to the perspective of the designer (Fig.2). The design resistance is governed by following expression:

$$\sqrt{n^2 + v_\perp^2 + v_\parallel^2} \le 0.5\frac{f_u}{\nu} \quad (4)$$

where: n - normal stress, $v_{\perp}$ - shear stress perpendicular to the length, $v_{\parallel}$ - shear stress parallel to the length, ν - safety factor (usually 1.5 for standard load cases).

This can be presented in form:

$$\sigma_u \le \sigma_{w,dop} \tag{5}$$

where σ_u represents the vector sum of the three components while $\sigma_{w,dop}$ represents the permissible stress in weld.

Obviously, the classic approach have different designation of stress components in the weld from the designation of stresses in throat section. This is proper hypothesis which provides valuable results when adequate governing condition is used (4).

In order to find mathematical correlation between the stresses from two different equations (1,4), one may use for 90º-fillet weld following:

$$\sigma_{\perp} = \frac{n+v_{\perp}}{\sqrt{2}} \tag{6}$$

$$\tau_{\perp} = \frac{n-v_{\perp}}{\sqrt{2}} \tag{7}$$

$$\tau_{\parallel} = v_{\parallel} \tag{8}$$

These relations are valid for the vertical plane, i.e. section ABC’D’, but can be similarly derived for horizontal plane [11].

3. MODEL DEFINITION

The object for two different approaches is the same and represents the corner beam-to-column joint which is widely present at structures in mechanical engineering. In order to expand the level of comparison between the classical and FEA approach, the results are obtained for two models of corner joint: Model 1 and Model 2. For both models, it is assumed that all the steel members are made of structural steel S235 with ultimate tensile strength of f_u=360 MPa and designed for normal working conditions. Hence, one may calculate permissible stresses by EC3 or by national requirements, as given in Table 1.

Table 1: Limit values of stresses [MPa]

JUS	EC3	
$\sigma_{w,dop}$	$\sigma_{w,Rd}$	$\sigma_{w,Rd,1}$
120	360	259

Model 1 includes a beam with rectangular hollow section (EN 10210) 200x100x8 mm and a column of HEA 300 (Euronorm 53-62). The joint is designed to be welded with the 90º-fillet weld with the throat size of 5 mm (*a*). Welds on the flanges of hollow section has length of 60 mm (l_1) while welds on the webs has length of 160 mm (l_2). With intention to find as much characteristic informations as possible, the arrangement of welds excludes the all-round fillet weld for rectangular section. The used lengths of welds can be considered as effective (load-carrying) lengths.

Model 2 includes a beam with circular tube section (EN 10210-2) 219,1x8 mm and a column with rectangular hollow section (EN 10210) 300x300x10 mm. The joint is welded all-round with the throat size of 5 mm (*a*). The idea behind this model is to check the effects of circular meshing in FEA.

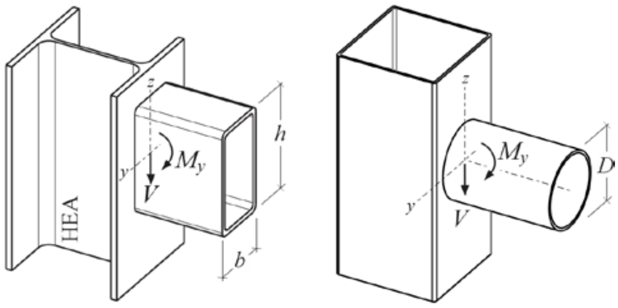


Figure 4: Beam-to-column joint: Model 1, Model 2, respectively

It is used the most common case of loading at frame structures which includes the presence of vertical force (*V*) and moment (M_y, in further text denoted only as *M*), i.e. loadings in vertical plane. In order to compare the stresses between the two approaches the influence of loadings will be considered individually and then combined. For the sake of clarity, the load cases are denoted as follows:

Case 1: vertical force-*V*
Case 2: moment –*M*
Case 3: combination, *V*+*M*

3.1 Classical approach

The postulation of statical model in chapter 3 is fairly simple according to classical approach. For Model 1, the known stress distributions from the influence of loadings are depicted in Fig. 5 where is assumed that section of vertical welds is carrying the vertical force. There are distinguished three characteristic points for calculation of stresses.

Normal stresses can be conducted upon following:

$$n = \frac{M_y}{I_{y,w}} z \tag{9}$$

where $I_{w,y}$ is second order moment of area of the weld section related to the y-axis.

Shear stresses can calculated by

$$v_{\parallel} = \tau = \frac{V}{2a \cdot l_1} \tag{10}$$

or with parabolic distribution, exceptionally for the point 3 and for Case 1, as

$$v_{\parallel} = 1,5\tau \tag{11}$$

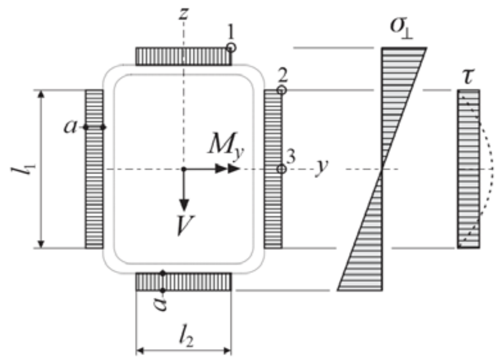


Figure 5: Model 1-Stress distribution

For Model 2, the stress distribution is presented in Fig. 6. The normal stresses can be conducted with (9) while shear stresses have to be obtained by

$$v_{\parallel} = \tau = \frac{V \cdot S_{w,y}}{I_{w,y} \cdot 2a} \tag{12}$$

where $S_{w,y}$ is first order moment of area of the weld section related to the *y*-axis.

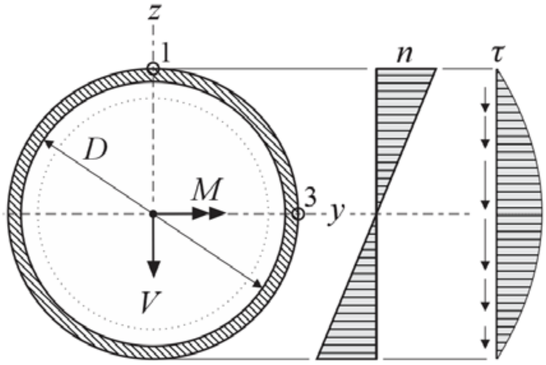


Figure 6: Model 2-Stress distribution

There are distinguished two characteristic points for calculation of stresses (denoted as 1 and 3) to correspond to points from Model 1.

The geometrical properties of weld section, for each model, belong to the class of calculation for simple shapes which can be found in literature [12].

For both models, the external loadings are taken as *V*=100 kN and *M*=1000 kNcm (10 kNm). It will be shown in further text that these values are assumed in order to get „high" values of stresses which are close to the permissible stresses.

3.2 FEA approach

As mentioned in introduction chapter, the FEA approach is here oriented towards particular tool for design of steel structural joints. It is used software IDEA StatiCa which introduced Component Based Finite Element Model (CBFEM) as extension of classical FEM with main parts of CM [13]. Hence, the behaviour of components such as column web in shear and in compression, beam flange and web in compression and column flange in bending are incorporated in software. According to the authors, this software is: comprehensive enough to provide: good informations about joint behaviour, stress, strain and about overall safety and reliability; fast enough in daily practice to provide results in a time comparable to other FE tools.

Regarding postulated object of interest, both the flanges and webs of connected members are modelled with shell elements with 4-nodes at its corners having six degrees of freedom. The welds are treated as multi-point constraints (MPC) and relates the finite element nodes of one plate edge to another. The constraint allows modelling the midline surface of the connected plates with the offset, which respect the real weld configuration and throat thickness. The load distribution in the welds is derived from the MPC, so the stresses are calculated in the throat section. This is important due to postulation given in chapter 2.1.

The basic representation of FEA models is given at figure 7, according to postulation in chapter 3. The external loadings are acting on the joints for both models and presented by software on the "free" beam ends due to visibility.

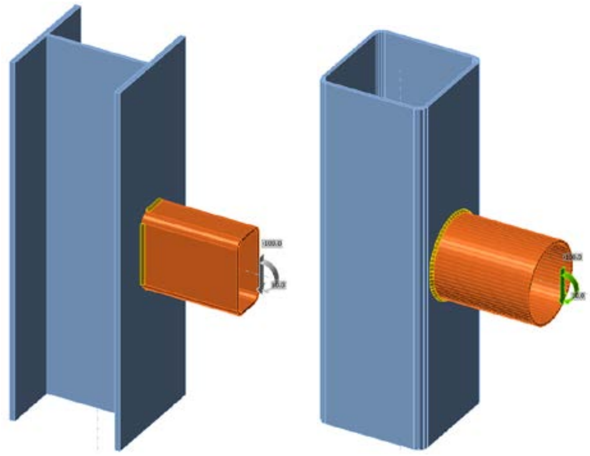


Figure 7: FE models- Model 1and Model 2, respectively

4. RESULTS AND DISCUSSION

Before consideration of stresses for characteristic models, one have to perceive the limit values of stresses (permissible stresses) obtained from two different standards (Table 1). It is obvious that values are significantly different. For used material (S235), the governing criterion for permissible stress by EC3 gives value which is three times bigger than obtained by JUS. This cannot serve for direct comparison due to the fact that permissible stresses by these standards correspond to the working stresses which are not observed in the same sections. EC3 uses von Mises criterion for calculation of nominal stress in weld while JUS uses direct sum of vectors for same purpose. Even for 90^o-fillet weld, where is known relations between the vectors from throat section and the neighbouring plane, one may find a gap between the limit values of stresses. It can be concluded that limit values of stresses by JUS (probably based on DIN) can be considered as conservative and very strict. The EC3, as relatively new standard, has to be considered as governing due to 30-years usage in European engineering practice. Regarding the design resistance of weld, it cannot be considered as so strict due to fact that limit values of stresses in weld are higher than for structural elements. One may assume that this point comes from the research and knowledge from development of calculation models in structural analysis. Due to postulation which concerns the behaviour of various components in joints, the EC3 can be considered as exceptionally comprehensive.

4.1 Model 1

Upon the model definition in chapter 3, one may find geometrical properties of the weld section and consequently the values of stresses. Table 1 presents review of calculated results obtained with (5,9,10,11). Furthermore, it is performed mathematical transformation with (6,7) to obtained corresponding equivalent stress.

The corresponding analysis is performed with CBFEM software and main results for stresses in welds are shown in Table 3. The results are shown for horizontal welds (length of 60 mm) and for vertical welds (length of 160 mm), for the purpose of distinction.

Table 2: Obtained stresses-classical approach [MPa]

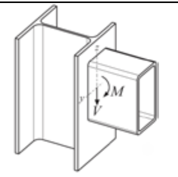
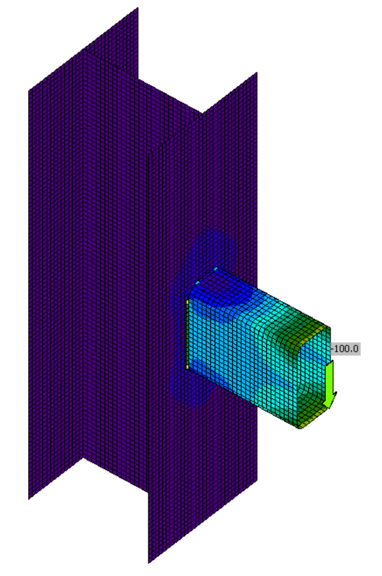
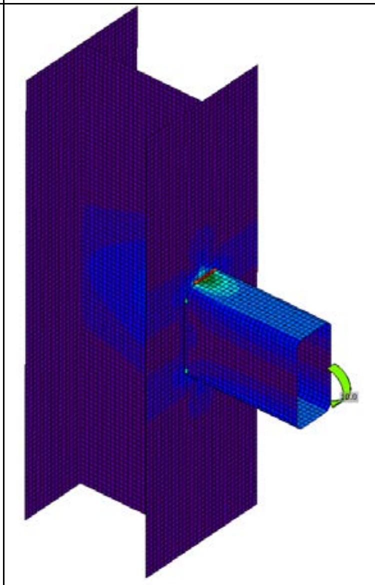
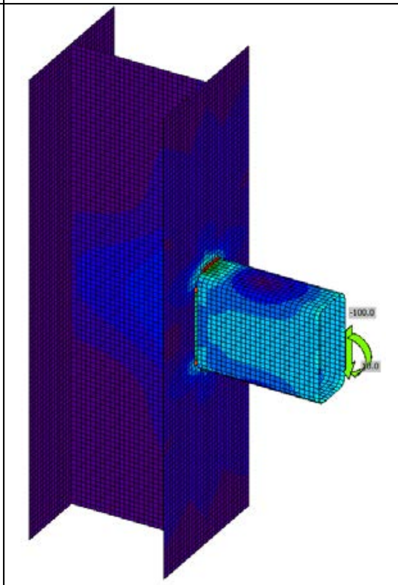
	Load	Case 1 (V)				Case 2 (M)				Case 3 ($V + M$)			
	Point	n	$v_{\|\|}$	σ_u	σ_{eq}	n	$v_{\|\|}$	σ_u	σ_{eq}	n	$v_{\|\|}$	σ_u	σ_{eq}
	1	0	0	0	0	109	0	109	154	109	0	109	154
	2	0	62.5	62.5	108	85	0	85	120	85	62.5	105	161
	3	0	93.7	93.7	162	0	0	0	0	0	62.5	62.5	108

Table 3: Obtained stresses-FEA approach [MPa]

	Case 1				Case 2				Case 3			
												
Weld	$\sigma_\perp$	$\tau_{\|\|}$	$\tau_\perp$	σ_{eq}	$\sigma_\perp$	$\tau_{\|\|}$	$\tau_\perp$	σ_{eq}	$\sigma_\perp$	$\tau_{\|\|}$	$\tau_\perp$	σ_{eq}
Horiz.	±26.2	-34.2	±9.2	66.6	±128	64.7	130	283	±128	73.3	±130.9	289.9
Vertic.	-13.8	±77	-1.1	134	-33.9	±29.9	31.3	82	20	±107	-32.5	194.7

The first comparison can be done with following observations of corresponding equivalent stresses (σ_{eq}) from different approaches (Table 2 vs. Table 3): there is not direct matching of results; the values from FEA are up to twice higher than from classical approach. Many reasons can be found to explain previous statement because this FEA includes: elastoplastic behaviour in equivalent weld; behaviour of plates (webs and flanges of connecting elements) under loadings. However, this cannot be considered as ending point for evaluation between these approaches.

In the second phase of comparison the values from FEA (Table 2) should be looked separately. The choice of load Case 1 and Case 2 can serve as background for analysis. One may notice following: for Case 1, the biggest value of the shear stress parallel to the length of the weld ($\tau_{||}$=77 MPa) is reserved for vertical welds; for Case 2, the biggest values are reserved for $\sigma_\perp$ and $\tau_\perp$ (128 MPa and 130 MPa), which mutually contribute to the stress oriented normal to the vertical plane; for Case 3, the previous observations are also preserved as their combination. Hence, it can be said that joint behaviour sustains known effects of vertical force and bending moment. Apart from main tabular data, software provides graphical distribution of equivalent stresses for welds. This option will be sketched in Figure 8 and is used for final phase of comparison. Due to symmetrical weld section, the presentation at this figure is given for both the characteristic cases (1 and 2).

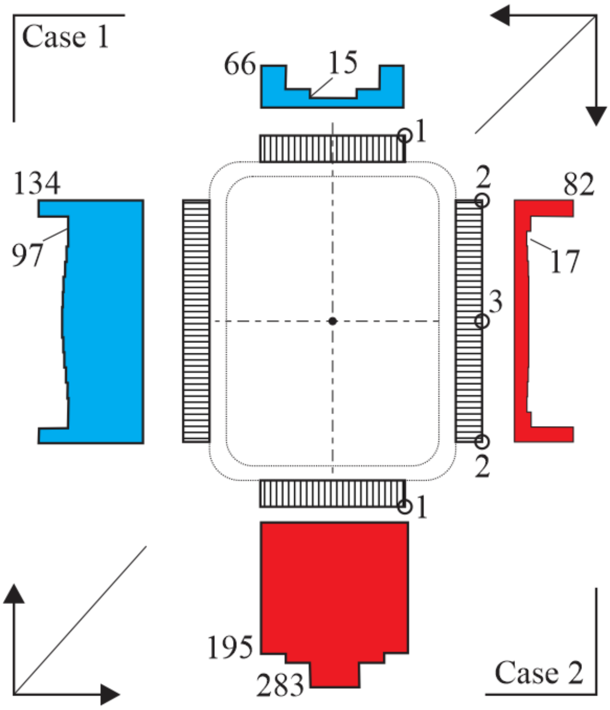


Figure 8: FE models, sketched stress distribution

It is obvious from Fig.8 that stress distribution is different than from classical approach (Fig. 5). It was expected because, as stated in software manual, IDEA StatiCA gives plastic stress redistribution in welds. The given stress distribution provides the highest and the lowest value of stress. The highest level can be named as stress peaks and can be an issue of discussion. Moreover, these stress peaks are only given in main tabular data (Table 3) and used directly for checks of the weld. It may be on the safe side of design resistance but certainly requires more explanations and instructions in manual. Disregarding the stress peaks in distribution for vertical welds, for Case 1, one may found relatively uniform distribution over length. This can be questioning point for the usage of the (11) in classical approach which is common approximation to preserve safe side of check.

Considering the current model, the main questionable point can be stress distribution for vertical welds in Case 2. In order to test this specific situation, the additional model is done with stronger column (HEM300) which have stiffeners, with the idea to prevent the local effects as much as possible (Fig. 9). The sketched distribution is given in figure 9 and has the shape which resemble the known stress distribution due to bending moment. This step of modelling can be considered as creating highly rigid column but one may found this as very useful possibility of the software for the review of results by the engineer with respect to hand calculations.

The classical approach relies only on geometrical properties of weld sections. It implies the knowledge and experience in engineering regarding the design of connections in terms of member thickness and need for stiffeners. However, it can be said that is related to the zone of rigid connections which can be considered as basic drawback.

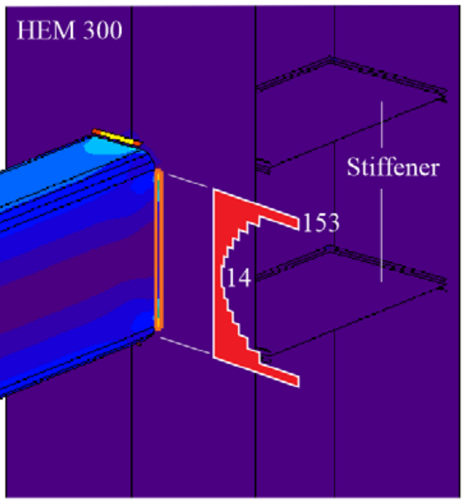


Figure 9: Sketched stress distribution (rigid joint)

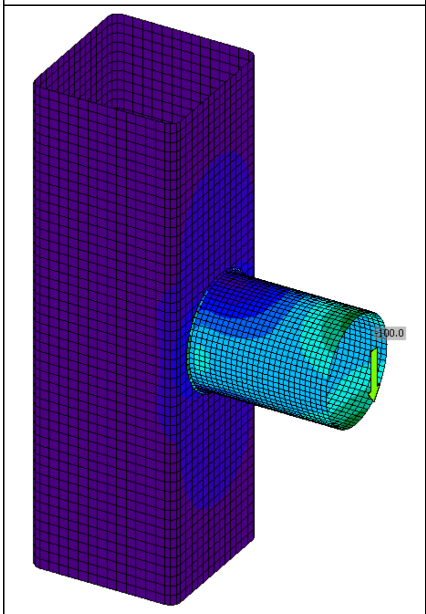
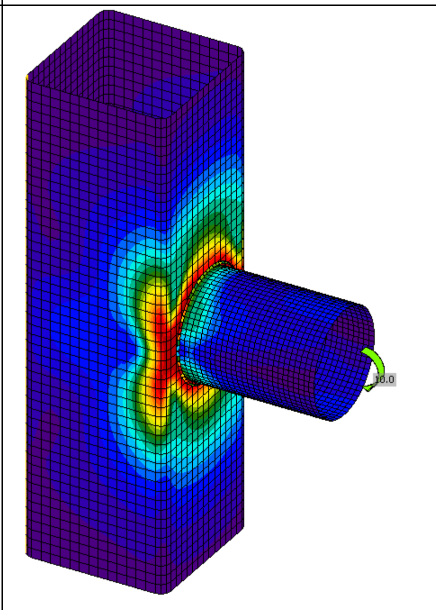
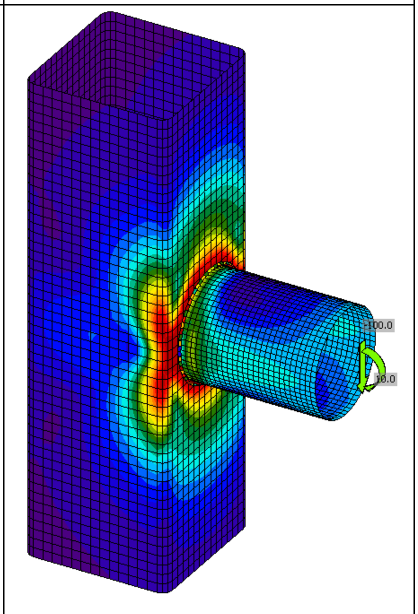
4.2 Model 2

The corresponding procedure from previous chapter is performed for Model 2. The results by classical approach are given in Table 4, upon the usage of (9) and (12) and consequently the (6) and (7) for determination of equivalent stress. The results from CBFEM are given in Table 5, based on maximal stresses found by software.

Table 4: Model 2-Obtained stresses-classical approach [MPa]

Load	Case 1 (*V*)				Case 2 (*M*)				Case 3 (*V* + *M*)			
Point	n	$v_{\|\|}$	σ_u	σ_{eq}	n	$v_{\|\|}$	σ_u	σ_{eq}	n	$v_{\|\|}$	σ_u	σ_{eq}
1	0	0	0	0	48	0	48	67	48	0	48	67
3	0	55	55	95	0	0	0	0	0	55	55	95

Table 5: Model 2-Obtained stresses-FEA approach [MPa]

Case 1				Case 2				Case 3			
											
$\sigma_\perp$	$\tau_{\|\|}$	$\tau_\perp$	σ_{eq}	$\sigma_\perp$	$\tau_{\|\|}$	$\tau_\perp$	σ_{eq}	$\sigma_\perp$	$\tau_{\|\|}$	$\tau_\perp$	σ_{eq}
5	-58	-2.2	100.6	63.8	132.5	-33.1	245	-123	112	92.3	280

For this model, one may also cannot find direct matching of results for corresponding equivalent stresses (σ_{eq}). This is not related to the Case 1 where is obtained good correlation which cannot serve as conclusion due to *low* importance of shear stresses in structural analysis.

The difference in results is especially noticeable for Case 2 and consequently for Case 3. The reason is shown in graphical presentation of stresses for these cases (by software) where it is obvious that local effects of the connecting plates have high influence on the stress level. The gap between the results could be smaller if one assumes that shear stress has uniform distribution in the vertical direction of the weld. This could be one point of view for increase of safety zone in weld calculation by scholar approach.

This kind of joint does not have much possibility for placing stiffeners at the column and modification towards increase of joint rigidity can be done only with bigger thickness of the box tube. It is not given here because of detailed comparison for Model 1 which can be used as general explanation for current model. The graphical distribution of equivalent stresses for welds, as capability of software, is not readable as for Model 1. It can be assumed that this problem arises from the circular shape of welds which certainly invokes the problem of presentation. Hence, the comparison with the stress distributions on the Fig. 6 cannot be performed.

5. CONCLUSION

Two different approaches are concerned here regarding the strength of fillet-welded joint connections. First one is calculation with classical (scholar) approach while second one is devoted to finite element analysis by particular software. The object for calculations represents the corner beam-to-column joint which is widely present at structures in mechanical engineering. The weldable structural steel is default material within this topic. The strength analysis is related to load-carrying welds which are often subjected to safety checks.

Within the classical approach, the design resistance of the fillet weld is considered with two different expressions. Basic one is requirement from national standard (JUS, most likely based on DIN) which has long-lasting tradition and fairly simplifies the process of calculation. This one can be named as scholar approach. The second one is requirement from EC3 which deals with stresses on the throat section of a weld. In order to perform calculations in classical way, one have to start with scholar approach and use mathematical transformations for vectors in corresponding planes. Hence, these two requirements are different and cannot be easily compared. One may say that requirements from EC3 invokes the finite element approach for determination of stresses.

The usage of FEA is performed with particular, comprehensive and modern software which includes the component method in joint design. The results are presented in tabular form, along with many informations about joint behaviour. The graphical presentation of stress state is extremely valuable, along with deformed shape which provides useful comprehension of joint behaviour under loadings.

The aim of this paper was not to compare the incomparable but to find some point of correlation with classical and FEA approach. It is shown that basic nature of weld resistance under loadings is preserved with both the approaches, without direct matching of numerical values. As presented in chapter 4, previous conclusion is not so obvious even considered as expected. According to presented procedures and current trends in engineering, many points go in favour of the usage of FEA. The main advantage is surely the implementation of local influence of plates in connected members. The software used in this research is especially valuable and one may found many possibilities for the design of joints. Regarding weld stresses only point of discussion can be the occurrence of stress peaks because of lack of informations for this in instructions which should be accompanied with mathematical postulations of MPC. The sort of recommendation is to give stress distribution oriented to the position of the weld which will improve the perception of the results. Previous statement is especially noticeable for round welds in Model 2. Some reference points in welds should be predefined in order to allow engineers to perform easier comparison with classical approach. The previous does not diminish the benefits of the presented software but only serve as a point of improvement for readability of results.

According to the authors, some level of validation or verification should be present in structural analysis. Even with FEA as dominant trend in engineering, the classical approach should be concerned in some form. The time spent for reading and understanding the results from software should be the cost for better qualifications of an structural engineer.

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