

TRUSS BEAMS WELDED JOINTS STRENGTHENING SOLUTIONS

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Summary: *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.

Keywords: *welded joints, truss beams, welding imperfections, steel structures*

1. 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.

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2. DESIGN OF WELDED JOINTS ACCORDING WITH 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.

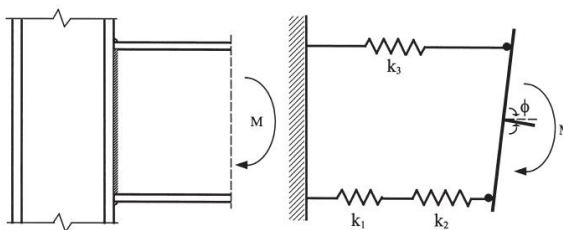


Figure 1. Welded beam to column joint – mechanical model, [3]

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 Figure 3. The two of the characteristics which are permitting the spring behaviour modelling are *axial rigidity* – K and the *plastic resistance* (Figure 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.

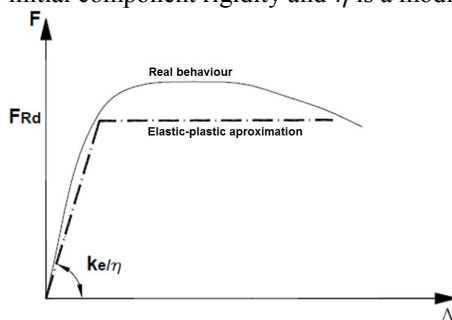


Figure 2. Idealizations of the force-deformation curve [2]



Figure 3. Modelling of a component under axial stress

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

From the components grouping point of view, for 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 of the minimum component to which is summed the deformations of the other components corresponding to the loading level. Through linear grouping of the 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.

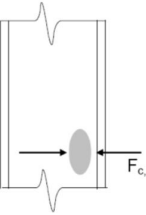
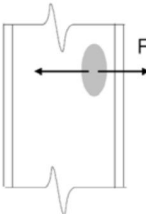
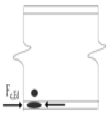
In case of any welded joint, the first step is to identify the active components which must be considered for the joints design. Thus for a welded beam to column joint there are the following active components:

- 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 (Table 1), there are presented the design formulas for a welded beam to column joint.

Table 1. Design formulas for the resistance design of a welded beam to column joint [1]

Component	Figure	Resistance design
1. Column web panel under share force		<p>According with EN1993-1-8 Ch. 6.2.6.1.</p> $V_{wp,Rd} = \frac{0,9 \cdot f_{y,wc} \cdot A_{vc}}{\sqrt{3} \cdot \gamma_{M0}}$ <p>where: A_{vc} is the shear area of column cross section (according with EN 1993-1-1). If column web stiffeners are used, then:</p> $V_{wp,Rd} = \frac{0,9 \cdot f_{y,wc} \cdot A_{vc}}{\sqrt{3} \cdot \gamma_{M0}} + V_{wp,add,Rd}$ <p>with $V_{wp,add,Rd} = \frac{4 \cdot M_{pl,fc,Rd}}{d_s} \leq \frac{2 \cdot M_{pl,fc,Rd} + 2 \cdot M_{pl,st,Rd}}{d_s}$</p> <p>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.</p> <p>The web panel resistance can be increased by adding stiffeners welded onto the web of the column, thus the shear area is increased by the area of the welded plate.</p>

<p>2. The column web under transversal compression</p>		<p>According with EN1993-1-8 Ch. 6.2.6.2.</p> $F_{c,wc,Rd} = \frac{\omega \cdot k_{wc} \cdot b_{eff,c,wc} \cdot t_{wc} \cdot f_{y,wc}}{\gamma_{M0}}, \text{but}$ $F_{c,wc,Rd} \leq \frac{\rho \cdot \omega \cdot k_{wc} \cdot b_{eff,c,wc} \cdot t_{wc} \cdot f_{y,wc}}{\gamma_{M1}}$ <p>where: ω is a reduction factor in accordance with share from the web panel of the column. The value of the ω factor depends directly of web share value, through the transformation parameter β and the ratio between the area from the compression resistance ($b_{eff,c,wc} \cdot t_{wc}$) and column share area A_{vc}; $b_{eff,c,wc}$ is effective width of the column web under compression which is 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 is needed to increase the compression resistance of the column web, can be applied two possibilities: a) increasing the column web by adding stiffeners onto the web panel of the column (as for the column web under share) with contour welding; b) adding transversal stiffeners in the area of the beams flanges (in this way stiffening the web of the column);</p>
<p>3. The column web under transversal tension</p>		<p>According with EN1993-1-8 Ch. 6.2.6.3.</p> $F_{t,wc,Rd} = \frac{(\omega \cdot b_{eff,t,wc} \cdot t_{wc} \cdot f_{y,wc})}{\gamma_{M0}}$ <p>where: ω is a reduction factor in accordance with share 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 which is calculated with 6.16 formulas [1] for the welded joints. $b_{eff,t,wc}$ represents the width of the column which is under tension from the beam web; The increasing of the column web tension resistance can be done with same solutions as presented at point 2 (in case of transversal compression of the web).</p>
<p>4. The beam flange and web under compression or tension</p>		<p>According with EN1993-1-8 Ch. 6.2.6.7.</p> $F_{c,fb,Rd} = \frac{M_{c,Rd}}{h-t_{fb}}$ <p>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 the profile flanges (considering that the resistance moment equals with coupling forces which are acting in the flanges axis).</p>

3. MANUFACTURING FLAWS. INFLUENCE AND STRENGTHEN SOLUTIONS

In the manufacturing process of the truss beams assemblies, even they are done in the workshop conditions, often the elements are presenting 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.

Table 2. Forces and profile types of the truss beam

Name	Description	Profile Type	Steel Quality	Axial Force
-	-	-	-	(kN)
GZ1-TS	Upper cord	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

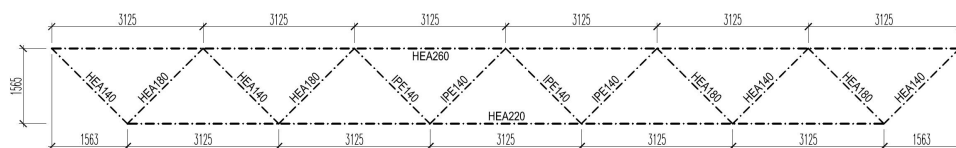


Figure 4. Geometry of the truss beam

In order to exemplify, is presented the case of a single storey building with a truss beam roof HEA truss profiles. The building has three openings of 25,00m and six bays of 18,75m.

The span of the presented truss beam is 18,75m with the geometry presented in the figure 4. Following the structural analysis, results the following profiles sections (table 2).

The joints of the truss beam elements are welded type and were designed according with the component method [1].

For case study it was taken the truss beam joint in which converge the D3 and D4 diagonal to the bottom chord. The joint is a welded type one with full penetrated welding for the flanges of the diagonals and with double corner welding for the web of the diagonals.

Table 3. Acting forces in the joint

Element	Axial Force	Joint Acting forces			
		(kN)			
-	(kN)	Nb1,Ed=	Nb1	:sin(α 1)=415,97kN	
GZ1-D3	588,27	Nb2,Ed=	Nb2	:sin(α 2)=411,55kN	
GZ1-D4	-582,02				

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.

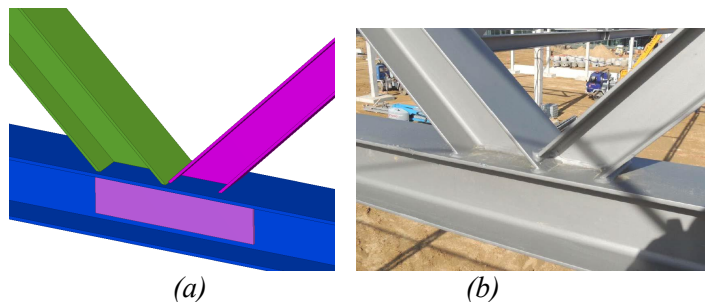


Figure 5. Truss beam joint (a) designed (b) manufactured – imperfection type 1.

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.

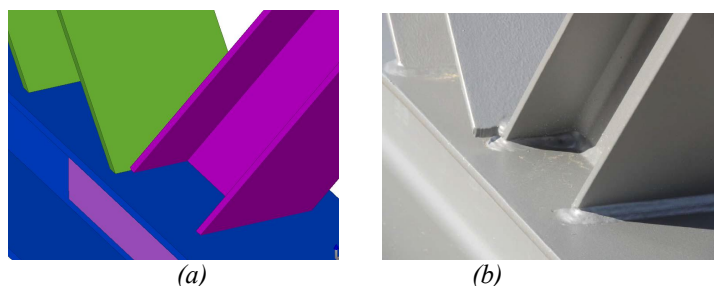


Figure 6. Truss beam joint (a) designed (b) manufactured – imperfection type 2.

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 [4] of the joint with imperfections type 1 are presented in Table 4 and in Figure 7.

Table 4. Imperfections type 1 – results.

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

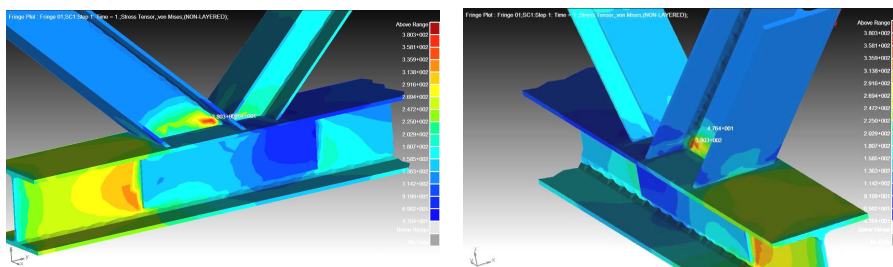


Figure 7. Imperfections type 1, FEM results, Von Mises stresses.

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.

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

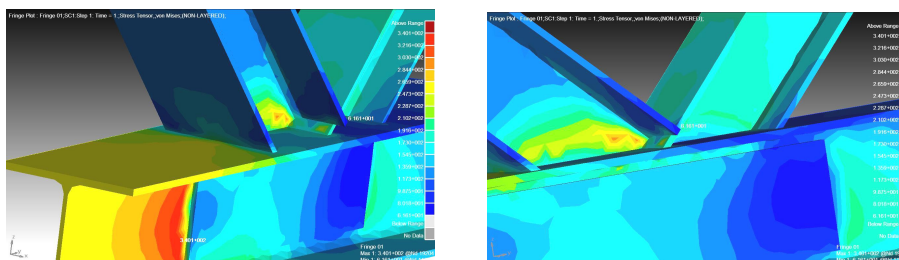


Figure 8. Imperfections type 2, FEM results, Von Mises stress.

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 dismantling), 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 figure 10 present values of Von-Mises stresses following FEM analysis for strengthened joint.

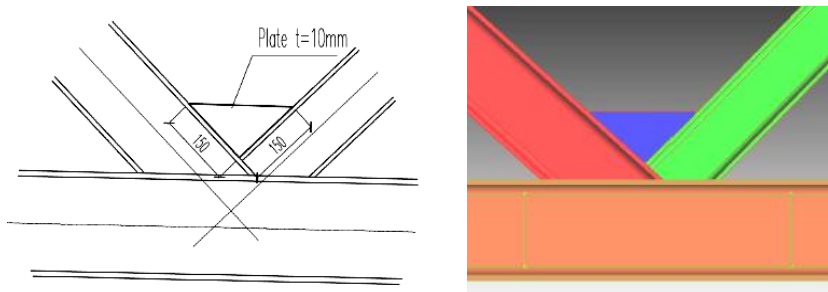


Figure 9. Strengthened joint – geometry.

Table 6. Strengthening solution – results.

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

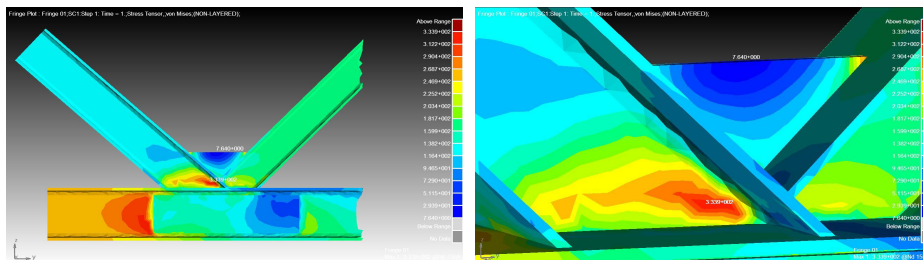


Figure 10. Strengthened joint, FEM results, Von Mises stress.

3. CONCLUSIONS

The manufacturing quality of truss beam joints is very important especially for welded joint type 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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РЕШЕЊА ЗА ОЈАЧАВАЊЕ ЗАВАРЕНИХ СПОЈЕВА

Rezime: У случају челичних конструкција са великим распонима, због лакоће производње и оптимизације тежине, све чешће се употребљавају заварени H или I профили. Да би се обезбедила снага и стабилност везе, врло је важно да се заваривање и положај дијагонала у горњем и доњем појасу изведе као у пројекту. У многим случајевима ови спојеви имају производне недостатке које доводе до високих напона у споју. У раду је приказан случај решетке од XEA профила дизајниране према EN1993-1-8. Израда открива ексцентритете у зглобовима и висок ниво несавршености. Представљени су ефекти несавршености и решења за јачање спојева.

Кључне речи : заварени спојеви, стубови, заваривање, несавршености, челичне конструкције