A long span structure in Romania

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Summary

The authors will describe in the following pages the reasons why the project of the Multi-functional Sports Hall from Cluj Napoca is attractive. The main lines of the building are: a hall with a capacity of 7000 seats, a structure dominated by precast concrete elements, a long span roof and an advanced analyze of the connections, all well-kept into the limited funds. The roof solution consists in using steel space trusses made out of square hollow sections (SHS). The truss has a clear span of 63.90m, a total length of 76,10m, a maximum height of 4,00m that is reduced on the length of the structural element, and a triangular cross section being 3,60m wide. Global stability checks and specific local stability problems were performed and are exposed in the following 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 $\Phi=90$ 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 $\Phi=50$ degrees.

Keywords: large span space trusses; precast concrete; joint analysis.

1. Introduction

In 2010 the local authorities of the city Cluj Napoca launched the selection process based on a Feasibility Study for the new Multi-functional Sports Hall, located near the new city stadium. Following the selection process which included a new structural solution proposal, a designer association was selected including architects, structural engineers and electrical engineers. Based on a new structural concept, Plan 31 Ro Ltd. was elected for designing the building structure.

The technical solution regarding force resisting structure was decided after taking in consideration a couple of monolithically concrete frames and a roof made out of laminated wooden arches. The monolithic concrete had the disadvantage of reduced time efficiency, and after the future owner decided that it would be better to reduce the time with execution from 36 months to 12 months in order to be able to organize a European Sport Competition, the design team had to find new solutions. The building has 115m x 130m in plane dimensions, the work on site need it to start 2 months before winter time (often the air temperature is below -4 degrees Celsius) came with many restrictions regarding pouring in situ concrete. The surface concrete slab of 15,000sqm have arisen the cracking issue due to concrete shrinkage after its hardening, so the decision was a widespread use of precast concrete elements. Where the design loads resulted in great reaction forces there were used prestressed concrete beams with cast in situ concrete columns, while for the rest of the elements were precasted and fixed with dry or wet connections.

The 64m span roof was first thought to be designed as several laminated wooden arches. Do to the fact the entrepreneur have neither the possibility nor the technology to build these type of elements, a new solution was brought and that was a space truss with SHS.

2. Building particularities

2.1 Architectural facts

The Multi-functional Sports Hall is organized on five levels: underground level, ground level and three stories (Figures 1 and 2). Underground level is a parking with 447 car capacity. Ground level together with first and second level includes public, officials, media, shopping and administration areas. Ground level also includes the reserved area for sport players, while second level has a conference centre. The third level is for media, special equipment and installations devices. In order to offer multiple use of the playing area, the floor surface can be changed from 30x47m to 38x56m by using extensible/retractable tribune structure on the ground level. In this manner handball, volley, basketball, box and gymnastics championship are possible to take place in the Multi-functional Sports Hall.

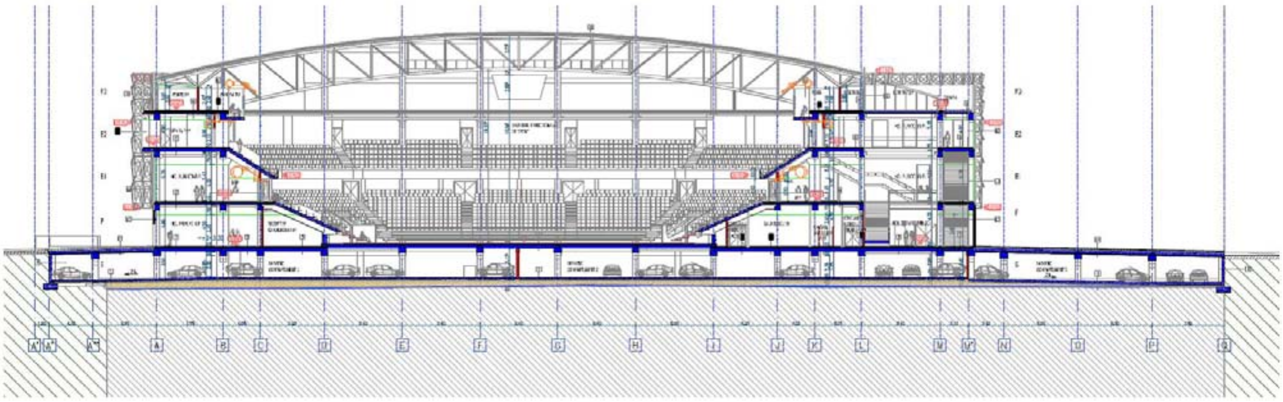

Fig. 1: Transverse section through the building

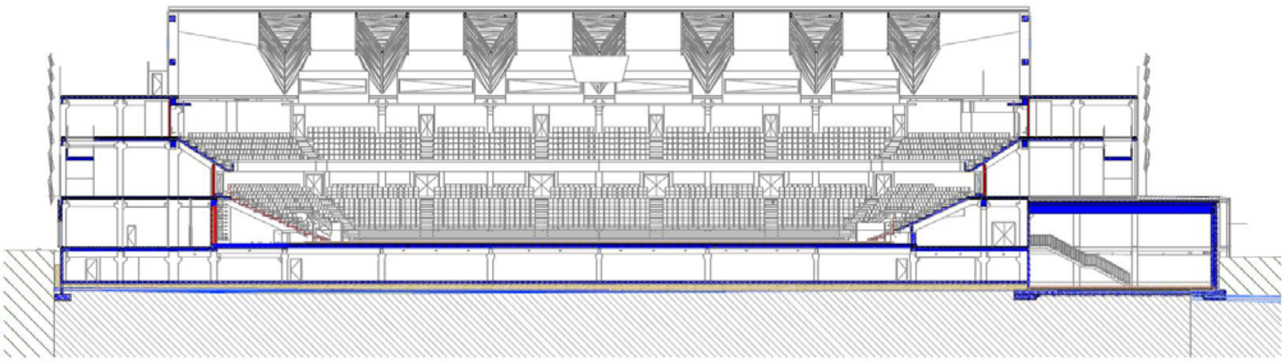

Fig. 2: Longitudinal section through the building

2.2 Design loads considered in calculus and structural analyses

In order to evaluate the structural response, in the design process were considered the following loads (characteristic values), other than the self-weight of the elements:

- Roof loads EN1991-1-1: dead load + technological load q_k = 1,0 kN/sqm;
- Concentrated technological loads (media box) applied in mid span, P_k= 60 kN;
- Temperature variation effects between erection phase (-15 to -20 degrees Celsius in winter) and service phase (+15 to +20 degrees Celsius during lifetime);
- Snow loads on the roof according to EN1991-1-3, $s_{0,k}$=1,5 kN/sqm;
- Wind loads on building envelope according to EN1991-1-4, q_{ref}=0,4 kN/sqm, v_{b0}=27m/s;
- Live loads on slabs EN1991-1-1, q_k = 10,0 kN/sqm – ground floor, q_k = 5,0 kN/sqm – upper floors;
- Technological loads q_k = 2,5 kN/sqm – ground floor, q_k = 3,5 kN/sqm – upper floors;
- Seismic action according to EN1998-1 [1], with peak ground acceleration a_g=0,08g and control period of seismic motion T_c=0,7 seconds;

- Load combination for ultimate limit state (ULS) and serviceability limit state (SLS) according to EN 1990 [2].

2.3 Structural compliance of the concrete structure

Building`s substructure consists mainly in pad foundations, where it was necessary to balance the pad foundations, moment resisting beams were added, due to the large bending moments appeared at the base of the concrete columns supporting the roof; strip foundations were used for the walls situated at the underground level, while for the training room a raft foundation was chosen being also the support of the boarded floor. The foundation soil type is clay marl with a bearing capacity of 750kPa.

The building is a concrete frame structure with a characteristic bay of 8,40x10,60m, for the arena area. On sides the bays are smaller and with varying dimensions. The concrete frames consist in precast concrete columns fixed in foundations using dry connections (Figure 3), except the columns supporting the steel roof which are cast in situ. The last ones, having the cross section 60x120cm, the transportation together with the control and site erecting it would have been difficult to perform. Due to short execution terms, most of the columns were precasted and because of the cold weather during execution time, there were used dry connections provided by PEIKKO.

Fig. 3: Precast concrete column base connection

All frame beams and stair beams are precasted, reinforced concrete or prestressed concrete. The beam-column joints are moment-resisting (floor level ±0.00m) and pinned for the upper levels. For the frames connected directly to the roof`s steel truss, rigid connections have been used for the beams which are in the same plane with the space truss (Figure 4).

(before concreting the joint) *(after concreting in the joint)*

Fig. 4: Beam-column connection

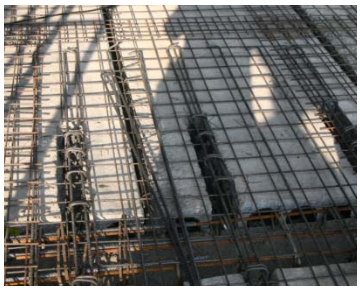

Fig. 5: Topping reinforcing

The slab consists in a precast part (hollow-core units with 200mm and 320mm height or preslabs of 80mm thick where the slab span is reduced) and a topping of 80mm thickness of C25/30 concrete, and S500 reinforcing steel. In order to assure enough stiffness to the vertical loads (the problem of floor vibrations was tested using PulseLabshop software from Bruel and Khaer for the in situ experiments, as well as a commercial FEM software for the numerical simulation. While for the horizontal loads, the cast in situ topping was design to transmit the reactions from the vertical envelope and to the vertical force resisting elements (concrete columns), and also to assure a horizontal rigid diaphragm effect. For a better connection between the partially precast slab and the beams, hollow-core units were partially cut off at the edges and extra reinforced (Figures 5) with triangular skeleton reinforcing; where preslabs were used, only the meshing wires in the topping were enough thanks of the connectors left out of the preslabs surface.

3. Design of the roof`s steel structure

The roof structure consists in 7 trusses with a clear span of 63,90m, and total length of 76,10m, supported by concrete frames and lateral interconnected with the rest of the structure through horizontal and vertical steel bracings. The space steel trusses are mounted using a spacing of 10,50m (Figure 2). Choosing the most appropriate roof structure solution has been driven by a number of factors including the span, roof geometry, load to be supported, economy and aesthetics. The truss section is 3600 mm wide and 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 the continuity of the roof structure over the lateral annexes. The principle was successfully used one of 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 EN1993-1-1 [4], EN1993-1-8 [5] and EN1998-1 [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 was used, which calculation procedure is based on the general method of EN1993-1-1 [4]. For individual member checks, both methods A and B of EN1993-1-1 were 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:

$$f = 165mm \leq f_a = \frac{L}{300} = 213mm \qquad (1)$$

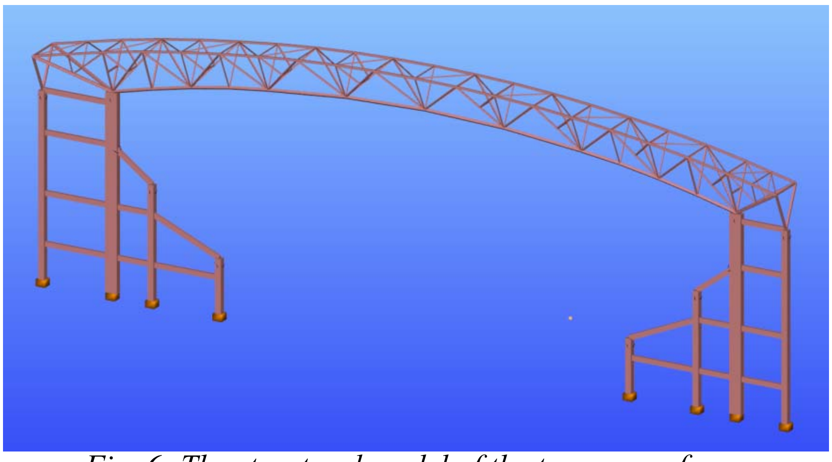

Fig. 6: The structural model of the transverse frame

3.1 Joints between the steel and concrete structure

Providing pinned supports for the roof trusses, positive effects in the internal effort distribution and the highest horizontal reaction over the concrete structures, were 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 7) 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.

Fig. 7: Sliding possibility of the support and a lateral view at support level

3.2 Global stability checks of the space steel truss

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 (corrugated steel sheets were used as part of the envelope, 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 8), corresponding to the stability loss of first compressed diagonals. Improving the model, considering the effects of additional braces of the compressed diagonals (Figure 9), the critical load multiplication factor increased at $\alpha_{cr,op}$=6,81. Alternative 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

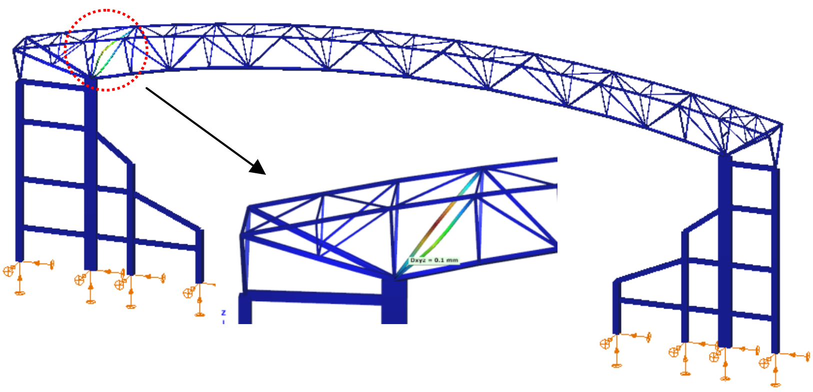

Fig. 8: Compressed diagonal buckling in the space truss ($\alpha_{cr,op}$=3,63)

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 ratio of utilization for the members, following the general method of EN 1993-1-1 resulted in less than 80% his way, while considering the simply supported configuration of the roof truss structure the level of 100% is achieved.

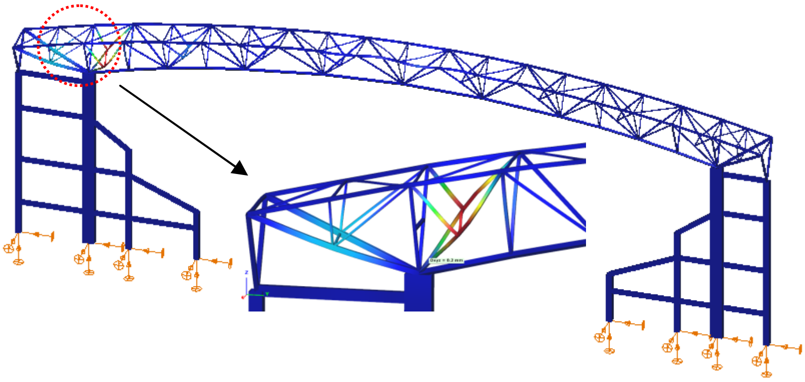

Fig. 9: Improved space truss ($\alpha_{cr,op}$=6,81)

3.3 Joints checks using FEM of the space steel truss

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 10), a specific finite element model was developed (Figure 11). Nonlinear elastic-plastic analysis considering geometric nonlinearities (GMNA and GMNIA) has been applied.

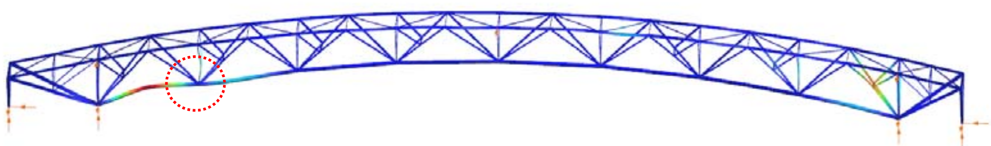

Fig. 10: Analyzed truss joint

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 13). The model's materials were defined as elastic – linear plastic (E=210000 N/mm^2, υ = 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.

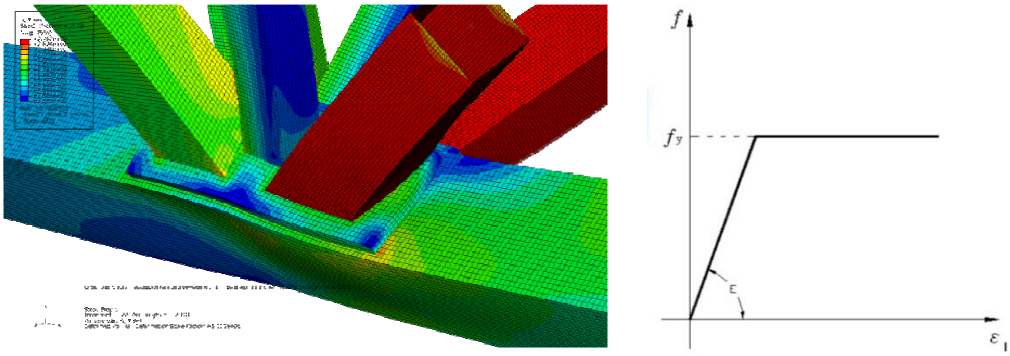

Fig. 11: FEM of the selected joint and material model

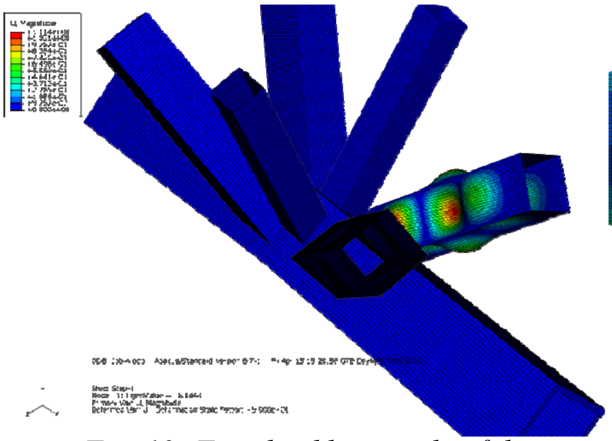

Fig. 12: First buckling mode of the joint ($\alpha_{cr,op}$=8,18)

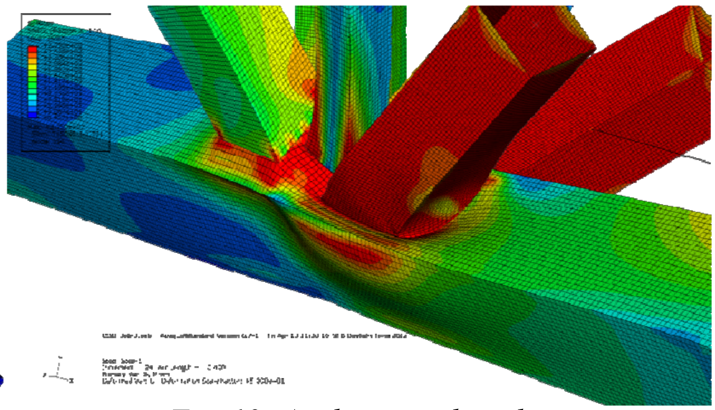

Fig. 13: Analysis result without strengthening plate

4. Structure`s erection stages

The structure erection stages are:

- Phase I – cast in situ columns together with the precast elements (columns, beams, slabs) are erected till the roof level, while the area beneath the playing field remains at the foundation stage in order to be able to accomplish later the steel roof structure from inside and from outside the building perimeter;
- Phase II – erecting work of the space steel trusses and mounting the bracings;
- Phase III – mounting the precast elements(several short columns, prestressed beams and concrete hollow-core units) underneath the playing field as well as around the site, till ±0,00m level is reached;
- Phase IV – installation of the roof and façade envelope.

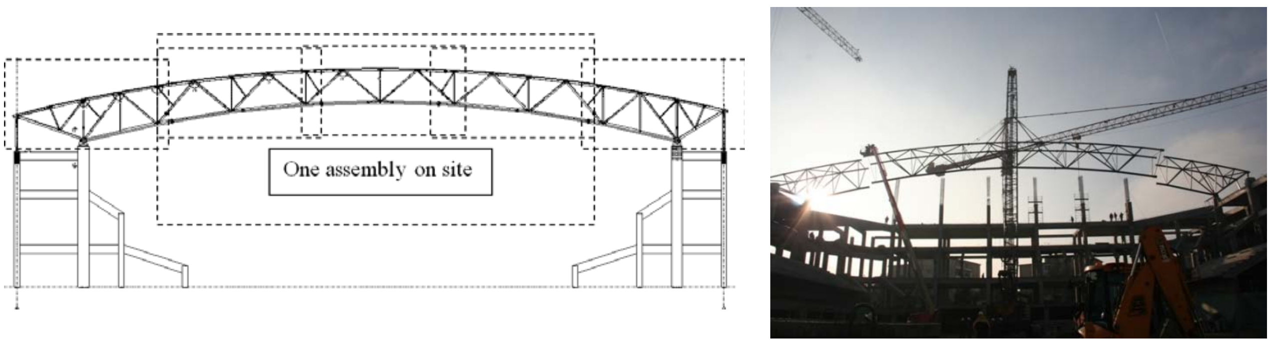

Fig. 14: Truss splicing for transport an erection on site

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 (76,10 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.

5. Conclusions

This paper describes the key aspects of matching a precast concrete structure with a large span space truss and searching for the optimal solution in this issue, from the safety point of view but also from the economical one too. Covering a large span area always arise problems even if we deal with a building of a small height (like a greenhouse for instance). But when the roof, in this case a space steel truss, is placed on top of a concrete structure several new issues need to be taken care off, like: the compatibility of the two types of structures (steel and concrete) that need to work together (displacements due to gravitational loads, horizontal loads, temperature differences), the construction stages (starting with mounting the first parts of the truss, continuing with the final assembling of the first truss and next the completion of the whole roof, and last but not least the final touching – in this case turning the simple support into a perfect pinned hinge) and other problems that may appear like the connection between SHS elements. Several local and global stability design checks for a large free span roof truss structure were made. 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.

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