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Paper ???

Local and Global Stability Analysis of a Large Free Span Steel Roof Structure

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Abstract

The global stability checks and specific local stability problems of a large span steel space truss made by square hollow sections (SHS) are presented in this paper. For the fabrication of the space truss, welded joints between the SHS profiles were designed. As a result of the fact that for characteristic failure checks of the welded TT and KK joints analytical methods are based only on a semi-empirical formulae, developed for Φ=90 degrees (the angle between the diagonal planes), for the design of joints finite element modelling was used. Good agreement between the results of the developed finite element joint model and the analytical method for TT and KK joints has been found, even though the semi-empirical formulae are applied for the analysed truss which had Φ=50 degrees.

Keywords: large span space trusses, hollow sections, TT and KK welded joints, local and global stability analysis, finite element joint modelling.

1 Introduction

In 2010 the local authority of the city of Cluj launched the selection based on a Feasibility Study for the new City Sports Centre with a capacity more than 7000 fixed seats, to be located near to the new city stadium. Following the selection process, which included a new structural solution proposal, a designer association was selected including architects, structural engineers, mechanical and electrical engineers. Based on the new structural concept, Gordias Ltd. was appointed for the design of the large span roof structure.

The architectural design process started in March 2011, whilst some changes were produced on the level of facilities to be included within the Sports Centre, in particular whether or not an ice rink to be included. The structural design process of the roof structure started in July 2011 and was completed in September 2011.

The article describes the applied structural solutions and gives details about the local and global stability analysis of the structural members and the joints under different load combinations. The paper content is limited only to the large span roof structure made by steel SHS profiles with welded joints and summarizes the results of the numerical study performed by the authors.

2 Building description

2.1 Architectural facts

The City Sports Centre with a capacity more than 7000 fixed seats is organized on five levels: one underground level, ground level and three stories (Figure 1, 2).

Underground level is exclusively for parking with 447 car capacities. Ground level, first and second level includes public, officials, media, shopping and administration area. Ground level also includes the reserved area for sport players, second level includes a conference centre. The third level is reserved for media, special equipment and installations.

To allow multiple use of the playing area, the surface can be changed from 30×47 meter to 38×56 meter using extensible / retractable tribune structure on ground level. In this way basketball, handball, volley, box, gymnastics championship are possible to take place in the City Sports Centre.

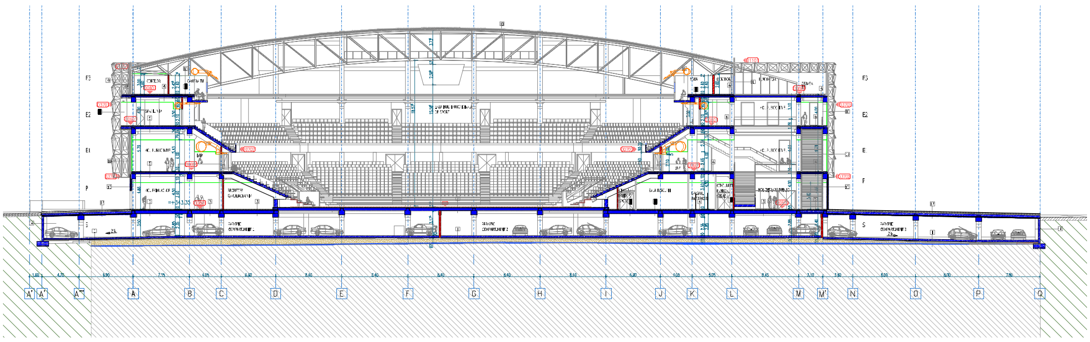


Figure 1: Transverse section through the building

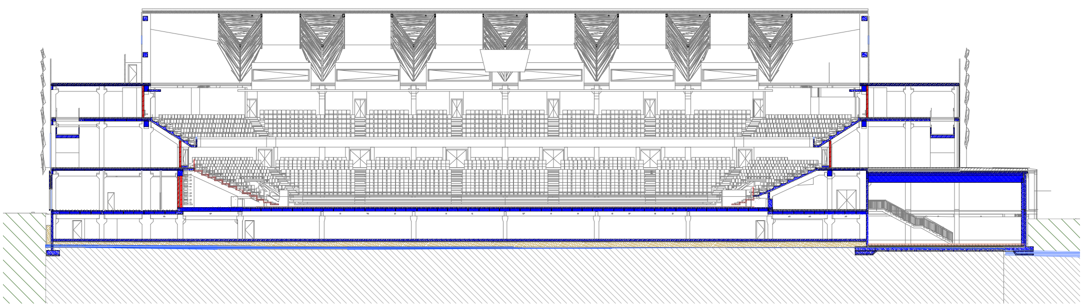


Figure 2: Longitudinal section through the building

2.2 Design loads of the roof structure

In order to evaluate the structural response, in the design process were considered the following loads (characteristic values):

- Roof loads (EN1991-1-1): dead load + technological load q_k = 1.0 kN / m^2
- Concentrated technological loads (media box) applied in mid span, P_k= 60 kN
- Temperature variation effects between erection phase (-15 to -20 degrees in winter) and service phase (+15 to +20 degrees during lifetime)
- Snow loads on the roof according to CR 1-1-3-2005 (EN1991-1-3), $s_{0,k}$=1.5 kN/m^2
- Wind loads on building envelope according to NP-082-04 (EN1991-1-4), q_{ref}=0.4 kN/m^2
- Seismic action according to P100-2006 [1] (EN1998-1), with peak ground acceleration a_g=0.08g and control period of seismic motion T_c=0.7 sec
- Load combination for ultimate limit state (ULS) and serviceability limit state (SLS) according to CR-0-2005 [2] (EN 1990).

2.3 Structural solution and conceptual design of steel structure

The sports centre scheme features one main hall with free roof span of up to 63.90 m, having the total width of 76.10 m. Selection of the most appropriate roof structure has been driven by a number of factors including the span, roof geometry, load to be supported, economy and aesthetics and the use of sustainable construction materials.

Triangular shaped spatial trusses at 10.60 m centres were design (Figure 2). The truss section is 3600 mm wide by approximately 4000 mm deep (variable along the span). The steel trusses of this size are able to span such distances as simply supported elements, but large vertical deformations and horizontal reactions were necessary to be managed. Several options were investigated to keep the sizes to a minimum including arch action (which would have required stiff concrete structure) and cantilevered truss structure (which would have resulted in vertical tie elements at both ends). The system that was decided combines both: the advantage of this system is that the end cantilever with the vertical tying elements of the truss effectively reduces the vertical deformations and axial forces in mid span, as well as allowing for continuity of the roof structure over the lateral annexes. The principle was successfully used by the authors in a similar project [3]. Pinned supports were provided on the top of the concrete columns, with limited horizontal displacement set up possibility for the erection phases. Longitudinal trusses were placed in mid span, at the supports and near the supports, where the inner flange of the truss compression effort change in tension.

The design of the steel structure was performed following the European standards. For strength, stability and stiffness requirements of the structural elements the prescription of SR-EN1993-1-1 [4], SR-EN1993-1-8 [5] and P100/2006 [1] were used. For the design of the structural elements, linear elastic structural analysis was performed. The design checks of the structural elements for ULS include persistent or transient design situations (fundamental combinations), where snow loads in combination with technological loads play the key role. For global stability checks Consteel software (www.consteel.hu) was used, which calculation procedure is based on the general method of EN1993-1-1 [4]. For individual member checks, both method A and B of EN1993-1-1 was also performed. Good agreement between the used methods was found. For the design of spatial truss joints the following failure modes have been considered: local brace failure (yielding, local buckling), chord face plastification, chord punching shear, chord side wall failure, chord shear failure [6]. Alternative checks using finite element modeling was also used.

For SLS design checks of the structural elements, fundamental and exceptional load combinations were used. The computed maximum vertical deflection of the space truss for SLS check under snow and technological load is:

$$f = 165mm \leq f_a = \frac{L}{300} = 213mm \tag{1}$$

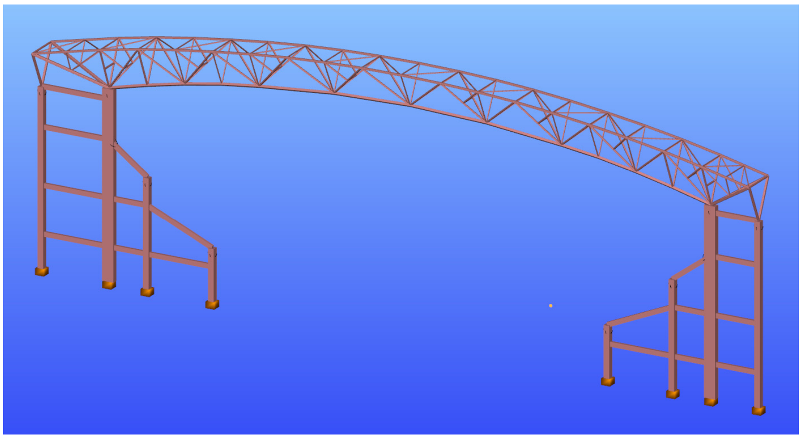


Figure 3: The structural model of the transverse frame

3 Particular problems in the design process

3.1 Connection between the steel and concrete structure

Providing pinned supports for the roof trusses, beneficial effects in the internal effort distribution and the highest horizontal reactions over the concrete structures was obtained. The use of simply supports for the roof trusses eliminates the horizontal reaction over the concrete structure, but has a negative influence over the roof trusses in terms of vertical deflection and effort distribution. Final solution for the connection (Figure 4) considered a combination of both effects: a limited sliding possibility of the support, used only for the erection phase of the structure (mainly deformations caused by the permanent actions are consumed), after that the support is transformed in a perfect hinge. In this way a lower horizontal support reaction resulted, consuming around 30 mm horizontal displacement by the sliding possibility of the support, from the total of 80 mm, calculated under permanent, technological and snow loading.

Figure 4: Sliding possibility of the support and view at support level

3.2 Global stability checks of the space truss structure

The global stability checks of the space truss were performed using Consteel software [7]. Buckling analysis according to the general method of EN 1993-1-1, including only the 3D effects of the structural members were performed (stressed skin beneficial action of the roof was neglected).

To have an overview about the global behaviour of the structure, the space trusses were calculated and checked using the full 3D model of the structure, including also the concrete structure. According to the buckling analysis, a critical load multiplication factor of $\alpha_{cr,op}$=3.63 was computed (Figure 5), corresponding to the stability loss of first compressed diagonals. Improving the model, considering the effects of additional braces of the compressed diagonals (Figure 6), the critical load multiplication factor increased at $\alpha_{cr,op}$=6.81.

Alternate design checks of the space truss were performed, using in a conservative way the simply supported configuration of the roof structure (concrete structure was not considered in this model), only the stabilizing effect of the longitudinal trusses placed over the support and in mid span.

A fair agreement between the 3D full model and simplified model was found. According to the buckling analysis, a very similar buckling shape and the associated critical load multiplication factor of $\alpha_{cr,op}$=4.04 was computed. Improving the simplified model, considering the effects of additional braces of the compressed diagonals, the critical load multiplication factor increased only at $\alpha_{cr,op}$=5.05, due to the stability loss of upper compressed chord in mid span, which was not present in 3D model, due to the moving possibility of the neglected concrete structure. The resulted degree of utilization for the members, following the general method of EN 1993-1-1 resulted less than 80% in this way, while considering the simply supported configuration of the roof truss structure the level of almost 100% is achieved.

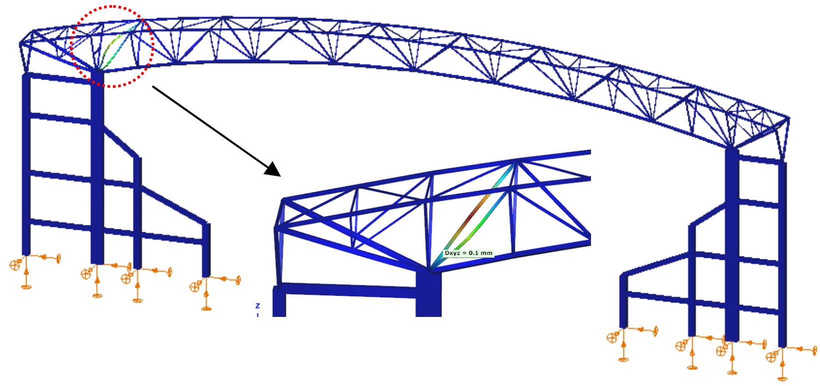


Figure 5: Compressed diagonal buckling in the space truss ($\alpha_{cr,op}$=3.63)

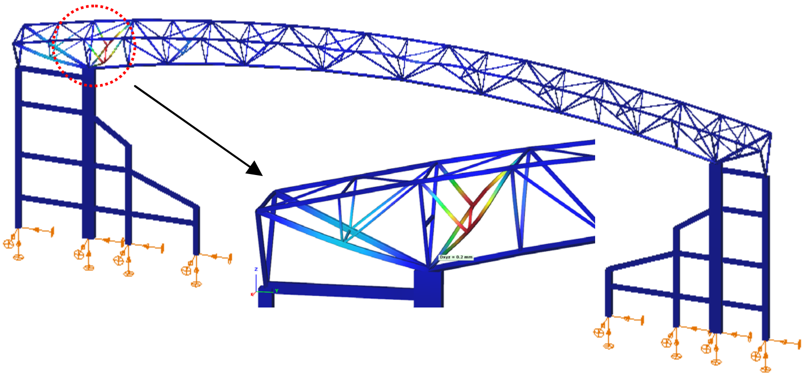


Figure 6: Improved space truss ($\alpha_{cr,op}$=6.81)

3.3 Joint checks using FEM of the space truss structure

The main concern after global stability analysis was to find out which is dominant: the global stability loss of the chords or the local stability issues in the welded joints of the truss? Local brace failure (yielding, local buckling), the chord face plastification, chord punching shear, chord side wall failure, chord shear failure checks are necessary according to [6] for TT and KK joints. The analytical methods given by [6] for TT and KK joints have limited range of validity, because the semi-empirical formulae are developed for Φ=90 degree (the angle between the diagonal planes). To have a satisfactory response, finite element modelling of the heaviest loaded truss joint was used.

For an accurate analysis including geometrical and material nonlinearities of the selected joint (Figure 7), a specific finite element model was developed (see Figure 8). Nonlinear elastic-plastic analysis considering geometric nonlinearities (GMNA and GMNIA) has been applied. The model was developed in ABAQUS [8] finite element program, using S4R-type elements (4-node shell elements with reduced integration) with 6 degrees of freedom on each node (translation and rotation with regard to the x, y and z-axis).

All strengthening plates and SHS sections were modelled in their mid-plane and the connections between chords and truss flanges were defined as a surface-to-surface tie between SHS section end – strengthening plates (Figure 8). The model's materials were defined as elastic – linear plastic ($E = 210000$ N/mm^2, $\upsilon = 0{,}3$) with a yield strength of $f_y = 275$ N/mm^2 for all the rectangular hollow sections. The loads from permanent, technological and snow actions were introduced as point loads, relationship between the magnitudes was considered according to the statical model. First buckling shape with the amplitude according to EN1993-1-6 was used as initial imperfection in the nonlinear-elastic analysis (GMNIA).

According to the analysis results, the local stability loss of the compressed diagonal members connected in the joint will define the joint capacity. This is associated with the brace failure (BF) in the analytical method. The computed load multiplication factor according to the level of dominant load case (permanent + technological + snow loads) resulted $\alpha_{cr,op}$=8.18, which is higher than those obtained in the global stability analysis of the truss structure.

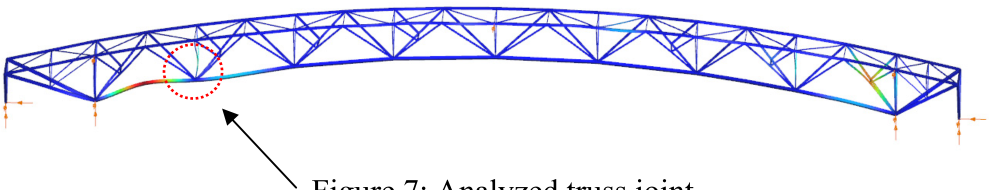


Figure 7: Analyzed truss joint

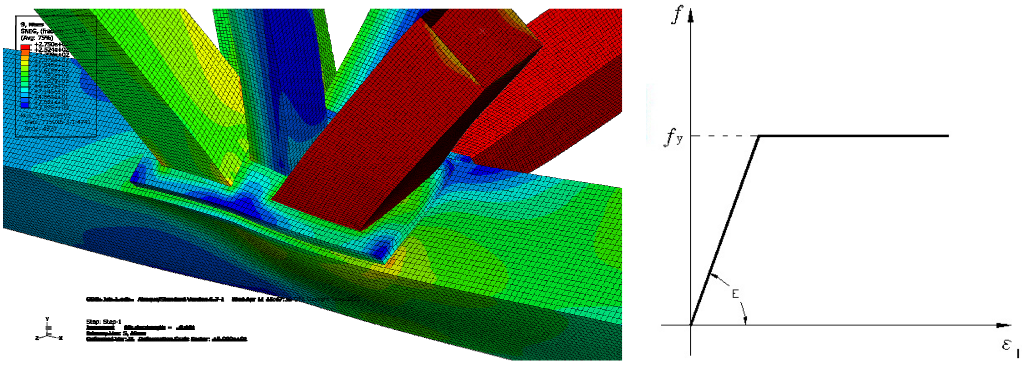


Figure 8: FEM of the selected joint and material model

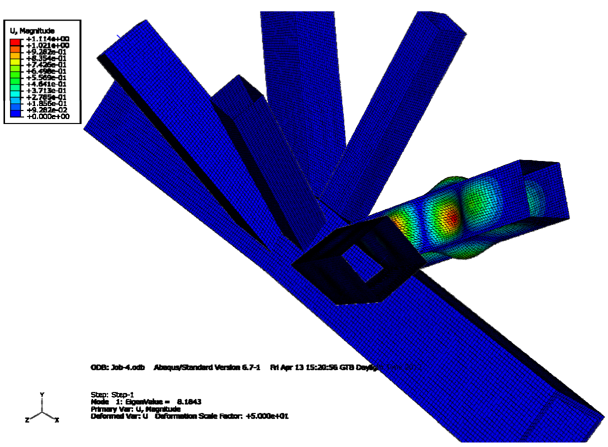
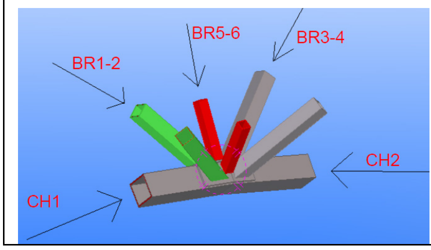


Figure 9: First buckling mode of the joint ($\alpha_{cr,op}$=8.18)

3.4 Joint checks using analytical method

Analytical models for the checking of the space truss joints were also applied. The chord face plastification (FP), chord punching shear (PS), brace failure (BF) and chord shear failure (CS) checks according to [6] for TT and KK joints have been evaluated. Strengthening plates were neglected in the evaluation. Table 1 centralizes the evaluated results for the first low chord joint from the main support, similar with the one used in the FEM. It can be seen that dominant component of the compressed brace BR1-2 is the brace failure (BF) mode, closely followed by chord face plastification (FP).



Failure type	BR1-2 [kN]	BR5-6 [kN]	BR3-4 [kN]
FP	781.5	780.1	999.6
BF	752.4	475	1274.4
CS	1685.8	1861	2173.4
PS	1164.1	1824	1384.7
Design efforts N_{Ed}	563.0	94.0	588.1

Table 1: Analytical results on low chord joint

3.5 Discussions

Comparing the analytical results with those obtained from the FE analysis with strengthening plates applied on the chord face, similar failure mode (brace failure due to local buckling) has been identified: Figure 9 shows the brace failure due to local buckling of the SHS profile walls, in Table 1 we can see the lowest capacity of the brace BR1-2 is given by the same failure mode as identified in FE analysis. Due to this failure mode, the supplementary stiffener plates are not contributing to the global capacity of the analysed joint; it helps only to obtain accurate perimeter welding for all the connected braces. The chord can carry the design efforts safely, even without stiffener plates, but more complicated execution details will result. According to evaluated results semi-empirical formulae, developed for Φ=90 degree which is the angle between the diagonal planes gives satisfactory results also for Φ=50 degree.

A FEM was performed for the joint without strengthening plates. Comparing the analytical results with those obtained from the FE analysis without strengthening plates, similar failure mode (brace failure due to local buckling) has been identified. Increasing the applied loads, the brace failure was followed by the chord face plastification (FP) (Figure 10).

As a conclusion we can say that FEM results and used analytical models for KK and TT welded joint design gives fair agreement for this particular space truss with Φ=50 degree, even the analytical models were developed for space trusses with Φ=90 degree (Φ is the angle between the diagonal planes).

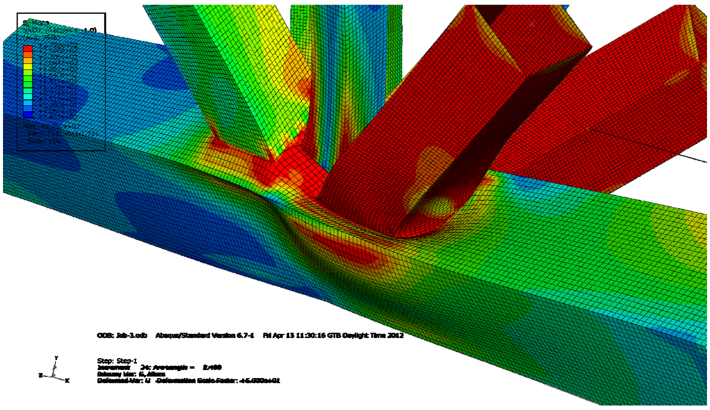


Figure 10: Analysis result without strengthening plate

4 Fabrication and erection of the space truss structure

The transportation of any truss from the steel shop - where the assembly is produced – to the site to be used, can be a very expensive process. The global dimension of one truss (76.1 m), imposed to be cut in five pieces (Figure 11). Due to the expensive transportation of the first truss, for the next six only the assembly parts were cut in the shop, the truss assembly welding was switched on site. The erection process started when the first two trusses were ready.

In the erection phase the end parts was positioned first, followed by the intermediate three subassemblies positioning as a single one (Figure 12). For all these intermediate phases, the stability of the parts has been checked. Without any significant loads (only self weight), the truss only with end lateral supports had a comfortable computed critical load multiplication factor of $\alpha_{cr,op}$=9.58.

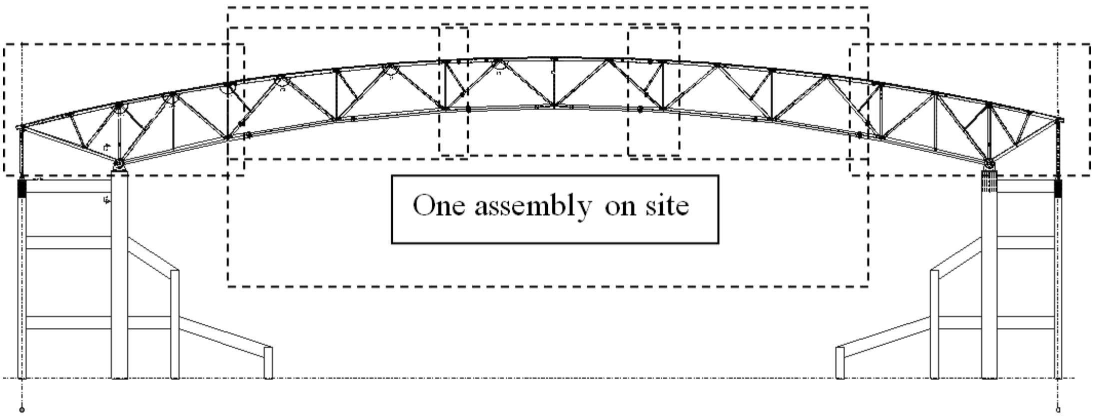


Figure 11: Truss splicing for transport

Figure 12: Truss erection on site

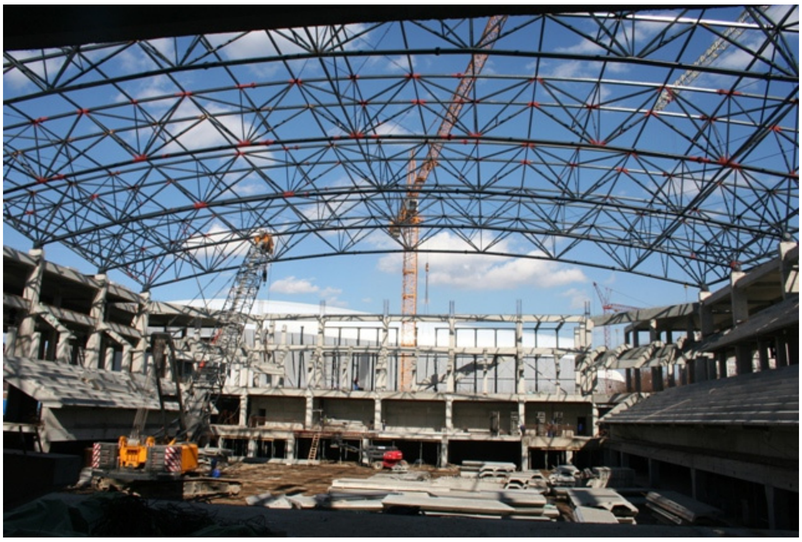


Figure 13: All trusses on the final position

5 Conclusions

This paper describes the key aspects of the local and global stability design checks for a large free span roof truss structure. The use of Consteel software based on the general method of EN1993-1-1, gave the possibility to the designer to use simple – step by step design strategies to improve the critical load multiplication factor, identifying and improving the weakest element in the truss structure and to make stability checks for intermediate technological phases, in which the truss is only partially fixed.

Using the finite element method, complex welded joint configuration specifically used in space trusses made by SHS tubular steel sections were analysed. Fair agreement between finite element method and analytical methods according to [6] for TT and KK welded joints have been found, even semi-empirical formulae, developed for trusses with Φ=90 degree where Φ is the angle between the diagonal planes, to give satisfactory results also for the analysed joints of the particular truss which had Φ=50 degrees.

References

[1] P100-2006: Romanian Earthquake Design Code

[2] CR-0-2005: Cod de proiectare pentru bazele proiectarii structurilor in constructii (Design Code. Basis of design. Romanian design code).

[3] Zs. Nagy, I. M. Cristutiu: Advanced nonlinear investigations of a 50 m span frame case study: The steel structure of the ice rink, city of Târgu Mureş, Romania, Stability and Ductility of Steel Structures, SDSS-2010, Rio de Janeiro, Brazil, 8-10 September 2010, pp 649-655 (2010);

[4] SR EN 1993-1-1 Eurocode 3: Design of steel structures Part 1.1: General rules and rules for buildings;

[5] SR-EN1993-1-8: Eurocod 3: Design of steel structures Part 1.8: Design of joints

[6] J. Wardenier, J.A. Packer, X.L. Zhao, G.J. van der Vegte: Hollow Sections in Structural Applications, Comité International pour le développement et l'Etude de la Construction Tubulaire, ISBN 978-90-72830-86-9

[7] Consteel v.6.0 user manual (2011), Developed by Consteel Ltd. Budapest

[8] Abaqus, v.6.5.1, Dassault Systems/Simulia, Providence, RI, USA (2008)