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TRUSS BEAMS WELDED JOINTS – MANUFACTURING IMPERFECTIONS AND STRENGTHENING SOLUTIONS

ZAVARENI SPOJEVI REŠETKASTIH NOSAČA – GREŠKE U IZRADI I REŠAVANJE OJAČANJA

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- truss beams
- welding imperfections
- steel structures

Abstract

In case of steel structure buildings with large spans, due to the ease of manufacturing and following weight optimization, the H or I type profiles for truss beam elements with welded joints are used increasingly more often.

In order to ensure the strength and stability of the assembly, the manufacture by welding and the position of diagonals at the top and lower chords are very important to be respected as presented in the design project (drawings). In many cases these joints have manufacturing imperfections that lead to high stresses in the joint area.

The paper presents the case of HEA type profile truss beam joints designed according to EN1993-1-8. The manufacture reveals eccentricities in joints and a high level of imperfections. Presented are the effects of imperfections and the strengthening solutions for the joints.

Ključne reči
- zavareni spojevi
- rešetkasti nosači
- greške zavarivanja
- čelične konstrukcije

Izvod

U slučajevima zgrada od čeličnih konstrukcija širokog raspona, zbog jednostavnosti izrade i poštujući optimizaciju težine, H ili I profili elemenata rešetkastih nosača sa zavarenim spojevima se sve češće koriste.

Radi postizanja čvrstoće i stabilnosti sklopa, izrada zavarivanjem, kao i položaj dijagonala u gornjim i donjim elementima rešetke su veoma bitne činjenice koje treba poštovati onako kako je predstavljeno u projektima (crtežima). U mnogim slučajevima se kod ovih spojeva javljaju greške u izradi koje dovode do pojave visokih napona i oblastima veza.

U radu je prikazan slučaj veze profila tipa HEA rešetkastog nosača projektovanog prema EN1993-1-8. Otkriveni su ekscentriciteti u vezama i velik broj grešaka. Prikazani su uticaji grešaka i rešenja za ojačavanje spojeva.

INTRODUCTION

In case of building steel structures, the welded joints truss beams solution had been used increasingly in the twentieth century with a scale large development due to the possibility of increasing spans in buildings. Due to cheap labour in the past, the solution with angle profiles welded on the gussets was first widely adopted.

Nowadays, the solution with RHS/MSH type profiles is used more often with joints made by welding directly onto the bottom and top chord of the truss beams. In case of large span truss beams or carrier truss beams, the solution with HEA/IPE profiles for the truss elements is adopted.

A very important step for the evolution of these structural solutions has been done by configuring a joint design calculation mode imposed by Eurocode. Following the application of EN1993-1-8 and EN1090/1, EN1090/2 standards, all kinds of welded joint types can be designed and manufactured with quality control.

DESIGN OF WELDED JOINTS ACCORDING TO EN 1993-1-8

Welded joints of truss beams with HEA/IPE elements can be assimilated with beam to column welded joints where the truss is the beam and the chord (bottom or top) represents the column. This kind of assimilation presents the advantage of applying the component method in the design of these types of joints.

According to the component method, /1/, each joint is divided into three areas with different kinds of stresses:
– area with tension,
– area with compression,
– area with shear force.

Each area can be identified by different deformability sources which represent simple elements (or "components") which contribute to the global response of the node. In theory, this methodology can be applied to any node configuration with any type of loading with the condition of an existing very accurate description of each base component.

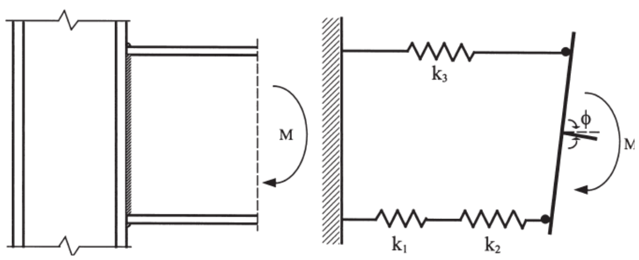


Figure 1. Welded beam to column joint – mechanical model, /3/.
Slika 1. Zavareni spoj nosača i stuba – mehanički model, /3/.

The base components of the joints are modelled through a linear spring with elastic-plastic characteristics. In fact, the complex answer of a spring is simplified through a bilinear elastic-perfectly plastic relation as presented in Fig. 3. The two of the characteristics which are permitting the spring behaviour modelling are *axial rigidity* – K and the *plastic resistance* (Fig. 2).

In case of modelling, the component characteristics are:
– secant rigidity to tension/compression k_e/η;
– plastic resistance to tension/compression F_{Rd};
where k_e represents the initial component rigidity and η is a modified rigidity coefficient.

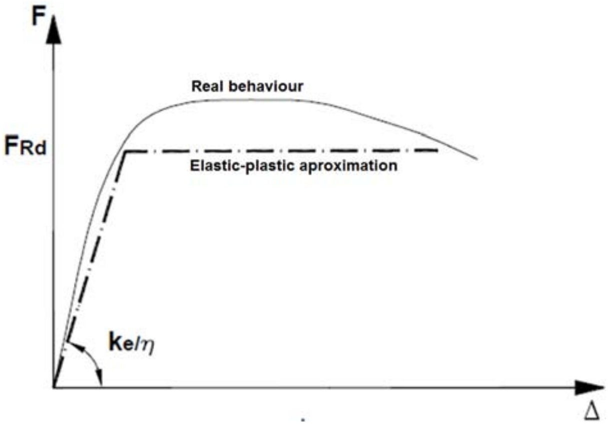


Figure 2. Idealizations of the force-deformation curve /2/.
Slika 2. Idealizacija krive sila-deformacija /2/

Figure 3. Modelling of a component under axial stress.
Slika 3. Modeliranje komponente pod aksijalnim naponom

Rigidity to translation, resistance and deforming capacities are considered separately for each component.

From the components grouping point of view, for components which act in parallel, the resistances and rigidities must be summed, but the minimal deformation capacity will determine the ductility of the whole assembly (joint).

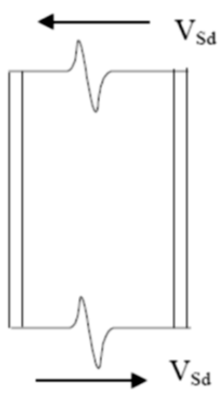
For components that are acting in a series (e.g. column flange under bending and tension in a beam flange), the initial rigidity can be obtained through a reciprocity equation and the resistance is that as in the component with the minimum. The deformation capacity is that as in the component with a minimum to which are summed deformations of other components corresponding to the loading level. Through linear grouping of compressed areas or ones with acting shear force, a single translational spring results for each rigidity group, resistance and deformation capacity.

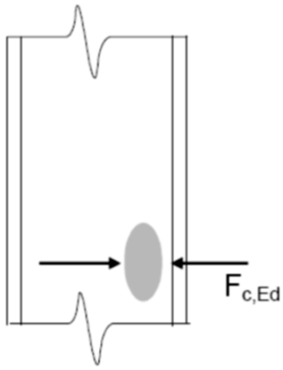
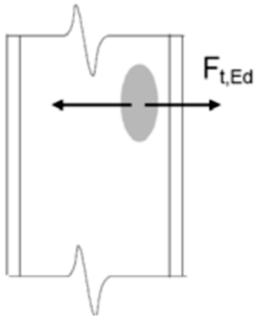
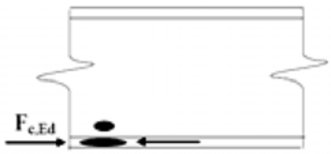
For rotational grouping of components, it is simplified considered the fact that the centre of rotation for all tensioned rows is located at the centre of the beam bottom flange, although this fact is valid only for joints with high rigidity of compressed components.

In case of any welded joint, the first step is to identify the active components which must be considered for the joints design. Thus for a welded beam to column joint there are the following active components:
– column web panel under shear force;
– column web under transversal compression;
– column web under transversal tension;
– beam chord and web under compression.

In the following Table 1, presented are design formulas for a welded beam to column joint.

Table 1. Design formulas for the resistance design of a welded beam to column joint.
Tabela 1. Proračunske formule projektovanje zavarene veze nosača i stuba.

Component	Figure	Resistance design
1. The column web panel under shear force		According with EN1993-1-8 Ch. 6.2.6.1. $$V_{wp,Rd}=\frac{0.9 f_{y,c} A_{vc}}{\sqrt{3}\,\gamma_{M0}}$$ where: A_{vc} is the shear area of column cross section (according with EN 1993-1-1). If column web stiffeners are used, then: $$V_{wp,Rd}=\frac{0.9 f_{y,c} A_{vc}}{\sqrt{3}\,\gamma_{M0}}+V_{wp,add,Rd}$$ with $V_{wp,add,Rd}=\frac{4M_{pl,fc,Rd}}{d_s}\leq\frac{2M_{pl,fc,Rd}+2M_{pl,st,Rd}}{d_s}$ where: d_s is the minimum distance between the median axis of the stiffeners; $M_{pl,fc,Rd}$ is the resistance plastic moment of the column flange from its own median axis; $M_{pl,st,Rd}$ is the resistance plastic moment of one stiffener from its own median axis. The web panel resistance can be increased by adding stiffeners welded onto the web of the column. In this way the shear area is increased by the area of the welded plate.

2. The column web under transversal compression		According with EN1993-1-8 Ch. 6.2.6.2. $$F_{c,wc,Rd}=\frac{\omega k_{wc} b_{eff,c,wc} t_{wc} f_{y,c}}{\gamma_{M0}}\text{, but}$$ $$F_{c,wc,Rd}\le\frac{\rho\,\omega k_{wc} b_{eff,c,wc} t_{wc} f_{y,c}}{\gamma_{M1}}$$ where: ω is a reduction factor in accordance with shear from the web panel of the column. The value of ω depends directly on web shear value, through the transformation parameter β and the ratio between the area from the compression resistance ($b_{eff,c,wc}$, t_{wc}) and column share area A_{vc}; $b_{eff,c,wc}$ is effective width of column web under compression calculated with 6.10-6.12 formulas EN 1993-1-8. $b_{eff,c,wc}$ represents the width of the column which is under compression from the beam web; ρ is a reduction factor which takes into account the buckling of the plate. If the compression resistance of the column web needs to be increased, two possibilities can be applied: a) increasing the column web by adding stiffeners onto the web panel of the column (as for the column web under shear) with contour welding; b) adding transversal stiffeners in the area of the beam flanges (in this way stiffening the web of the column).
3. The column web under transversal tension		According with EN1993-1-8 Ch. 6.2.6.3. $$F_{t,wc,Rd}=\frac{(\omega b_{eff,t,wc} t_{wc} f_{y,wc})}{\gamma_{M0}}$$ where: ω is a reduction factor in accordance with shear from the web panel of the column on the basis of the value $b_{eff,c,wc}$; $b_{eff,t,wc}$ is effective width of the column web under tension calculated with 6.16 formulas, /1/, for welded joints. $b_{eff,t,wc}$ represents the width of the column which is under tension from the beam web. The increase in the column web tension resistance can be done with same solutions as presented at point 2 (in case of transversal compression of the web).
4. The beam chord and web under compression		According with EN1993-1-8 Ch. 6.2.6.7. $$F_{c,fb,Rd}=\frac{M_{c,Rd}}{h-t_{fb}}$$ where: $M_{c,Rd}$ is the resistance bending moment of the considered cross section; $h-t_{fb}$ is the distance between the median axis of profile flanges (considering that the resistance moment equals to coupling forces acting in the flanges axis).

FLAWS FROM MANUFACTURE – INFLUENCE AND STRENGTHENING SOLUTIONS

In the manufacturing process of truss beam assemblies, even they are done in workshop conditions, often the component elements have imperfections. Imperfections found most frequently are:

– setting the axes of truss element in the nodes – the elements are not centred or are not with respect to the detailed execution project;
– corner welding is done partially or without respecting the project thickness, in many cases also with welding flaws;
– penetrated welding is done incomplete.

In order to exemplify, the case of a single story building is presented with a truss beam roof of HEA truss profiles. The building has three openings of 25 m and six bays of 18.75 m.

The span of the presented truss beam is 18.75 m with the geometry presented in Fig. 4. The structural analysis results are shown in the following profile sections (Table 2).

Joints of truss beam elements are welded and designed according with the component method, /1/.

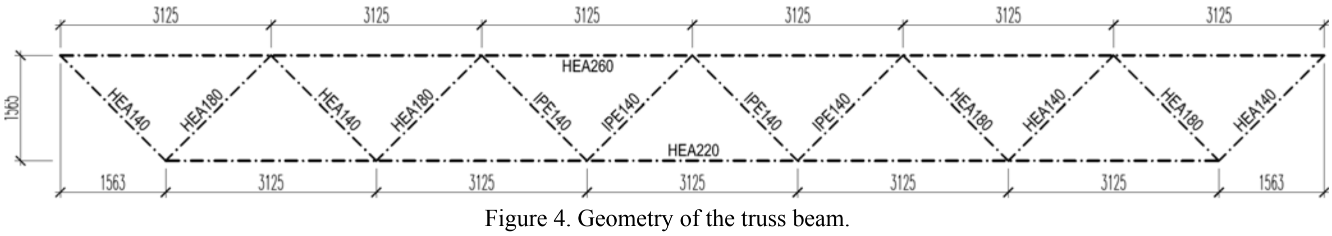


Figure 4. Geometry of the truss beam.
Slika 4. Geometrija rešetkastog nosača

Table 2. Forces and profile types of the truss beam.
Tabela 2. Sile i tipovi profila rešetkastog nosača

Name	Description	Profile type	Steel quality	Axial force
–	–	–	–	(kN)
GZ1-TS	Upper chord	HEA260	S355JR	–1478.67
GZ1-D1	Diagonal	HEA140	S235JR	608.75
GZ1-D2	Diagonal	HEA180	S235JR	–588.45
GZ1-D3	Diagonal	HEA140	S235JR	588.27
GZ1-D4	Diagonal	HEA180	S235JR	–582.02
GZ1-D5	Diagonal	IPE140	S235JR	6.82
GZ1-D6	Diagonal	IPE140	S235JR	–2.78
GZ1-TI	Bottom chord	HEA220	S355JR	1666.48

D3 and D4 with TI joint design

Presented below is the design of the GZ1 truss beam joint with converging D3 and D4 diagonals to the bottom chord. The joint is a welded type with fully penetrated welding of the flanges at the diagonals and with double corner welding of the diagonals web.

Table 3. Acting forces in the joint.
Tabela 3. Sile koje deluju u spoju

Element	Axial force	Joint acting forces
–	(kN)	(kN)
GZ1-D3	588.27	$N_{b1,Ed} = \lvert N_{b1} \rvert \cdot \sin(\alpha_1) = 415.97$
GZ1-D4	–582.02	$N_{b2,Ed} = \lvert N_{b2} \rvert \cdot \sin(\alpha_2) = 411.55$

Shear force in the web of the truss chord

According to Ch. 6.2.6.1, /1/,

$V_{wp,Rd} \geq N_{b1,Ed}$ and $V_{wp,Rd} \geq N_b$.

For a joint configuration, the resistance plastic force for shear of the web panel without stiffeners, $V_{wp,Rd}$ under design shear force, can be obtained with:

$$V_{wp,Rd} = \frac{0.9 f_{y,c} A_{vc}}{\sqrt{3}\,\gamma_{M0}} \tag{1}$$

Thus $V_{wp,Rd} = 381.31$ kN.

One may notice that $V_{wp,Rd} < N_{b1}$, thus an additional plate (stiffener) on the web in the area of the joint is needed, and so adopted: PL 8 × 150.400.

The resistance shear force

$$V_{wp,Rd} = \frac{0.9 f_{y,c} (A_{vc} + b_s t_s)}{\sqrt{3}\,\gamma_{M0}} = 602.66 \text{ kN}.$$

Thus the joint checking condition is satisfied.

Web of truss chord in transversal compression check

According to Ch. 6.2.6.2, /1/, the capable force of the chord web without stiffeners can be determined with:

$$F_{c,wc,Rd} = \frac{\omega k_{wc} b_{eff,c,wc} t_{wc} f_{y,c}}{\gamma_{M0}}, \tag{2}$$

but

$$F_{c,wc,Rd} \leq \frac{\rho\, \omega\, k_{wc} b_{eff,c,wc} t_{wc} f_{y,c}}{\gamma_{M1}} \tag{3}$$

The strain is produced by diagonal GZ1-D4 (HEA180).

Considering a welding height of 7 mm (a_{b2} = 7 mm), the check is done

$F_c \leq F_{c,wc,Rd}$, where $F_c = 0.5 \cdot N_{b2,Ed} = 205.77$ kN.

The capable force of the web under transversal compression is $F_{c,wc,Rd} = 359.34$ kN.

The active width of chord web under compression is:

$$b_{eff,c,wc} = t_{fb2} + 2\sqrt{2}\, a_{b2} + 5(t_{fc} + r_c) = 174.3 \text{ mm}$$

and the reduction factor ω, which takes into account all the possible effects of the web panel shear force:

$$\omega = \frac{1}{\sqrt{1 + 1.3\left(\frac{b_{eff,c,wc} t_{wc}}{A_{vc}}\right)^2}} \tag{4}$$

$\omega = 0.83$

$k_{wc} = 1.0$; $E = 210000$ MPa

$$\lambda_p = 0.932 \cdot \sqrt{\frac{b_{eff,c,wc} d_c f_{y,c}}{E (t_{wc})^2}} \tag{5}$$

$\lambda = 0.891$;

$$\rho = \begin{cases} 1, \text{ if } \lambda_p \leq 0.72 \\ \dfrac{\lambda_p - 0.2}{\lambda_p^2} \end{cases} \tag{6}$$

$\rho = 0.87$.

The condition is satisfied:

$F_c \leq F_{c,wc,Rd}$ (205.77 kN ≤ 359.34 kN).

Web of truss chord in transversal tension check

According with Ch. 6.2.6.3, /1/, the tension force from the web chord must be lower than the capable tension force of the web without stiffeners $F_t \leq F_{t,wc,Rd}$.

The strain is produced by diagonal GZ1-D3 (HEA140).

Considering a welding height of 6 mm (a_{b1} = 6 mm), the check is done $F_t \leq F_{t,wc,Rd}$, where $F_t = 0.5 N_{b1,Ed} = 207.99$ kN

The capable tension force of chord web without stiffeners can be determined with:

$$F_{t,wc,Rd} = \frac{(\omega b_{eff,t,wc} t_{wc} f_{y,wc})}{\gamma_{M0}} \tag{7}$$

$F_{t,wc,Rd} = 351.45$ kN,

where $b_{eff,t,wc} = t_{fb1} + 2\sqrt{2} \cdot a_{b1} + 5(t_{fc} + r_c) = 170.47$ mm and $\omega = 0.83$.

The condition is satisfied:

$F_t \leq F_{t,wc,Rd}$ (207.99 kN ≤ 351.45 kN).

Bottom chord flange in tension check

The strain is produced by diagonal GZ1–D3 (HEA140) – tension and diagonal GZ1–D4 (HEA180) – compression.

The checking relation for diagonal GZ1–D3 is

$$F_{tc,Rd1} \geq 0.5 \cdot N_{b1,Ed} .$$

The tension capable force of chord flange is determined with the following relation:

$$F_{tc,Rd1} = \frac{b_{eff,b1} t_{fc} f_{y,c}}{\gamma_{M0}} \tag{8}$$

$F_{tc,Rd1} = 468.6$ kN,
where $b_{eff,b1} = t_{wc} + 2r_c + 7k_1 t_{fc} = 120$ mm. But

$b_{eff,b1} \geq \left(\frac{f_{y,b1}}{f_{u,b1}} \right) b_{b1} \Rightarrow b_{eff,b1} \geq 91.34$ mm and $k_1 = 1.0$.

Thus $0.5 \cdot N_{b1,Ed} = 207.99$ kN.

The condition is satisfied

$$F_{tc,Rd1} \leq 0.5 \cdot N_{b1,Ed} \quad (207.99 \text{ kN} \leq 468.6 \text{ kN}).$$

The check relation for diagonal GZ1–D4 is

$$F_{tc,Rd2} \geq 0.5 \cdot N_{b2,Ed} .$$

Taking into account that $b_{eff,b2} = 120$ mm and $k_2 = 1.0$, the tension capable force of chord flange is:

$$F_{tc,Rd2} = 468.6 \text{ kN, and } 0.5 \cdot N_{b2,Ed} = 205.78 \text{ kN}.$$

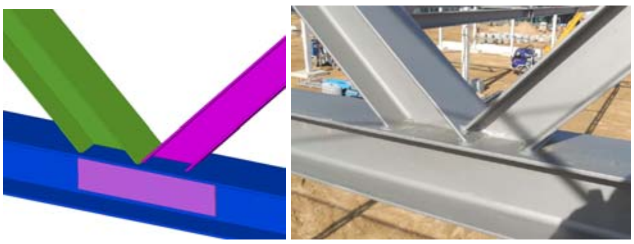
The condition is satisfied

$$F_{tc,Rd2} \leq 0.5 \cdot N_{b2,Ed} \quad (205.78 \text{ kN} \leq 468.6 \text{ kN}).$$

Manufacturing imperfections

Following manufacture quality control in the truss beams erection phase, the following flaws have been noticed:

Imperfection type 1
- eccentricity imperfection – the measured eccentricity is different from the design project – execution details;
- overlapping of the truss diagonals in the joint – the HEA 140 profile on the HEA 180 profile.



(a) (b)

Figure 5. Truss beam joint (a) designed (b) manufactured – imperfection type 1.
Slika 5. Spoj rešetkastog nosača (a) projektovan (b) proizveden – greška tipa 1

Imperfection type 2
- eccentricity imperfection – the measured eccentricity is different from the design project – execution details;
- overlapping of the truss diagonals in the joint – the HEA 180 profile on the HEA 140 profile;
- welding length is lower than the execution details indications.

(a) (b)

Figure 6. Truss beam joint (a) designed (b) manufactured – imperfection type 2.
Slika 6. Spoj rešetkastog nosača (a) projektovan (b) proizveden – greška tipa 2

Analysis of imperfections implications to the resistance of structural elements

In order to assess the real behaviour of truss beam joints, a finite element modelling of joints is done. Also, the imperfections revealed in the execution/control phase are taken into account and determined the maximal stresses in the area of the affected joints.

The values of the Von-Mises stresses following FEM analysis of the joint with imperfections type 1 are presented in Table 4 and in Fig. 7.

Table 4. Imperfections type 1 – results.
Tabela 4. Greške tipa 1 – rezultati

Description	Element	Stress max Von-Mises	f_y	f_u
		N/mm^2	N/mm^2	N/mm^2
Eccentric HEA 140 diagonal	Diagonal HEA180	3803	2350	3600
	Diagonal HEA140	2916	2350	3600
	Chord HEA220	1585	3550	5100

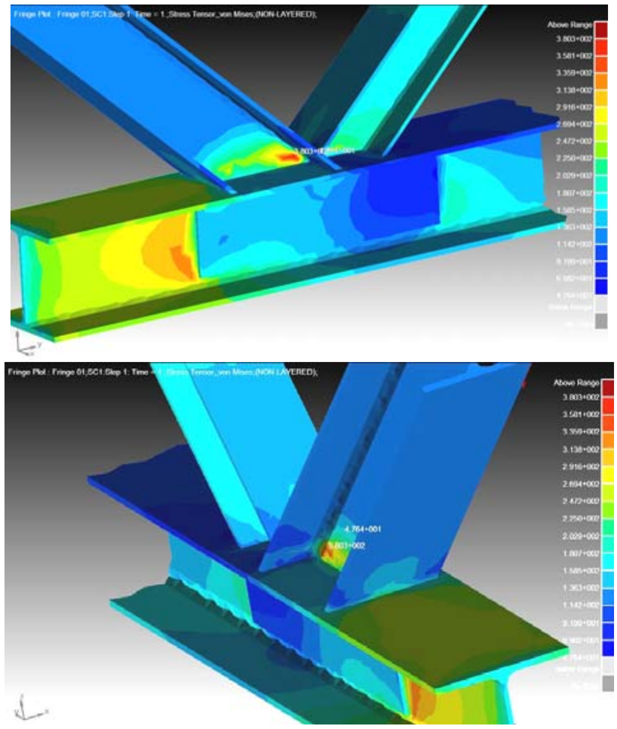


Figure 7. Imperfections type 1, FEM results, Von Mises stresses.
Slika 7. Greške tipa 1, FEM rezultati, fon Mizes naponi

Von-Mises stresses following FEM analysis of the type 2 imperfections joint are presented in Table 5 and in Fig. 8.

Table 5. Imperfections type 2 – results.
Tabela 5. Greške tipa 2 – rezultati

Description	Element	Stress max Von-Mises	f_y	f_u
		N/mm^2	N/mm^2	N/mm^2
Eccentric HEA 180 diagonal	Diagonal HEA180	3030	2350	3600
	Diagonal HEA140	2473	2350	3600
	Chord HEA220	3401	3550	5100

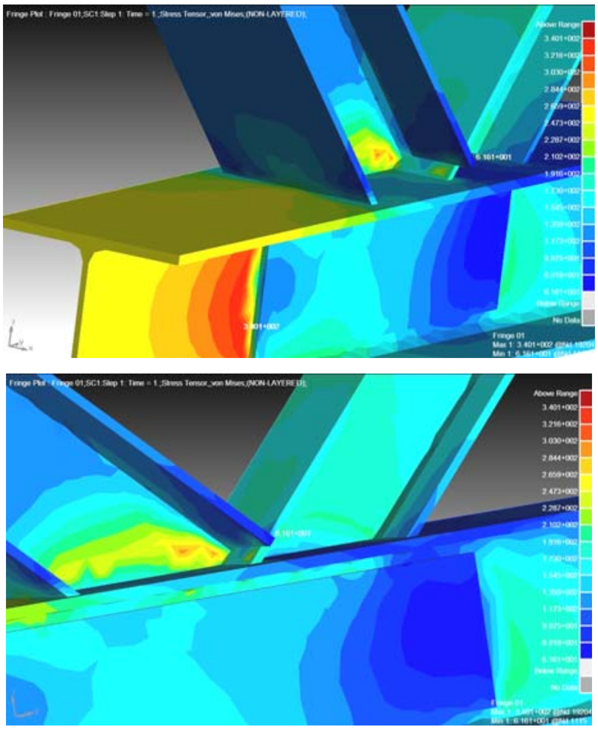


Figure 8. Imperfections type 2, FEM results, Von Mises stress.
Slika 8. Greške tipa 2, FEM rezultati, fon Mizes naponi

Strengthen solutions of the affected joints

Following several strengthening solution proposals, due to the ease of the erection (welding), a solution is adopted that can be done directly on site (on erected truss beams without dismounting), to the affected joints.

It was decided to weld additional gusset type plates with the role of redistributing the stress in the joint area, thus decreasing the stress to a value lower than the ultimate resistances.

Table 6 and Fig. 10 present values of Von-Mises stresses following FEM analysis for strengthened joint.

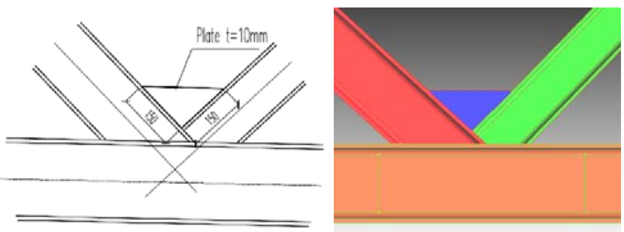


Figure 9. Strengthened joint – geometry.
Slika 9. Ojačani spoj – geometrija.

Table 6. Strengthening solution – results.
Tabela 6. Izbor ojačanja – rezultati

Description	Element	Stress max Von-Mises	f_y	f_u
		N/mm^2	N/mm^2	N/mm^2
Additional gusset	Diagonal HEA180	3339	2350	3600
	Diagonal HEA140	2490	2350	3600
	Chord HEA220	3300	3550	5100

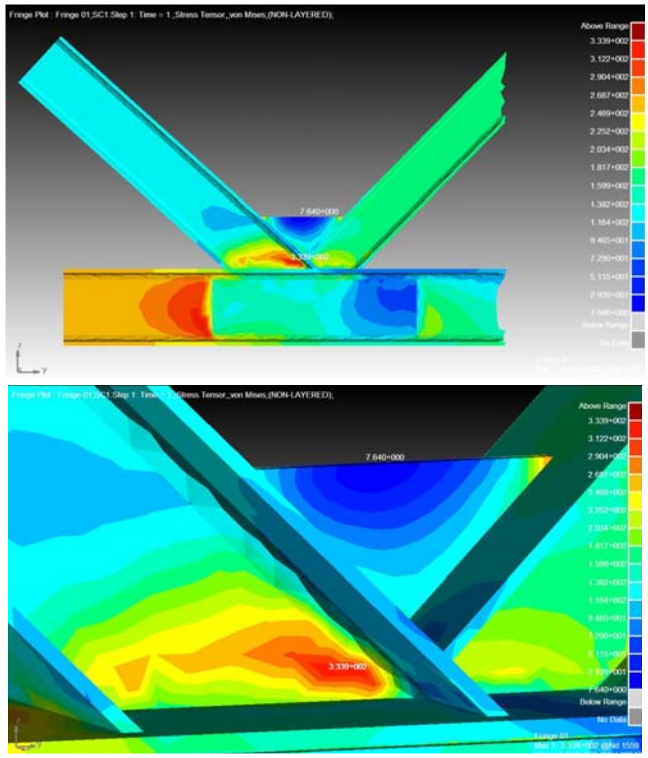


Figure 10. Strengthened joint, FEM results, Von Mises stress.
Slika 10. Ojačana veza, FEM rezultati, fon Mizesov napon

CONCLUSIONS

The manufacturing quality of truss beam joints is very important especially for HEA/IPE type elements truss beams. Imperfections in joints (eccentricity or welding flaws) can lead to stress values higher than the element capacity, affecting the strength and stability of the whole structural system in the end.

Another important consideration is the quality assessment of the steel structural elements. A better quality control in different phases of construction may be a matter of high importance.

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