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ADVANCED NONLINEAR INVESTIGATIONS OF A 50 M SPAN FRAME CASE STUDY: THE STEEL STRUCTURE OF THE ICE RINK, CITY OF TARGU-MURES, ROMANIA

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***Abstract**. The article describes the applied technological solutions to transform an existing ice rink into an indoor arena in the city of Târgu Mureş, Romania. The new indoor arena will have a capacity of 1800 fixed seats. Using a large free span (50 m) structure will overcome the in situ technological constraints due to the position of the existing building. There is limited access due to the fact that the ice rink is situated between two buildings and the river Mureş which flows alongside the third side. Also, the existing refrigeration system makes access impossible within the ice pad structure area. Taking into account the above mentioned restrictions, the article describes the applied structural solutions which will make the structural steel work erection possible. The structural solution using steel will ensure fast and easy erection of the structural steel framework without causing damage to any of the existing buildings and installations. The paper summarizes the results of the numerical study performed by the authors on the frame structure. The frames were designed to withstand horizontal and vertical loads and also to satisfy the ULS and SLS criteria. The frames have fixed base connections, tapered columns, hunched and king-post truss rafters and a pitch roof angle of* 3^0.

1 INTRODUCTION

1.1 About the scope of works

To find a technological solution to transform an existing ice rink into an indoor arena in the city of Târgu Mureş with a capacity of 1800 fixed seats imposed to use a large free span (50 meter) steel structure. With the condition of a clear height of 9,00 m over the ice pad area, the geometrical dimensions of the proposed building resulted 58,60x 67,00 x13.00 m (width x length x height) . The building on the ground floor consists of the ice pad area - 1800 m^2 (60 x 30 m) and the necessary annexes (public area, offices and dressing rooms etc.) of 2200 m^2. The scope of works included the following main requirements:

- -To cover the existing ice rink in order to extend the usage lifetime;
- -To have a capacity of 1800 fixed seats;
- -To ensure 90 min fire rating of the steel structure;
- -To ensure the specific internal micro climate.

1.2 Constrains

Due to the destination and the particular position of the building - limited access due to the fact that the ice rink is situated between two buildings and the river Mureş flows alongside the third side, there were the following constrains:

- -Access for erection only from one side, without the access on ice pad structure due to the existing refrigeration system;
- -To keep the existing buildings;
- -To control the designed assemblies self weight, in order to facilitate the erection.
- -Also the pressure of a short deadline acted as constraint. The site conditions and the proposed architecture should be seen in Figure 1.

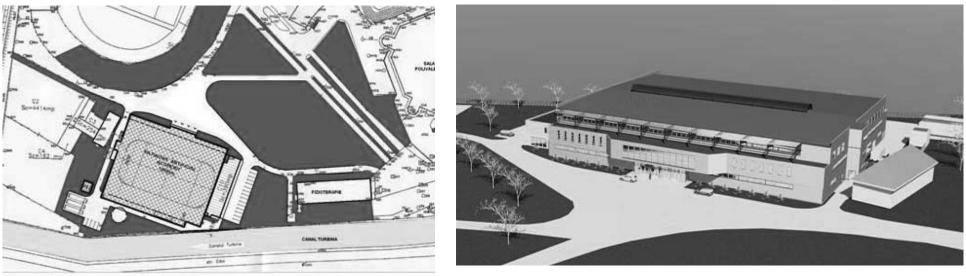


Figure 1. The site conditions and the architecture of the building (view from access side)

2 DETAILED DESCRIPTION OF THE BUILDING

2.1 Steel structure

The primary load-bearing structure of the building uses a simple steel portal frame shape based on a 6.00 m grid (Figure 2), combined with a king post truss rafter. The clear span of the frame is 50 m, with additional 4 m extension on both sides. The frames have fixed base connections, tapered columns, hunched and king-post truss rafters and a pitch roof angle of 3^0. The rafter was extended over the two lateral extensions and fixed at the top level of the columns from the extremities. In that way instead of having a simple frame, we have transformed the rafter in a continuous beam, increasing both its strength and stiffness. The supporting structure of the tribune is fixed to the frame column in the transverse plane, increasing in that way the lateral stiffness of the whole transverse frame. In order to prevent lateral-torsional buckling of the rafter, its lower flange was braced laterally to the roof purlins (see Figure 3 a&b). Supplementary lateral restraints were provided by means of longitudinal beams, stiffened together by the roof bracings. At the mid span, king post truss was laterally restrained in order to prevent its lateral displacement in case of horizontal actions (e.g. seismic action-see Figure 3a). All the assemblies (excluding longitudinal beam, bracings) are made from welded steel sections. A structural steel with S355 steel grade (f_y=355 N/mm^2) have been used.

For the first and second floor slab in situ reinforced concrete solution was applied. For the composite action of steel and concrete, mechanically fixed shear studs have been used on floor beams. Precast concrete elements were designed for the tribune. A central skylight cut out of the roof to bring daylight down to the ice rink.

The 90 min fire resistance of the structural steel columns and 60 min for rafters and floor beams is assured by intumescent coating of the steelwork.

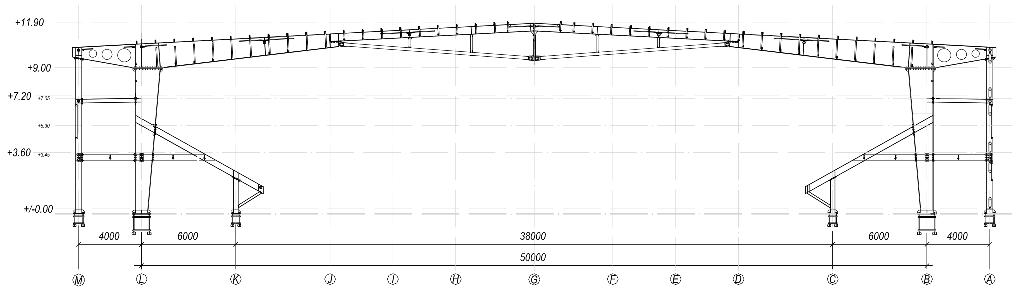


Figure 2. Characteristic section of the structure

a) lateral restraining of the king post truss b) rafter-to-column detail

Figure 3. Connections detail

2.2 The building envelope

The insulated building envelope makes it possible to control the indoor climate regardless of the outdoor climate. In the case of this type of buildings, air tightness is a more important feature of the envelope than thermal insulation. Large glazing of the facade has been avoided due to energy costs by operating the facility. Windows are placed mainly on facility area, because the most optimized ice rink can be done by a fully closed casing.

The wall cladding is made of 120 mm thick horizontal sandwich panels. On the roof 200 mm thick rock wool insulation is laid down on the supporting trapezoidal steel profiles, waterproofing is assured by a protective membrane.

The supporting structure of the facade is a steel framework of rectangular hollow sections.

3 STRUCTURAL DESIGN OF THE BUILDING

3.1 Loading of the main 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
- -Live loads on floors (EN1991-1-1) u_k= 5.0 kN / m^2
- -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
- -Fire loads of 120 MJ/m^2
- -Seismic action according to P100-2006 (EN1998-1), with peak ground acceleration a_g=0.12g 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 (EN 1990).

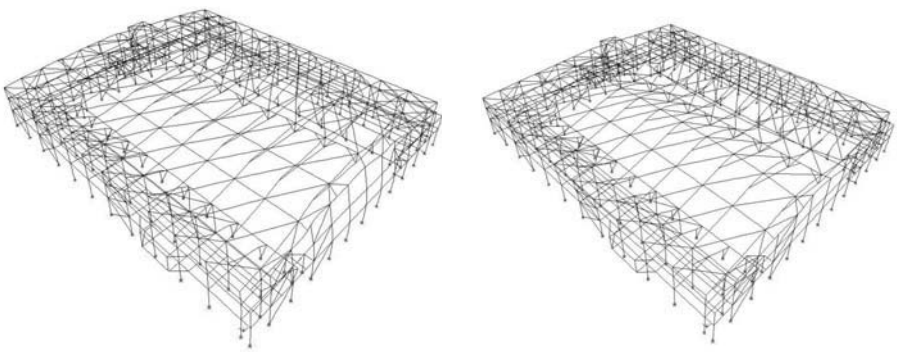
3.2 Design of the main structure-linear elastic analysis (LEA)

The design of the steel structure was performed following the Romanian code STAS 10108/0-78 [6]. 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 [3] were also used.

In the case of large spanned structures, the vertical deflection under gravitational loads represents one of the major constraints in the design process. In order to keep under control the deformations of the frames, fixed base connections, tapered columns and hunched king-post truss rafter solution were chosen [6]. The rafters were extended on both sides over the annexes, increasing both the vertical and horizontal stiffness of the frame. A suitable horizontal and vertical bracing system were provided in order to control structural flexibility, eigen values and deflections of the main structure. Fly braces were provided at the inner flange of the rafter in order to improve the flexural-torsional buckling resistance of these elements.

Having class 3 section of the structural elements, linear elastic structural analysis was performed, using a seismic behavior factor of q=1 according to P100-2006 [3]. Even with q=1, the combinations of actions for seismic design situations were not the dominant load combinations. The design checks of the structural elements for ULS include persistent or transient design situations (fundamental combinations) where snow loads play the key role.

For SLS design checks of the structural elements fundamental and exceptional load combinations were used. Performing a dynamic 3D analysis of the structure, with the structural masses concentrated on joints, first longitudinal eigen period of T_{long}=0.588 sec and first transversal eigen period of T_{transv}=0.448 sec were obtained (see Figure 4).



First longitudinal vibration mode T=0.588 sec First transversal vibration mode T=0.448 sec

Figure 4. Eigen vibration modes and periods

The maximum transversal and longitudinal sway displacement for SLS check under seismic loads according to P100-2006 are:

$$d_{r,x}^{SLS} = 0.014 \le \frac{d_{r,a}^{SLS}}{\nu \cdot q} = \frac{0,005 \cdot h}{0.4 \cdot 1.0} = 112.5mm \tag{1}$$

$$d_{r,y}^{SLS} = 0.047 \le \frac{d_{r,a}^{SLS}}{\nu \cdot q} = \frac{0,005 \cdot h}{0.4 \cdot 1.0} = 137.5mm \tag{2}$$

The maximum vertical deflection of the rafter for SLS check under snow load is:

$$f = 161.4mm \le f_a = \frac{L}{300} = 166.7mm \tag{3}$$

In order to have an overview about the real behavior of the structure, a finite element linear elastic analysis (FEM) of the transverse frame has been performed with Ansys computer program. The elements of the frame were modeled using shell finite elements (Shell 43- see Figure 5). The forces on the rafter were applied as point loads (points where purlins are fixed on the frame). The connections between structural elements rafter-to-column, beam-to-column, rafter-to-rafter, column base connections were considered fully rigid. The results of the detailed linear-elastic analysis (LEA) confirmed the previously evaluated ULS and SLS results. The recorded vertical displacement in case of FEM linear elastic analysis was 152 mm (instead of 203 mm –linear elastic analysis). It might be emphasized that the resulted structure is more rigid in case of FEM analysis, explained by the shift of the neutral axis along the elements with variable cross sections (i.e. tapered column, and hunched rafter). Figure 6 shows the stress distribution along the transverse frame, where we can observe the maximum stress concentration around the joint of the king post rafter and the hunched frame rafter.

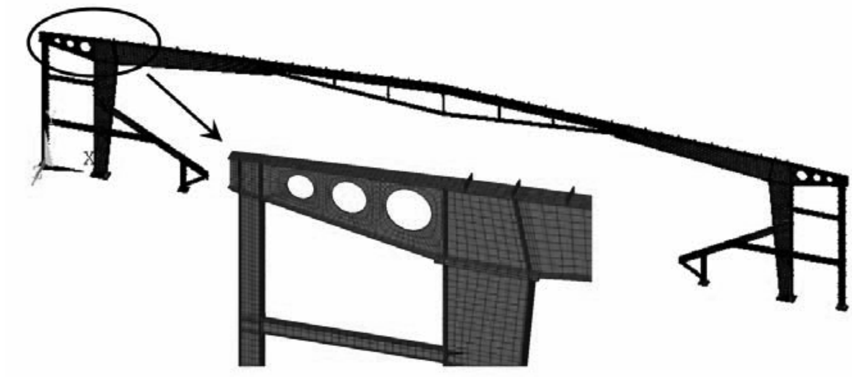


Figure 5. FEM model of the transverse frame

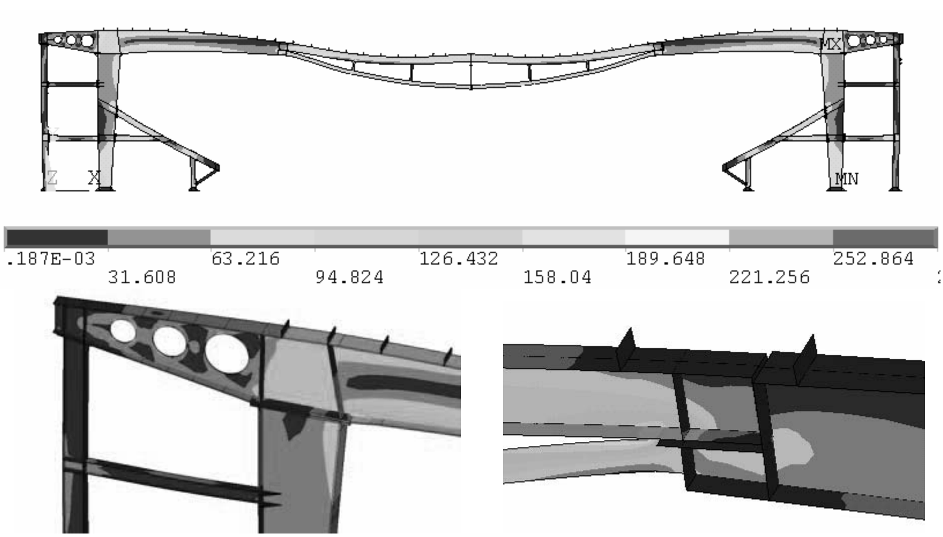
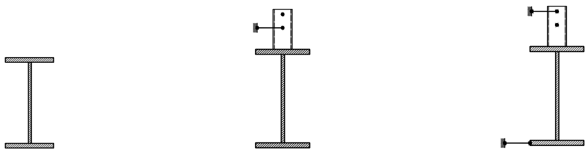


Figure 6. Stress distribution along the transverse frame under gravity load combinations-linear elastic analysis

The resulted maximum stress does not exceed 252 N/mm^2. There were many concerns about the stress distribution in the connection of the king post truss with the rafter. As it can be observed from Figure 6 the distribution of the stresses does not exceed the maximum allowable yield limit.

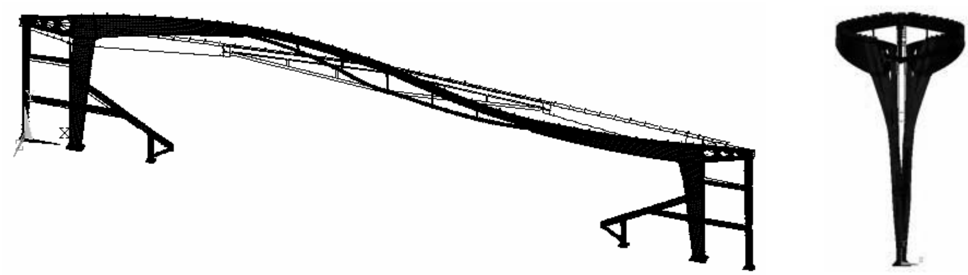
3.3 Design of the main structure- non linear elastic-plastic analysis

In Figure 8, it is illustrated the way in which the initial bow (out of plane) imperfection is considered in the nonlinear-elastic analysis (GMNIA). Three types of lateral restraints of the rafter were considered separately in the analyses (see Figure 7 [7],[8]). Types 2 simulate the purlin/sheeting effect, when the purlin can be connected with one or two bolts, respectively. Type 3 is the same with type 2 with an additional fly brace. Type 1, the reference case, actually means no lateral restrains introduced by purlins and side rails.



(a) restraint type 1 (b) restraint type 2 (c) restraint type 3

Figure 7. Types of lateral restraints considered in the analysis



a) global view b) lateral view

Figure 8. Manufacturing imperfections considered in non-linear elastic-plastic analysis [6]

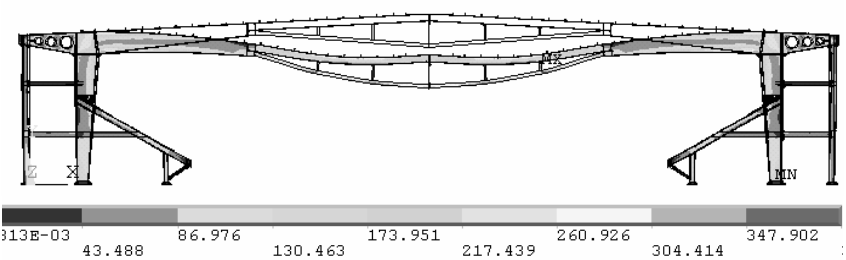


Figure 9. Stress distribution along the transverse frame under gravity load combinations-non linear elastic-plastic analysis

To simplify the computational model, in the analysis the lateral restraints were considered axially rigid. The values of the applied imperfection is 167 mm (50 m span frame) for initial bow imperfection (er), l/150 corresponding to curve c, for plastic analysis, according to clause 5.3.2 (3)-a) of EN1993-1.1 [1]. The material behavior was introduced by a bilinear elastic-perfectly plastic model, with a yielding limit of 355 N/mm^2. In Figure 10 are illustrated the capacity curves for different type of analysis. As it was expected the lateral restrains of the rafter played an important role in the total capacity of the main frame. Also it must be emphasized that there is more than 25% structural capacity reserve.

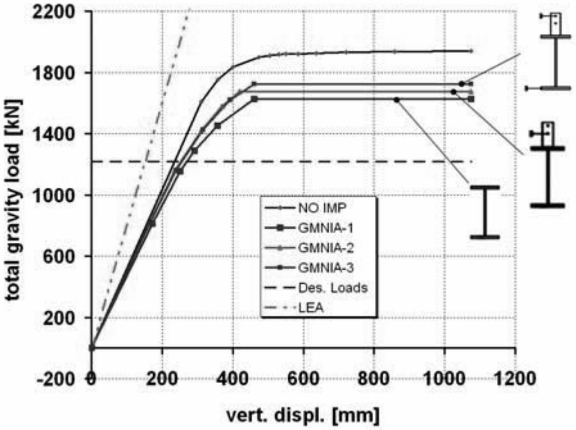


Figure 10. Capacity curves for different type of analyses

4 CONCLUSION

The paper illustrates the successful application of the steel structure for a large span using a simple portal frame shape, combined with a king post truss rafter. A wide range of design parameters are briefly summarized. The paper emphasizes the whole design process, assisted by FE analysis - in order to perform supplementary stability checks of the framed structure. Due to the unusual shape of the transverse frame, there were many concerns about its real behavior under gravitational load, the most important ones in this particular case. For this purpose a linear elastic analysis (LEA) followed by a nonlinear elastic-plastic analysis (GMNIA) were performed in order to determine the real behavior of the frames. From structural point of view a good agreement between 3D structural analysis and LEA-FEM has been found. GMNIA analysis confirm at least 25% overstrenght of the structure by applying the chosen structural solution and lateral restraints of the main rafter.

Even with behavior factor q=1, the combinations of actions for earthquake design situations were not the dominant load combinations. In the design checks of the structural elements, gravity loads played the key role.

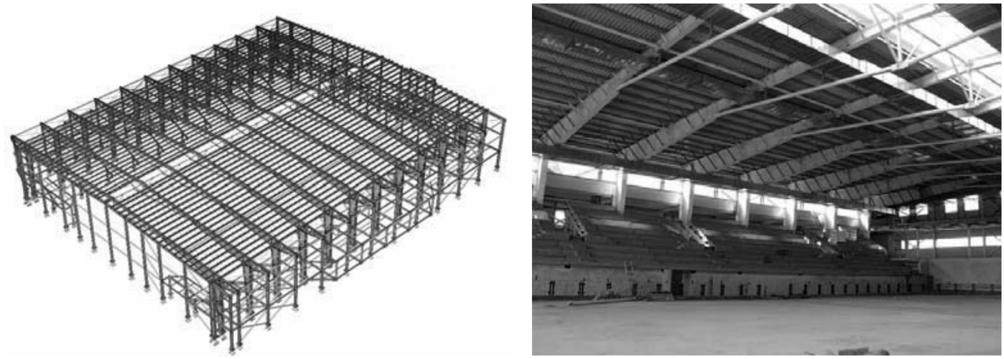


Figure 11. The whole structural model and the actual stage of the building

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