УДК 624.014

Roof over PGE Arena in Gdansk. Review of structure and monitoring system

Żółtowski K., Romaszkiewicz T., M.Sc. Eng.

Technical University of Gdansk, Poland

Анотація. У статті викладені загальні конструктивні рішення сталевої конструкції, спроектованої для нового футбольного стадіону для Євро-2012 в Гданську. Основна конструкція зведена на існуючому зміненому фундаменті. Фасад і покриття складаються з 82 сталевих трубчастих елементів, що становить грандіозну споруду з полікарбонатним покриттям. Описані труднощі, що виникли під час її складання та монтажу, а також розроблена і впроваджена спеціальна процедура з моніторингу.

Аннотация. В данной статье изложены общие конструктивные решения стальной конструкции, спроектированной для нового футбольного стадиона для Евро-2012 в Гданске. Основная конструкция возведена на существующем измененном фундаменте. Фасад и покрытие состоят из 82 стальных трубчатых элементов, что представляет собой грандиозное сооружение с поликарбонатным покрытием. Изложены трудности, возникшие при его сборке и монтаже, а также разработанная и внедренная специальная процедура по мониторингу.

Abstract. This paper presents general structural assumptions of the steel construction designed for a new football stadium to be built for UEFA Championship 2012 in the city of Gdańsk. The superstructure is founded directly on the modified abutment. The facade and the roof are collected from 82 steel girders made of tube profiles. All this creates a grandiose building covered by polycarbonate cladding. There are described major difficulties which arised during fabrication and erection and the special monitoring system which was designed and implemented.

Key words: Stadium, UEFA EURO 2012, steel roof, monitoring.

Introduction. The stadium which was built for UEFA EURO 2012 in Gdansk is the biggest sport arena in the city. Its architecture refers symbolically to a piece of amber over seashore. As a result of a building process a complex structure composed of steel and concrete was built. According to UEFA standards, the stadium was planned to hold nearly 41000 people. The whole infrastructure of the arena is placed on a specially prepared area of 43 650 m^2.

The most spectacular part of the stadium is a steel roof and facade structure. Its characteristic shape and transparent cladding in tones of yellow shows the architect's vision mentioned before – a lump of amber.

The design team is listed below:

- Architect – RKW Rhode Kellermann, Wawrowsky GmbH from Düsseldorf.
- Structures – Bollinger&Grohmann – steel structure conception works in preliminary stages.
- Structures – APK Wojdak Consulting Engineers – concrete structures final design.
- Structures – KBP Żółtowski Consulting Engineers – steel structures conception works and final design.
- Structures – Eilers & Vogel – secondary concrete structures and small architecture.
- Structures – prof. Tadeusz Godycki-Ćwirko – consultation and verification.
- Structures – Gdansk University of Technology – laboratory tests and expert works.

Fig. 1. PGE Arena. Photo taken 22.8.2010 (www.bieg2012.pl) and after the opening (www.euro.gdansk.pl)

Steel roof construction. The main structure of the roof is composed of 82 steel, sickle shape space truss girders connected by a circumferential structure [1, 2, 3]. Typical girder is made of tube profiles and stands on a 7 m high concrete ring. The steel roof (Fig. 2) is statically and structurally independent of the concrete part. The height of the structure is about 38 meters, measured from bearing to the roofing surface. The length of the cantilever over the tribune is about 48 meters, measured from bearing to the edge of the roof (centre ring).

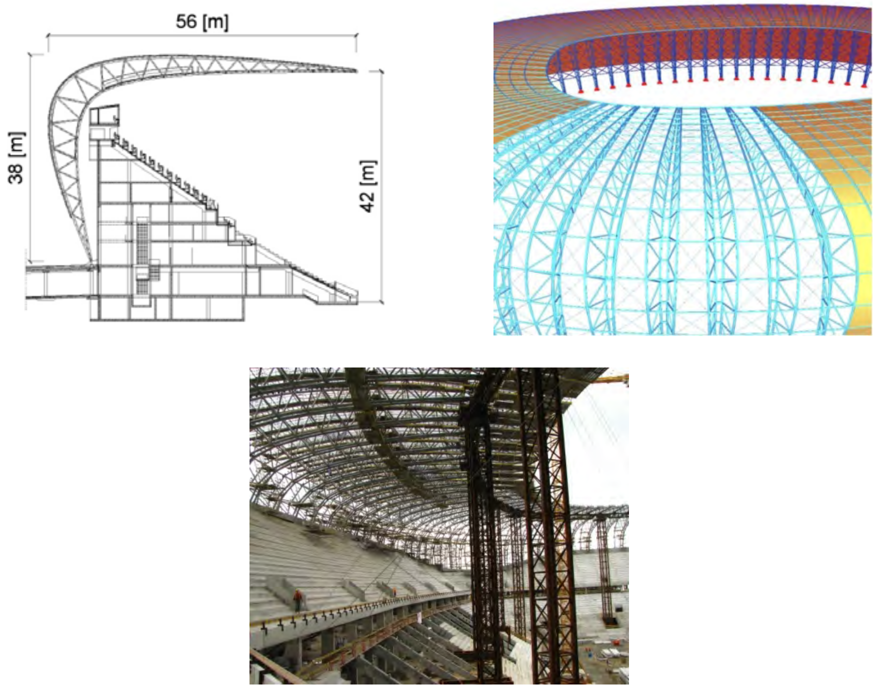


Fig. 2. Roof structure. Static model (SOFiSTiK) and photo made 18.08.2010

Main girders. The main radial girders (Fig. 3) are placed around the stadium at the distance of circa 8 meters. The framework of a single girder is made of welded steel tube profiles. The top and bottom chords are tubes of 355,6 mm in diameter and varying thickness. They are connected by horizontal tubes: top – ϕ 219.1 mm, bottom – ϕ 355.6 mm. Diagonal members of the truss are tubes of 219.1 mm in diameter and thickness of 8 mm.

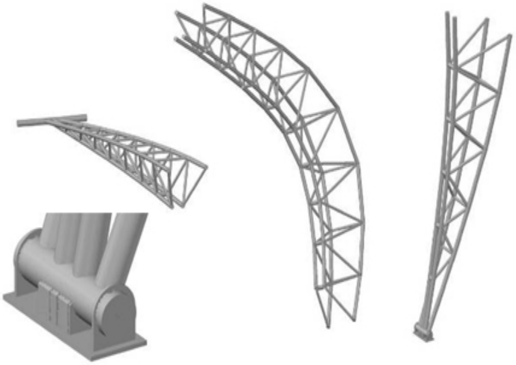


Fig. 3. Geometry of three main parts of a single girder

The girders are connected to each other by several horizontal rings made of circumferential tubes. Additional X bracings made of tension rods in the system X are implemented in every field between each girder. Thanks to the stiffening system, the roof works as an quasi-dome with the hole in the centre. The total weight of the roof is circa 71500 kN [2].

Stiffening. The stiffening system consists of horizontal rings and X bracings. The horizontal rings are placed in every outer structural node of the girders above the height of 6 meters, measured from the bearing. The rings are made of tube profiles ϕ 219.1 mm except for the interior board, which consists of the tubes of 508 mm in diameter and thickness of 8 mm. The X bracing works as only tensioned. 52 mm diameter rods are equipped with fork connectors and locking nuts. The rods and horizontal ring elements are connected with girders by gusset plates (fig. 4).

In the middle of the girder, over tribunes there is a cross truss stiffening which connects the top and the bottom chords to each other. It is situated near a technical platform suspended to the construction.

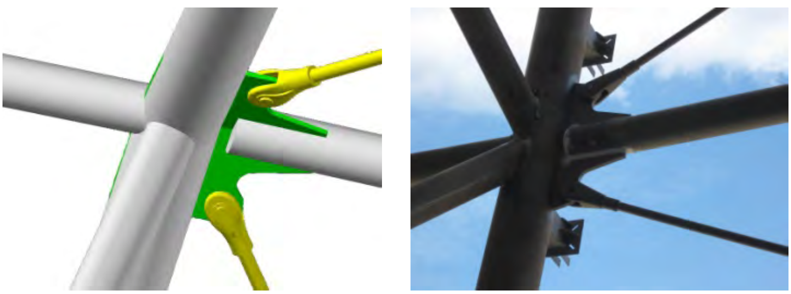


Fig. 4. Connection of stiffening to girder – design and realization

Roof cladding and peripherals. The cladding of the roof, designed as a membrane made of polycarbonate, is supported by a special aluminium sub construction. (Fig. 5) The roof is equipped with inspection platforms, rainwater drainage system, snow fences, lighting installation, four media screens, sound system and other equipment.

Fig. 5. Aluminium sub construction and laboratory test of aluminium roof profile

Statical analysis of the roof. For the purpose of the project laboratory tests were made in an aerodynamic tunnel [4]. The main task was to define wind action to the real shape of the construction. As a result, pressure distribution at eight different wind directions was obtained. Other loads were defined according to the Polish code. The whole numerical model was made in SOFiSTiK FEM environment.

It consists of: 13120 – nodes, 24518 - beam elements, 2624 - cable elements, 5904 - membrane elements.

The membrane with a low stiffness is described on a grid of beam elements as cladding. It does not cooperate with the main structure, it is only used to transfer area loads to the beam structure.

In computation a superposition rule was used to define boundary forces in the elements. The extreme configuration of loads reached after linear steps was taken as a global load to geometrically nonlinear analysis. The main purpose of such an approach was to assess the influence of switch off effect in tension rods bracings to the main structure (Fig. 6).

Due to the uniqueness of the structure there were made several detailed numerical models of key elements of the structure (Fig. 7).

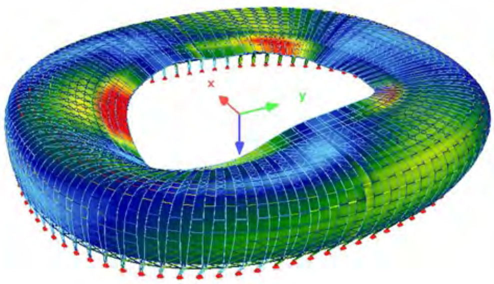


Fig. 6. Roof deformation caused by a north wind

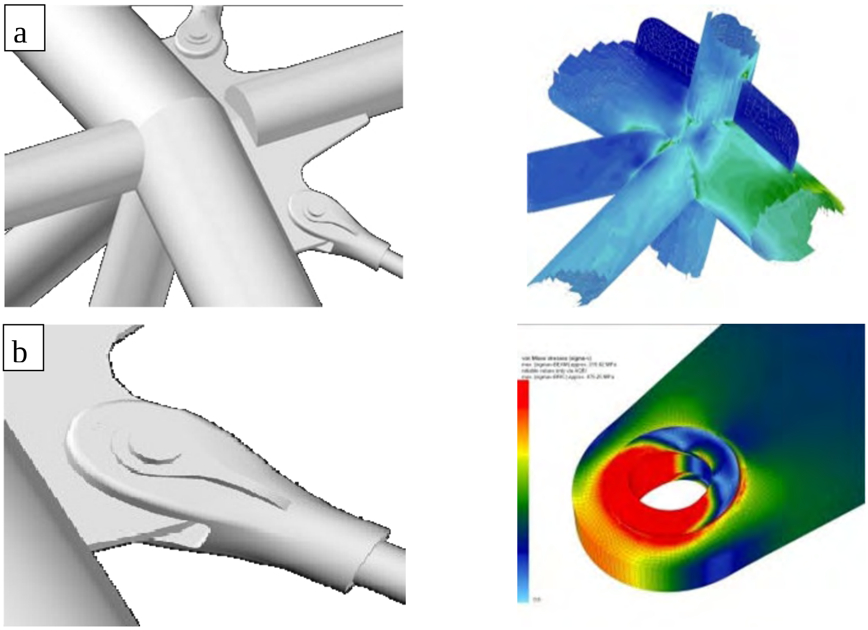


Fig. 7. CAD visualization, shell nad bricks FEM model:
a – typical node; b – connection of bracing

Roof assembly. The roof elements were prefabricated as flat members in a workshop and assembled on site to space parts. The erection of the roof was divided into 6 phases:

1. Installation of temporary supports.
2. Installation of a facade girder part.
3. Installation of a roof girder part.
4. Installation of circumference tubes and bracings.
5. Exemption from temporary support.
6. Installation of sheathing and peripherals.

The whole process of mounting the roof was analysed by numerical FEM models (Fig. 8). The process was monitored by surveyors and four girders which had active strain gages. These results were compared to those obtained from numerical analysis (Fig. 9).

An extreme deflection was measured on the girder 79 with value of 440 mm. The value obtained from numerical analysis was 417 mm. The biggest difference of 12,4 % was on the girder 50. An average difference was calculated as 4,92 %.

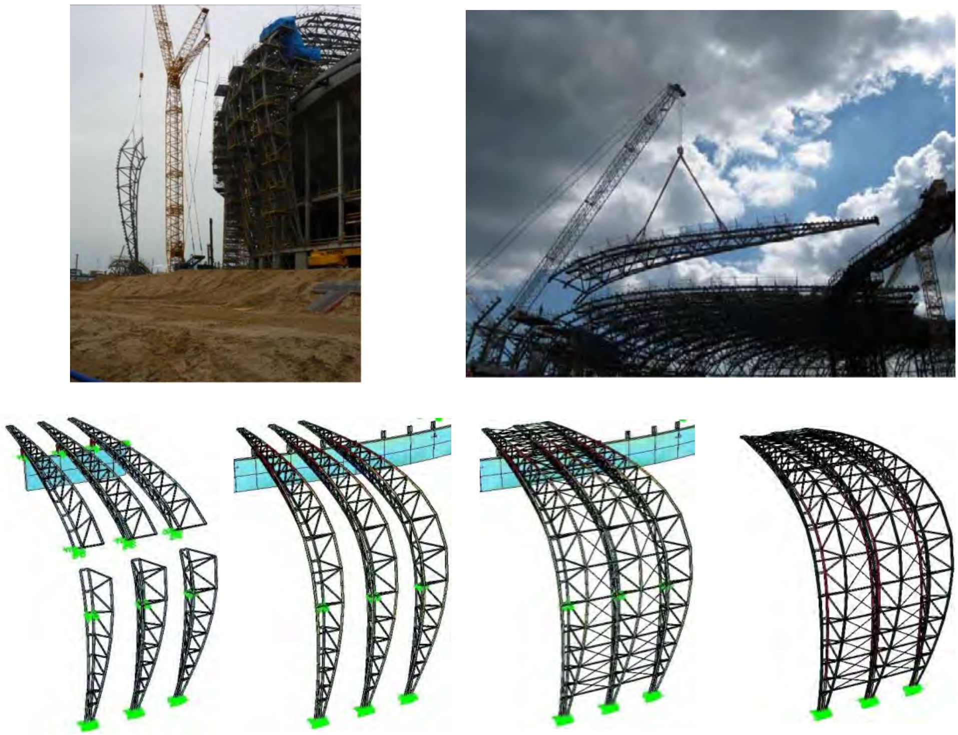
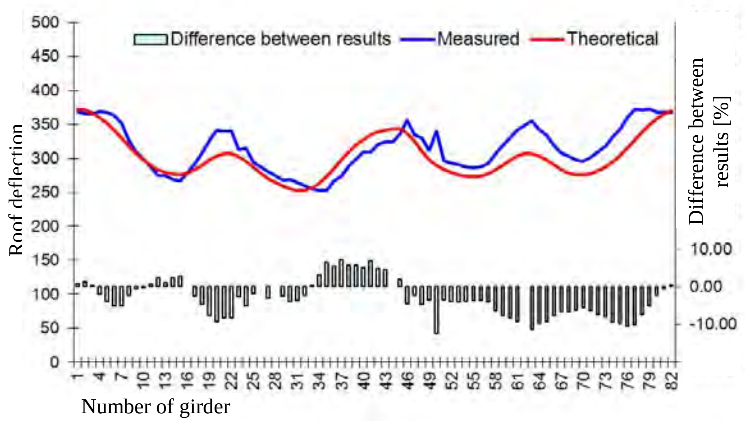


Fig. 8. Installation of facade and roof parts of girder – photos and phases in numerical environment



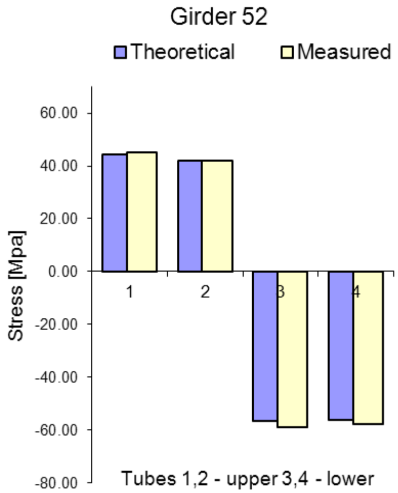


Fig. 9. Comparison of numerical and measured results

Selected problems of designing and erecting the roof. From the very beginning there was a discussion with the architect regarding several aesthetic aspects negative from the engineering point of view. The constructor convinced the architect to make the tube members straight between the nodes of the truss girders, but the whole geometry of the girder had to stay as in the architect's vision. This created big difficulties in detail design and fabrication of the steel sections. Despite all available quality procedures there were mistakes, which had to be solved:

1. shape distortion of the horizontal tube in the supporting node as a result of welding (detail shown on Fig. 14),
2. geometric deviations in gusset plates,

3. accuracy problems in the lengths of the tube members accrued while fitting the facade and the roof part of the girder,
4. problems with quality of filled concrete in the horizontal tube (supporting node).

The shape distortion of the horizontal tubes in the supporting nodes forced change in tolerance of the cast bearing parts. The already manufactured elements had to be ground. More significant were the geometric deviations in the position of the gusset plates in the few first girders (Fig. 10). They caused imperfections in the connection of the circumference tubes and bracings to the node, which exceeded the allowed values. The contractor in consultation with the designer and investor managed to repair the positioning of the plates basing on prof. Edmund Tasak's method [5]. The method involved: partial incision of the plates, thermal straightening and welding. An important element of the process was a nonlinear analysis of a detail with a plate deformed by straightening [6]. For this purpose several shell MES models, including various degree of geometric imperfection, were made. One of them is shown in figure 11.

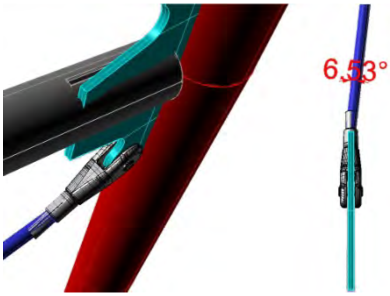


Fig. 10. Visualisation of an improper placing of plates

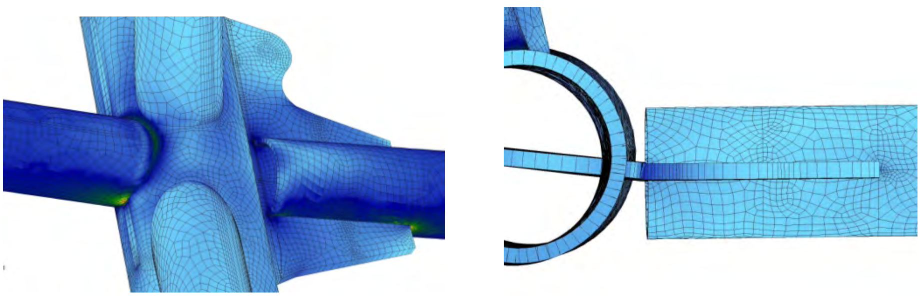


Fig. 11. Visualisation of the analysed node with a deformed plate

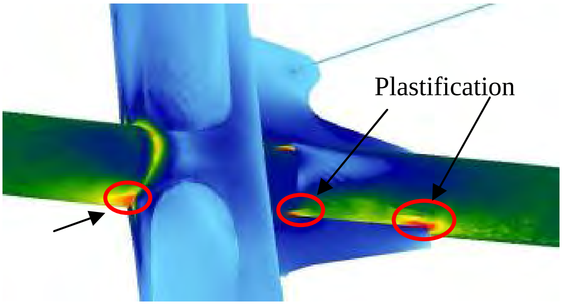


Fig. 12. Visualisation of the node with marked places where steel exceeds the yield strength

The numerical analysis revealed strength reserves of steel in the deformed node, which allowed to accept the procedure of repairing. After revealing the imperfection in the few first girders, it was eliminated in the next ones.

The next problem appeared while fitting the connecting roof part of the girder to the façade part. It was hard for the contractor to connect the two parts precisely in the four points keeping the whole girder in the correct position. In the few first girders, the gap between the two parts of one tube was too big (Fig. 13). Tightening was very easy by regulation of the free end of the girder but in this case it was absolutely forbidden, because of the additional internal stress in the range of 200 MPa. In that case the contractor had to cut off about 700 mm of the tube and insert a new fitted piece. After that the positioning procedures were upgraded and the problem was solved.

Fig. 13. The connection process of the girder roof part with the facade part

An expected problem occurred with the support node filled with concrete (Fig. 14). The support detail was designed as a hinge in the erection stages. The final shape was created by a steel designer to fulfil aesthetic aspects. The main element of the seating part designed as welded was finally manufactured as cast steel (decision of the contractor). The horizontal tube was filled with concrete to increase its bearing capacity. The inspection of this concrete showed that its parameters do not correspond to the design assumptions (B-28 instead of B-50). The replacement of the concrete filling was difficult due to the already erected structure and timetable. Therefore, there was decided to analyze the carrying capacitate of the support node with actual quality of concrete. For this purpose, a laboratory test was done on the model in scale 1:1 at Gdansk University of Technology. The support piece was filled with concrete ~B28 (Fig. 16). The laboratory test was designed on the base of numeric FEM model shown in figure 15.

Fig. 14. Support of the girder and the place where the samples were taken

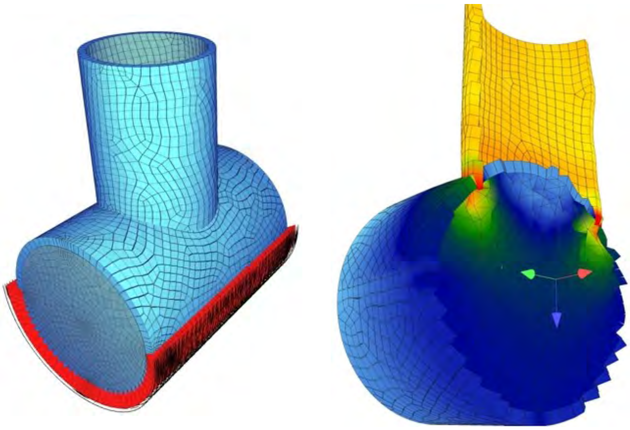


Fig. 15. FEA model of the support and stress concentration

Fig. 16. Laboratory tests of a real scale model

The maximum normal force in the vertical tube of the support node was estimated as 2500 kN. During the tests a load of ~5000 kN was reached (the limit of the hydraulic press) without a serious symptom of overloading. This could happen thanks to the change of the seating part of the bearing form, welded to the cast (contact area increased radically). On the base of the laboratory test the existing concrete filling was accepted.

Safety of the roof structure. The roof and the facade of approximately 45000 m^2 is open to environmental influences and as a big and unique structure it naturally generates several questions about safety. Our European experience of the last decades shows that the most serious (except for an earthquake) is snow load. PGE Arena is a huge surface open to snow load (approximately 27000 m^2). Huge surface and very sensitive cladding (polycarbonate) create big problems in the case of snow removal from the roof, which is then distorted (in Poland it often happens with flat roofs, even they are built according to the actual snow code – 1.2 kN/m^2 for PGE Arena).

To avoid panic and to help to administrate the stadium in decision regarding snow load a monitoring system of the structure was designed and implemented [7]. The system consists of 256 strain gages (16 girders x 4 tubes x 4 gages), 16 displacement sensors, 16 accelerometers, 16 wind sensors and 4 cameras. The gages and the displacement sensor are placed on the girder, as shown in figure 17.

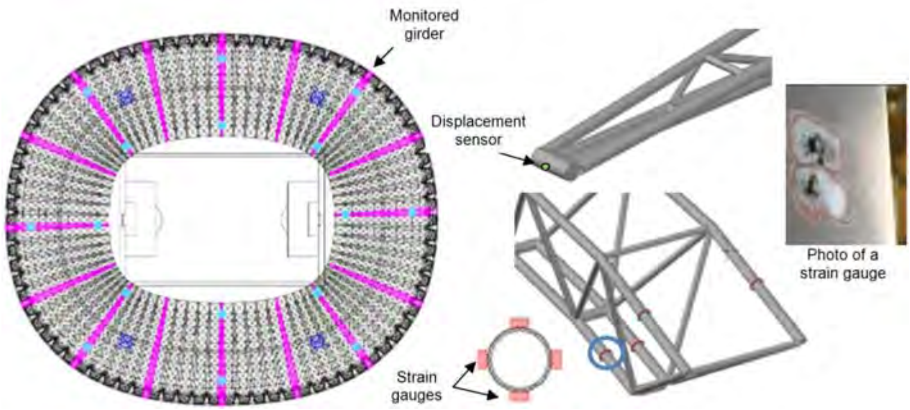


Fig. 17. Monitoring system – localization of sensors

Monitoring is used to diagnose the structure in real time and react to its failures. The installed sensors enable to assess stresses in the tubes and the interior ring deflection. Expert system powered by genetic processor can estimates snow load on the roof, on the base of finite monitored data. For the snow load prediction the roof is divided into 16 fields. Each field has monitored girder in the middle (Fig. 18). Single field consists of 11×5 subfields, where the snow is being considered. The whole genetic process is divided into small iteration steps, such an approach involves interaction between neighbouring girders. The effectiveness of the system was verified on several numerical tests (Fig. 19, 20). In the last step the numeric model of the structure is used to predict stress level in every structural member under developed snow load.

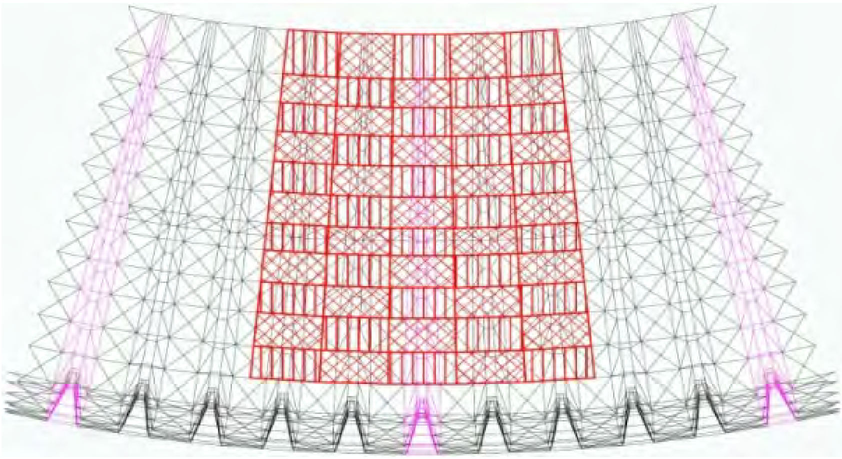


Fig. 18. Section example

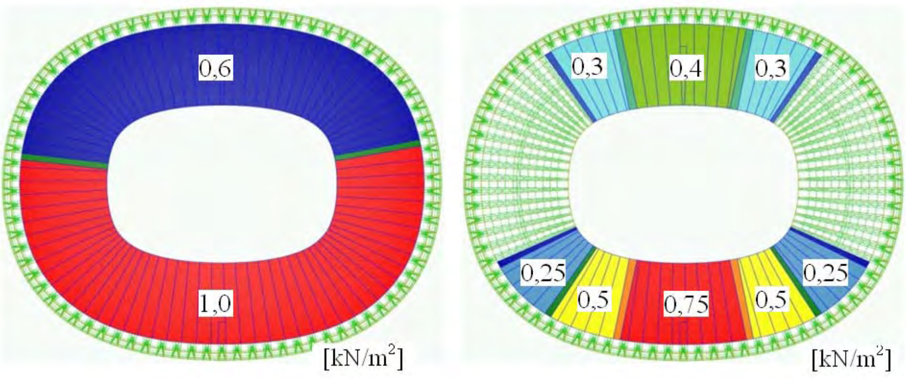


Fig. 19. Reference snow load in numerical model

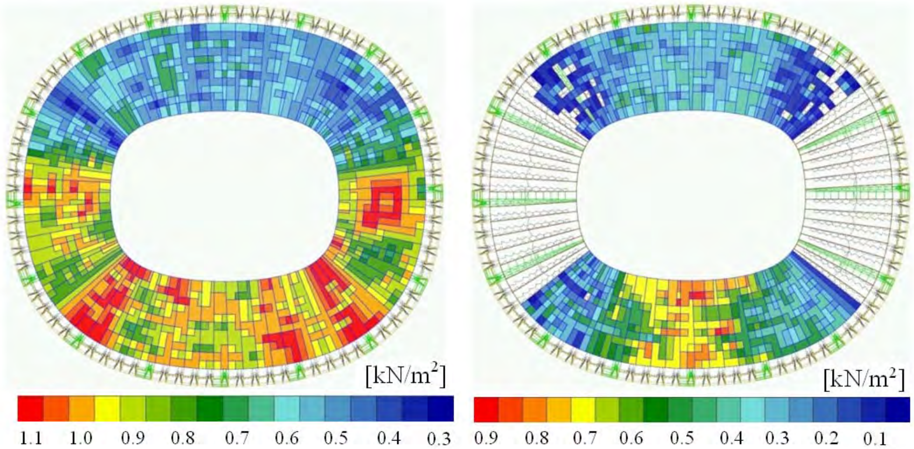


Fig. 20. Estimated snow by evolutionary algorithm for given reference load

Unfortunately, the last winter didn't allow to do a full test of the expert system. The biggest measured snow cover in northern Poland had maximum thickness of approximately 25 cm. Analysis of the monitoring data showed that the biggest deflection of a girder (67 mm) took place on 5÷6 February (Fig. 21), which is consistent with meteorological measurement of snowfall.

Fig. 21. Measured deflection of girder nr 47 on 5÷6 February 2012

The measured deflection was caused by temperature and snow load. After compensation of thermal effects, the remaining displacement was approximately 20 [mm]. the results of calculations made by the expert system are shown on figure 22. It means that a snow cover lying on the roof had locally a thickness of 10÷15 [cm]. This evaluation is approximate because of accuracy of implemented procedure.

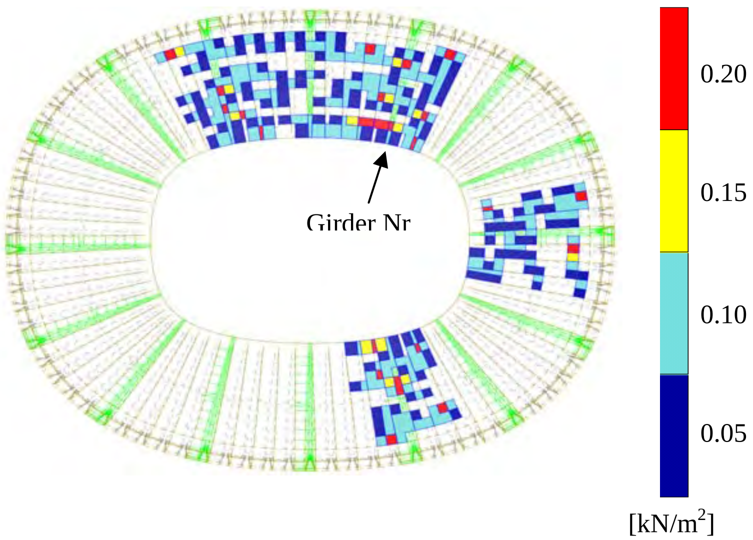


Fig. 22. Distribution of snow estimated by the system on 5÷6 February 2012

Conclusions

PGE Arena in Gdańsk is successfully opened to service thanks to continuous cooperation between the client, the contractor and the designer. Consideration of the constructing stages, imperfections and other unintentional failures was necessary for the estimation of real safety. Monitoring of the structure under environmental loads lifts safety and helps administration in the maintaining procedures.

References

[1] RKW GmbH «Architectural design of Stadium in Gdansk-Letnica» Gdańsk 2008 (in Polish).

[2] KBP Żółtowski «Design and executive plan of PGE Arena – steel structure» Gdańsk 2010 (in Polish).

[3] K. Żółtowski, T. Romaszkiewicz: «Roof of PGE Arena – The stadium built for Euro 2012 in Gdansk», Bauingenieur 3/2012.

[4] HÜGLE V. «*Wind Loads – Report N°. 592*» GBF mbH, Aachen Germany 2008.

[5] TASAK E. «Study of steel plate properties after straightening by different methods» AGH Kraków 2010. (in Polish).

[6] KBP Żółtowski «Roof over PGE Arena in Gdansk. Carrying capacity analysis of selected structural details» Gdańsk 2010 (in Polish).

[7] KBP Żółtowski «Design of monitoring system for roof structure over PGE Arena Gdansk» Gdańsk 2010 (in Polish).

Надійшла до редколегії 24.07.2012 р.