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Finite element calculation of redesigned welded joint at support for frame stage-like structure

Aleksandra Arsić[a], Željko Flajs[b], Vlada Gašić[a], Nenad Zrnić[a]

[a]*University of Belgrade, Faculty of Mechanical Engineering, Serbia*
[b]*Institute for Materialls Testing, Belgrade, Serbia*

Abstract *A numerical analysis of welded connection between two steel structures is performed with the usage of the finite element approach. A frame stage-like structure is placed between the main horizontal girders of the structure of the hall. A visual inspection of the frame revealed numerous irregularities of welds which brought demand for redesign and overall check. The emphasis in this paper is comparative calculation of initial and redesigned welded end-joints, without imposing the optimal technical solution. It is used modern and particular finite element software for calculation of welded joints. The values of stresses in the welded joint as well in plates and members are lower than the permitted values for both models. However, the influence on the main structure is reduced with redesigned technical solution. The level of stresses is lower along with reduction of rotational stiffness of the joint. This can be considered as a relief to the main girders of the hall structure and should be a cost and a goal for supporting any additional structure.*

Keywords *visual inspection, FEA, welded joint, stresses, stiffness*

1. INTRODUCTION

As part of the reconstruction and modernization of facilities at the Belgrade Fair, hall number 5 was planned to be equipped to hold various fair events with modern equipment.

Positioning and movement of various equipment that is necessary during the modern manifestations is ensured by the appropriate additional frame stage-like structure made of steel S235JR. It has built-in hooks for hanging steel ropes or chains for carrying equipment which can weigh up to 1 t for all load combinations. An additional steel structure has been placed in the attic space at the level of the bottom flange of the profile of the main horizontal girders of the steel structure of the hall, Fig. 1.

The structure contains two rows of longitudinal steel girders - UPN 240 with welded cross girders - UPN 180 and diagonal stiffening of box cross-sections with dimensions 60x60x3 mm. Fig. 2 depicts the linkages between the longitudinal and cross profiles of the structure and its connections to the main support of the hall frame.

The primary supporting steel structure of the hall was built in 1971 and made of structural steel which is assumed to correspond to S235JR. The main frame of the hall is a welded I profile 1040 mm high with 20 mm thick flanges and a 10 mm thick rib.

Corresponding author

MSc, Aleksandra Arsić
aarsic@mas.bg.ac.rs

University of Belgrade, Faculty of Mechanical Engineering
Kraljice Marije 16
Belgrade, Serbia

Fig. 1. Roof structure in the attic space

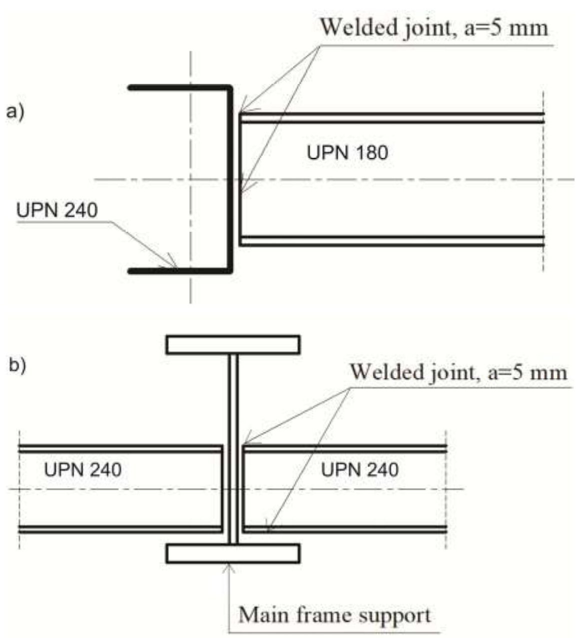

Fig. 2. a) Joints of longitudinal and cross profiles; b) The joint of the longitudinal profiles and main girder

After non-destructive testing (visual inspection) of the welded joints of the additional steel structure, shown in chapter 2, it was determined that the quality of the welded joints does not meet the acceptance criteria prescribed by the standard. Its reconstruction was carried out, whereby the connection of the additional structure to the main frame of the hall was modified. Due to the new specific design of the joint through the stiffening plates, a numerical analysis of the welded joint was performed and presented in chapter 3. In order to compare the results, a numerical analysis of the initial way of supporting the structure was also carried out.

2. NON-DISTRUCTIVE TESTS OF WELDED JOINTS OF ADDITIONAL STRUCTURE

Properly carried out technical diagnostics of the condition of the elements and profiles of the steel frame, intended for carrying equipment for fair events in hall 5, using non-destructive testing, ensures the strength of the quality of the structure as a whole, provided by the project, without the possibility of immediate damage, safe work for employees, rational techno-economic exploitation, maintenance, and environmental protection.
After 100% visual testing of all available welded joints of the steel structure [1], the following irregularities were found:

- The imperfect shape of the external surfaces of the welds or incorrect geometry of the welded joints on the cross supports and diagonal stiffeners.
- Excess weld metal on the face of butt welds - big bulge on cross members and diagonal stiffeners.
- Linear shearing of parts on longitudinal members.
- Angular shear – the planes of the surfaces are not parallel on the longitudinal members.
- Insufficiently filled groove on the longitudinal members.
- Uneven weld widths on longitudinal members.
- Incompletely welded joints between the longitudinal members and the main frame support.
- Splashing of molten metal on the cross members.

Several mentioned irregularities in the zones of the tested welded joints are shown in Figs. 3-5.
Based on the performed tests, it was determined that the quality of the welded joints does not meet the acceptance criteria prescribed by the standard [2]. Stress concentrators, generally, appear at locations

with radiuses inadequately defined during the design process and at welded structures.

Various technological, constructional, and exploitation factors influence welded constructions to have a possibility for high level of stress concentration on the fillet and butt welds with multiple errors. The defects in the welded joints were such that it was necessary to replace the relevant elements and profiles with new ones. Due to that, it was necessary to make changes to the observed construction.

Fig. 3. Irregularity in the butt-welded joint (uneven width of the weld)

Fig. 4. Irregularity in the butt-welded joint (excess weld metal on the face of the weld)

Fig. 5. Irregularity in the welded joint (insufficiently filled grooves)

The main point of support of the additional structure for the roof supports of the hall is completely reconstructed. The emphasis here is only on comparative calculations of joints between the main structure and the additional structure, without imposing the optimal technical solution. Hence, it is performed strength and stiffness analysis of the initial and redesigned end-joints.

3. NUMERICAL ANALYSIS OF A WELDED JOINT

Numerical analysis was performed for two models of the welded joint between the main girder (welded I section) and longitudinal beam of the additional structure (UPN240).

The first model (M_1) represents the initial way of supporting the additional structure - welded on the bottom flange of the main frame profile (Fig. 2b), while the second joint model is welded joint over steel plates for the rib of the main frame profile (M_2) shown in Fig. 6. Numerical analysis of both models was done in the software package IDEA StatiCa [3].

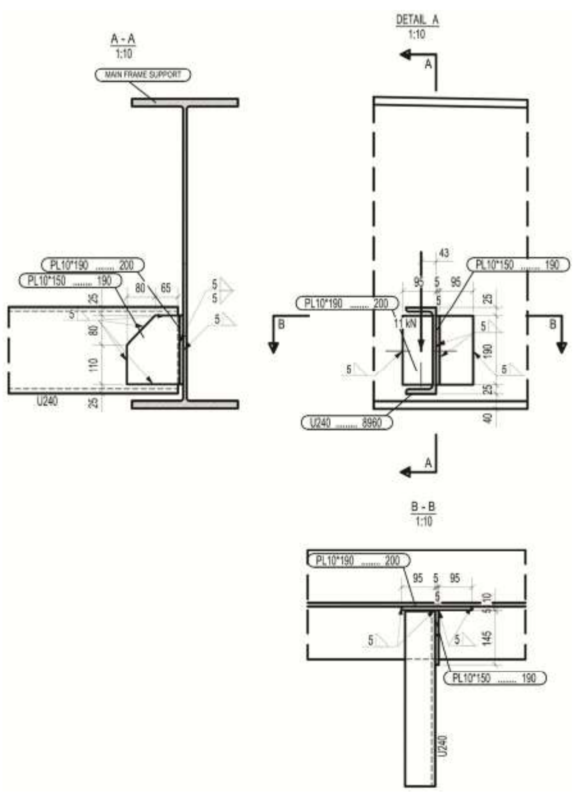

Fig. 6. The details of joint model M_2

IDEA StatiCa combines FEA with the analytical method of joints based on the component method (CM) specified in EN 1993-Part 8 (EC3) [4]. It can be used for structural evaluation or design of a variety of welded and bolted structural steel connections and base plates. This type of custom-made, semi-automated and specialized software are easy-to-use and sufficient for daily engineering practice [5]. Modelling in this software considers an elastic-plastic material model for all the elements of the joint while the steel members are meshed with 4-node quadrilateral shell elements. The welds are modelled as a special elastic-plastic element on the multi-node constraint (MPC). The software by default use approximation of 8 elements in the critical edge with mesh size between 50 to 10 mm and limit plastic strain of 5%. The joint models are shown in Figs. 7-8.

According to the conceptual design of the additional structure, which is out of scope of this paper, all combinations of loads from modern equipment intended for fair events give a maximum vertical reaction on the main girder with the value of force of 11 kN. The force is applied at the joint node and only conceptually (by the software) depicted to act on the free end of the UNP240 (e.g. on Figs. 9-10).

Global stress states as well as the maximum stress values in welds for joint models M_ 1 and M_ 2, respectively, are given in Fig. 9.

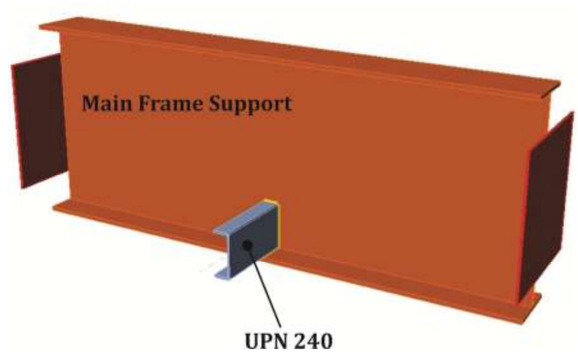

Fig. 7. Joint model M_1

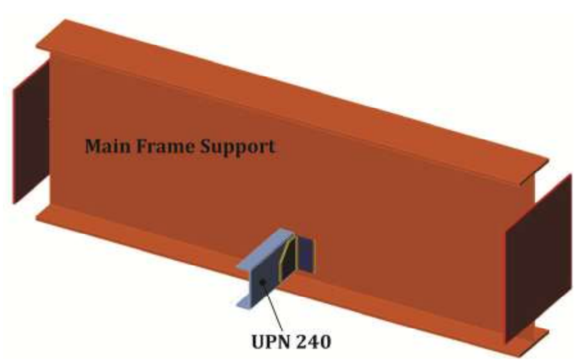

Fig. 8. Joint model M_2

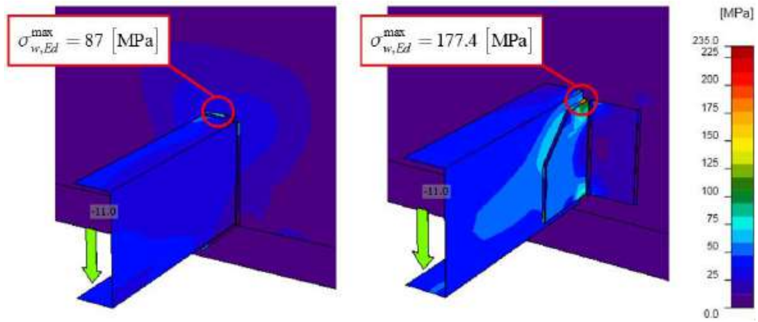

Fig. 9. Maximum equivalent stress of welds: M_1, M_2, respectively

Numerical analysis showed that the values of the equivalent stresses of welds for both models are lower than weld resistance. The permissible stress value for fillet welds is 360 MPa and is determined according to Eq. (1), [4].

$$\sigma_{w,Rd} = \frac{f_u}{\beta_w \cdot \gamma_{M2}} \tag{1}$$

Where $f_u = 360\ \mathrm{MPa}$ is the nominal ultimate tensile strength of the weaker part is joined; $\beta_w = 0.8$ is the appropriate correlation factor; and $\gamma_{M2} = 1.25$ is the partial safety factor, [6].

A comparative analysis of the stress states of models 1 and 2 shows that in M_2 the rib of the main frame support has significantly lower stress values. The highest stress value on the rib of the main support in the first case is 46.7 MPa, while in the second case, the highest value is 13.5 MPa. The elementary structure of hall 5 was relieved by the new method of supporting the additional structure. That is evident from the numerical analysis results, which show that the highest stress values now occur on the additional plates.

The highest equivalent stress in the first model, M_1, occurs on the bottom flange of the UPN profile at the point of contact with the bottom flange of the main support, while in the second model, M_2, the highest equivalent stress is in the upper zone of the connecting plate of the longitudinal element of the additional structure. The exact values and zones for maximum stresses are shown in Fig. 10. Indeed, the values are lower than the yield stress for material S235. According to national regulations, the limit value of stresses can be taken as 157 MPa which is also satisfied.

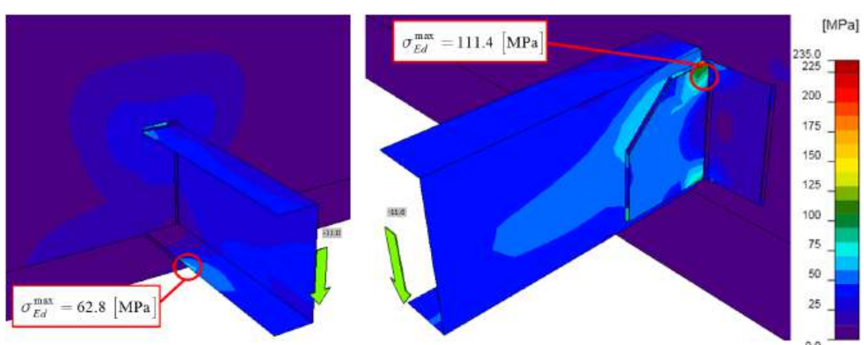

Fig. 10. Maximum equivalent stress in plates: M_1, M_2

Generally, a joint may be classified as rigid, nominally pinned or semi-rigid according its rotational stiffness. The pinned joint (or flexible) permits rotation of connected parts (ϕ), not transmitting bending moments between the elements of the frame [7]; on the other hand, the rigid connection forbids the relative rotation between the elements and, consequently, transfers bending moments [8]. Semi-rigid joints should be capable of transmitting the internal forces and moments [9]. Therefore, a stiffness analysis was

performed for the given models. The software classified both models as pinned joints, while the moment-rotation behaviour curves are given in Fig. 11.
We can see from the M - ϕ diagram that with the new solution of joint between the two structures, the joint is considerably weakened in terms of stiffness (the initial stiffness, $S_{j,ini}$, is double reduced in the second model).

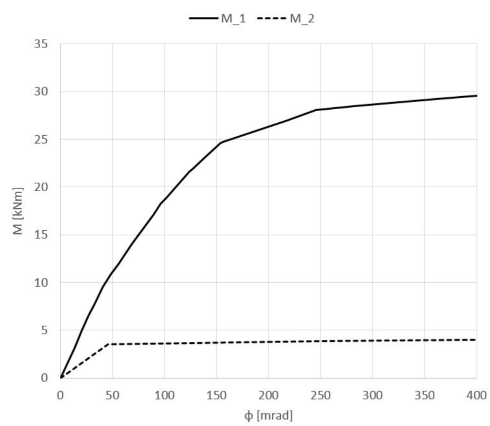

Fig. 11. Stiffness diagram M - ϕ: M_1, M_2

This indicates that the main girder of the hall is much relieved because the load is now carried by the so-called pinned joint which does not allow the load to be transferred further to the structure.

4. CONCLUSION

The numerical analysis of joint behaviour between the two structures is carried out. The visual inspection of the initial frame stage-like structure and a way of joining revealed numerous irregularities which brought demand for redesign and overall check.
By the usage of particular finite element analysis, it is performed comparative calculation of initial and redesigned welded end-joints. It is shown for both the models that values of stresses on members and welds do not go beyond permissible values. The level of stresses in the main girder is lower with redesigned joining solution, along with reduction of rotational stiffness. This could be important for the main girders when there is no informations if civil engineer (long time ago) has taken in account additional loads from secondary structures. The results for redesigned joining solution can be considered as a relief to the main girders of the hall structure. According to the authors, this should be always the goal for supporting any additional structures on the long-lasting civil structures.

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