CONSTRUCTION METHOD STATEMENT FOR THE STEEL ROOF OF THE BAKU OLYMPIC STADIUM IN AZERBAIJAN

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1 SUMMARY

The present paper is focused on the steel roof that covers the Olympic stadium of Azerbaijan, Baku, and especially on the construction methodology. It addresses the selected roof segmentation and construction sequence and the verification of the various stages of the partially completed structure for temporary loadings (e.g. wind). Special attention is given to the calculations of deflections, stresses, etc., following a staged construction numerical approach and to the differences that occur with respect to corresponding magnitudes as they are calculated following the ordinary design approach (the self-weight is applied on the completed structure). The differences are quantified for critical roof components and are found to be within acceptable limits.

2 INTRODUCTION

The under construction Olympic Stadium of Baku, Azerbaijan is planned to open in 2015, to host the inaugural edition of the European Games, a multi-sport event for European athletes. The stadium will have a capacity of 66,000 spectators and is constructed by the Turkish construction company TEKFEN, under the funding of SOCAR, the state oil company of Azerbaijan. The architectural design was performed by Heerim (Seoul) and the structural one by Thornton-Tomasetti (New York). The structural system of the stadium consists of stepped reinforced concrete frames, equally spaced around the seating bowl. The spectators' seats are covered by a large steel roof. A typical cross-section of the stadium and the general plan view are shown in Figure 1.

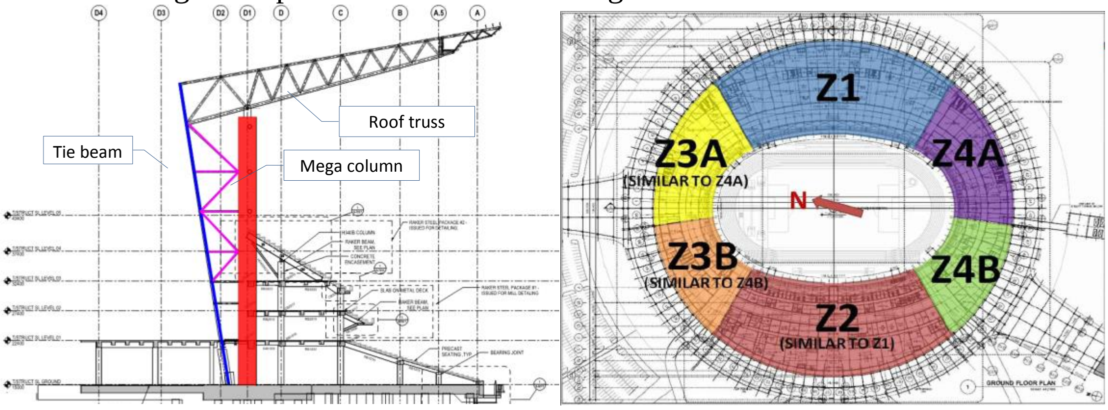

Figure 1. The typical cross-section and general plan view of the stadium

The roof consists of 60 main tri-chord trusses. Each one of these trusses is supported on a hybrid truss (Figure 1) consisting of concrete mega-columns (3.0x1.5m cross-section dimensions) and a tie-beam that develops tensile axial force due to gravity loads. Between the trusses purlins are arranged and, at specific locations, bracings that ensure the spatial stability of the whole roof. Moreover, at certain locations, corresponding to the expansion joints of the concrete sub-structure, the purlins are connected with the main trusses so that no axial force is transmitted, in order to reduce the effects of thermal expansion/contraction phenomena. These locations divide the roof in six large segments (areas Z1, Z2, Z3A, Z3B, Z4A, Z4B of Figure 1). Over the sidelines (zones 1 and 2), eyebrows elements (Figure 2) extend inwards, off of the main trusses, to provide more shade for the seating below. The structure supporting the eyebrow consists of radial tapered plate girders with aluminum louvers spanning between adjacent plate girders. The length of each main truss is roughly 54m and its weight about 90t.

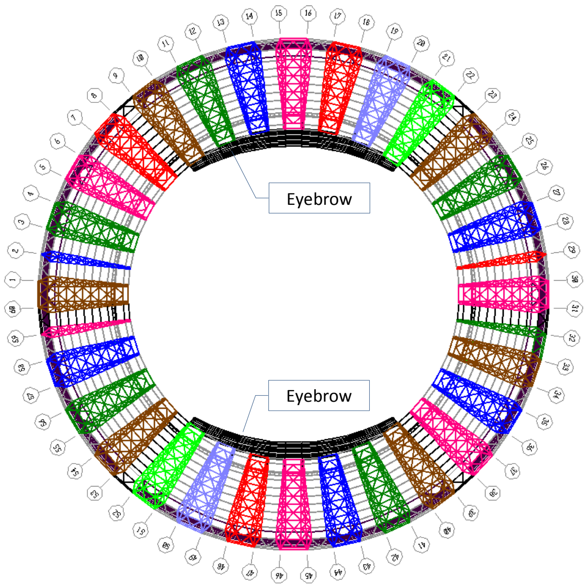

Figure 2. The structural elements of the steel roof.

3 THE ERECTION METHODOLOGY

One of the main issues of the erection was the great length and weight of the main trusses. Another significant issue that affected strongly the final erection methodology, stemmed from the fact that the installation of a single truss on top of the columns did not result into a laterally stable structure. Therefore, after the evaluation of the pros and cons, it was decided to lift and install dual truss modules, consisting of two main trusses, purlins and horizontal and vertical bracings. This decision, however, increased the weight of the modules to be elevated to more than 200t. Finally, the roof structure was divided into 28 dual truss modules and 4 single truss modules, which were unavoidable due to geometric restrictions. However, special care was given to the installation sequence of these 4 single truss modules so as to be connected to completed and already stable roof subparts. The site area was equipped with a prefabrication yard where the assembly of the roof modules took place, under the direction and supervision of the surveying team that made all the necessary geometric measurements and verifications. The completed modules were moved to a temporary storage area, from where they were, once more, moved to their final position at the perimeter of the stadium. From this last location, the modules were taken by the crawler crane that elevated and positioned them into place.

The dimensions of the dual truss modules were roughly 54x21x6.5m. Given the required distances between the crane and the stadium and the weight of the roof modules, a very large crawler crane with a capacity of 1250t (Terex CC6800) was utilized. Apart from the above mentioned main crane, 8 tower cranes were used. Moreover a 500t capacity crawler crane was employed (Terex CC4000), which moved the ready-made dual truss modules from the prefabrication bed to the temporary storage and, finally, to a location near the main crawler crane.

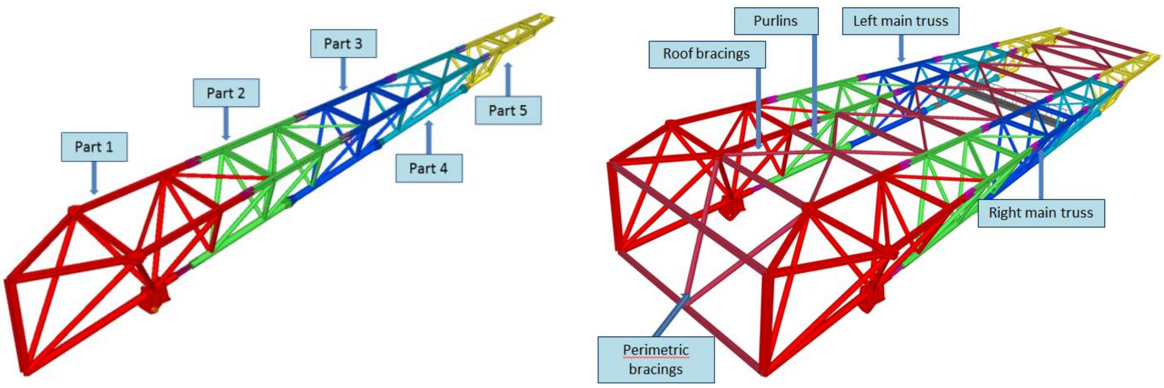

Figure 3. The single truss and a typical dual truss module.

4 ANALYSIS VERIFICATIONS

For the needs of the method statement, analysis models were setup in order to verify that the roof structure has adequate strength to sustain the loads that may be applied on it during the construction periods, for any temporary stage in which it may exist. Moreover, the developed deflections and section forces under self-weight in critical components, resulting at the end of the erection procedure, were compared against the corresponding magnitudes that develop when the roof structure is analysed as a whole (load application on the completed structure). By this comparison it was verified that the erection procedure did not actually alter the design assumptions. The AISC 360-10 LRFD [1] steel design was employed for the verification of the structural members, which was also used by Thornton-Tomasetti for the primary design.

The dominant load during the construction period was the wind one. It has to be noticed that the name Baku is widely believed to be derived from the old Persian names of the city

Bād-kube, meaning "Wind-pounded city". The wind loads were calculated according to ASCE 7-05 [2]. Instead of the 53m/s wind speed (3sec gust) that was used for the primary design, a reduced wind speed of 45m/s was adopted for the verifications (corresponding to a return period of 5 years) in order to take into account that these calculations concern the limited time period in which the construction will be completed. For the same reason, the importance factor was reduced from 1.4 to 1.0. The exposure of the structural elements was considered for 16 different wind directions. The structural elements were loaded according to their projected area normal to the wind direction. The methodology of par. 6.5 of ASCE 7-05 was followed.

The actual construction procedure was closely simulated, taking into account the installation sequence of the truss modules. Figure 4 presents the construction sequence for roof segment Z1. The construction sequence of the other segments is similar. All the possible construction stages were considered and each stage was verified independently for gravity and wind loads.

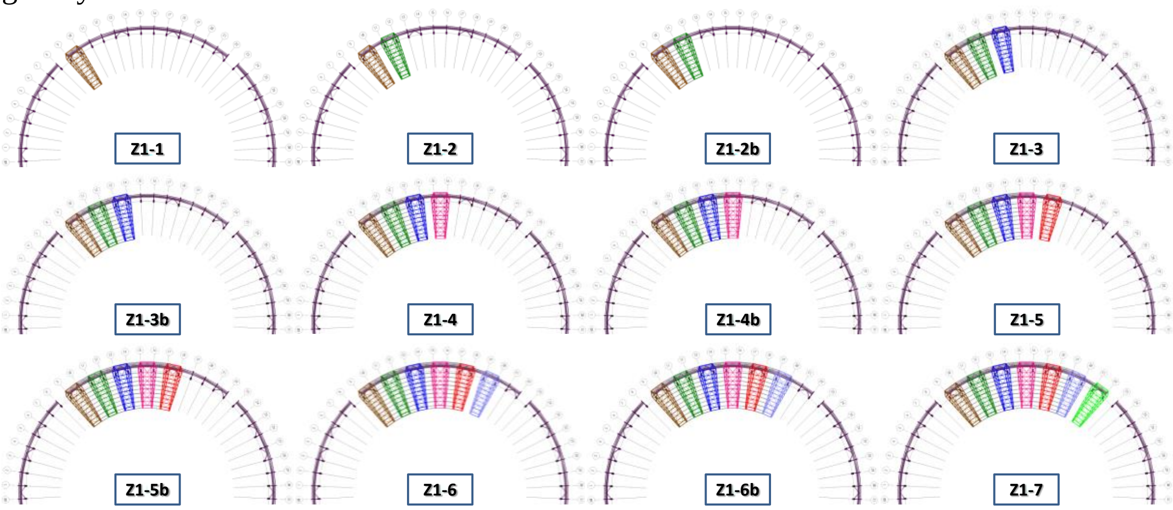

Figure 4. The various stages considered for the construction of the roof segment Z1.

4.1 Deflections at the ends of the main trusses due to gravity loading

In order to verify that the erection procedure does not affect the shape of the roof, the deflections at the edges of the trusses developing at the end of the installation sequence were compared with the deflections that would occur if the self-weight of the roof would have been applied on the completed structure (normal design procedure). The results are presented in

Figure 5. It is noticed that the deflections that resulted by following the erection sequence differ from those that would occur if the load had been applied on the completed structure. In general, the displacements computed by following the erection sequence are greater; however, the increase is rather small. Besides the absolute values of the deflections, the differences of the deflections that occur in adjacent trusses are of importance because they might affect the installation of the purlins. The maximum calculated differences of deflections are of the order of 12mm (0.14% of the purlin length) and occur at the positions where single trusses are installed (axes R2, R29, R32, R59). Special attention was given also at the positions where the eyebrows are installed. At these positions (regions between axes R9-R22 and R39-R52) the relative deflections of the edges of the trusses are of importance. The comparison is given in *a) around the stadium b) at the positions of the eybrows.*

Figure 5. It can be easily verified that the erection sequence concludes to an even distribution of edge deflections, despite being a bit larger than those that would occur by the normal design procedure.

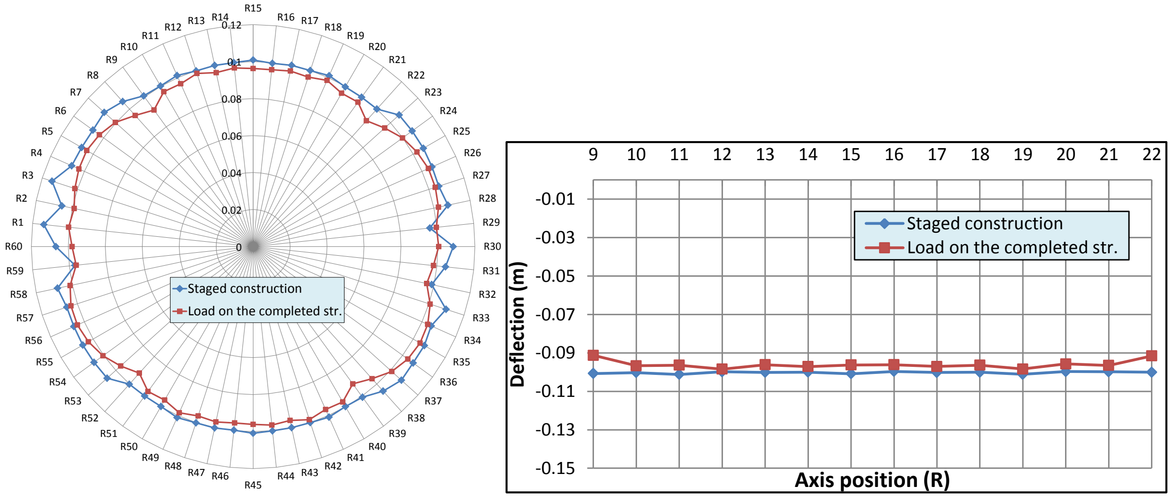

Figure 5. Comparison of truss edges deflections a) around the stadium b) at the positions of the eybrows.

4.2 Section forces due to self-weight loading

In order to verify that the erection procedure does not affect the design assumption, i.e. that the self-weight is applied on the completed structure, the section forces that develop at critical locations after the finalization of the erection procedure (staged construction analysis) are compared with those that occur under the design assumption, for critical structural components. Figure 6 presents the differences occurring for the axial forces of the top and bottom chords of the main trusses, The differences range between -4.1% to +7.7% for the axial forces of the bottom chords and between -12.8% to -6.0% for the axial forces of the top chords.

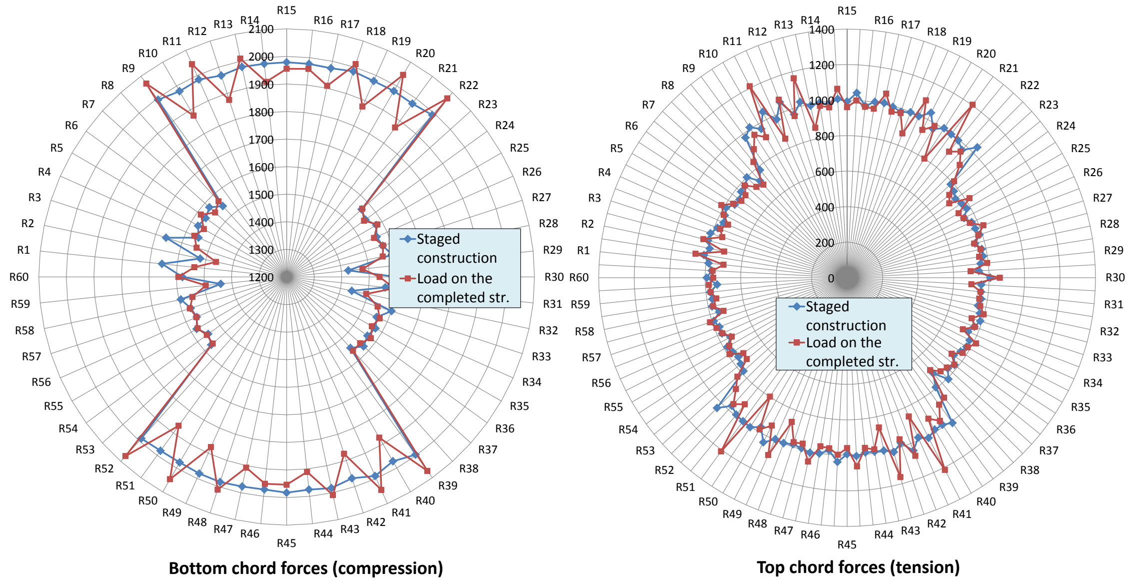

Figure 6. Differences between staged construction and normal design procedure for the axial forces of the top and bottom chords.

Figure 7 presents the differences for the axial forces at the tie-beams and for the support reactions on the mega-columns. The forces that arise in the tie beams are affected by the erection process in a greater extent than the previously studied structural elements (max +17.9%). Finally, the reaction forces on the concrete columns present minor differences (max +4.7%). Despite the recorded differences, the levels in which the structural members are stressed are only a fraction of their ultimate strength. Given the fact that apart from the self-weight, all the other design loads (including eyebrow loads) are actually applied on the completed structure, the recorded positive differences of the section forces are not significant. The most critical components resulted to be the tie-beams, in which the maximum positive differences occurred (forces resulted by the erection procedure larger than those of the normal design case). However, these differences account for about 1.1% of the tensile strength of the tie-beam, i.e. they are not significant.

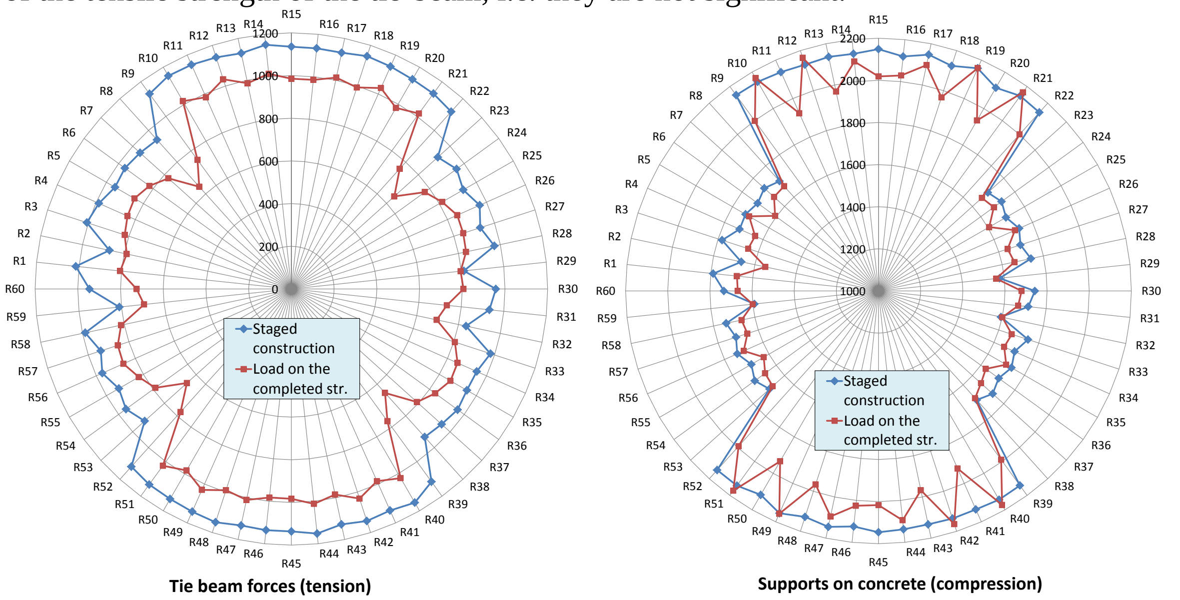

Figure 7. Differences between staged construction and normal design procedure for the axial forces of the tie-beam and for the support reactions on the mega-columns.

4.3 Verification for wind loads

As mentioned earlier, each construction stage was checked for strength and developed displacements for wind loading, considering 16 different directions of wind. In general, such wind loading had not to be considered in the original design of the structure because it corresponds to temporary situations, in which the structure is not covered by the final cladding. However, despite the fact that the wind loaded area is significantly reduced (only the areas of the structural members are loaded), the whole structure is laterally loaded in every possible direction, even in directions in which the completed structure cannot be loaded. In some of these directions the partially completed structure has low stiffness and the temporary wind loading may become critical. Although all the possible construction stages were checked, two characteristic cases are presented, the one corresponding to an isolated dual truss module and the one corresponding to two dual truss modules connected by purlins. In Figure 8, the case of individual pairs of trusses at axes R9, R10 and R11, R12 is presented. The diagrams depict the change of the horizontal and vertical displacement of the truss tip for different wind angles. Each one of the 16 values of the diagrams corresponds to a different wind direction, starting from the 0 degrees. In the horizontal displacements diagram it is noticed that the stiffness of the structure in the wind direction changes significantly with the wind angle. The maximum calculated value is about 0.13m. The vertical displacement changes also according to the wind direction. The depicted displacements include also the effect of the application of the self-weight. Moreover, the results indicate that the tips of the two pairs are, more or less, exhibiting similar displacement values for a specific wind direction.

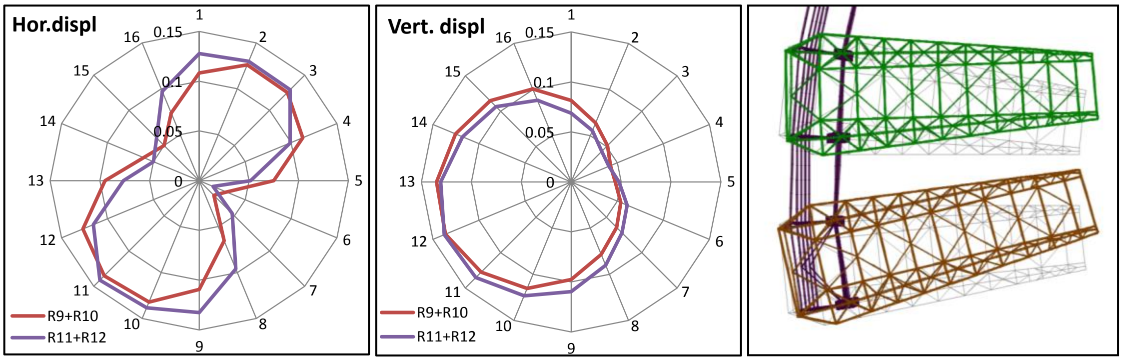

Figure 8. Horizontal and vertical tip displacements for 16 wind directions, for isolated modules.

When the two pairs of trusses are connected by the purlins (Figure 9), it is noticed that the horizontal displacements are significantly reduced. In this case, the maximum calculated displacements are of the order of 0.08m. The maximum values of the vertical displacements are actually not affected by the addition of the purlins. However, the diagram indicates that the truss tips are not moving “together”. For a specific wind direction, the windward tip exhibits smaller displacement values while the leeward tip larger ones, indicating an overall bending behavior.

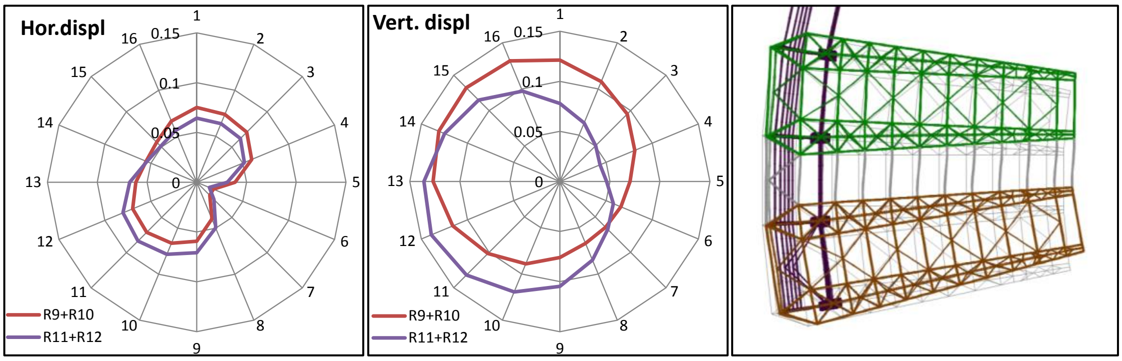

Figure 9. Horizontal and vertical tip displacements for 16 wind directions, for connected modules.

5 CONCLUSIONS

The paper refers to the method statement for the construction of the steel roof of the Olympic stadium of Azerbaijan. After the performed analyses for the various temporary stages in which the structure may exist, it was verified that the construction methodology did not affect significantly the arising stresses and deflection under self-weight, therefore, it did not invalidate the original design. Moreover, the structure was confirmed to be safe for the temporary wind loading in any spatial direction and no additional measures were necessary.

6 REFERENCES

[1] ANSI/AISC 360-10, Specification for structural steel buildings, American Institute of Steel Construction, Chicago, 2010
[2] ASCE 07 Minimum Design Loads for Buildings and Other Structures, American Society of Civil Engineers, Reston, VA, 2006.

ΜΕΘΟΔΟΛΟΓΙΑ ΚΑΤΑΣΚΕΥΗ ΣΤΗΣ ΜΕΤΑΛΛΙΚΗΣ ΟΡΟΦΗΣ ΤΟΥ ΟΛΥΜΠΙΑΚΟΥ ΣΤΑΔΙΟΥ ΣΤΟ ΜΠΑΚΟΥ ΤΟΥ ΑΖΕΡΜΠΑΪΤΖΑΝ

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ΠΕΡΙΛΗΨΗ

Το υπό κατασκευή Ολυμπιακό Στάδιο του Μπακού, στο Αζερμπαϊτζάν, θα εγκαινιαστεί το 2015, προκειμένου να φιλοξενήσει την εναρκτήρια διοργάνωση των «Ευρωπαϊκών Αθλητικών Αγώνων», που θα διοργανώνονται ανά τετραετία στο πρότυπο των Ολυμπιακών Αγώνων. Το στάδιο έχει χωρητικότητα 64.000 θεατών και κατασκευάζεται από την Τουρκική εταιρεία TEKFEN με τη χορηγία της κρατικής εταιρείας πετρελαίου SOCAR. Η αρχιτεκτονική μελέτη του έργου έγινε από την Κορεατική εταιρεία Heerim και η στατική μελέτη από την Thornton-Tomasetti στη Νέα Υόρκη. Η εργασία επικεντρώνεται στη μεθοδολογία κατασκευής της μεταλλικής οροφής του σταδίου, που αποτελείται από 60 τρίχορδα δικτυώματα που εδράζονται σε στύλους από σκυρόδεμα διαστάσεων 3.0x1.5m, ενισχυμένων με μεταλλικά δικτυώματα. Το μήκος των τρίχορδων δικτυωμάτων είναι περίπου 54m και το βάρος τους 90t. Παρουσιάζεται η μεθοδολογία κατάτμησης της στέγης σε επιμέρους ενότητες (συστήματα δίδυμων δικτυωμάτων) που αναγείρονται ξεχωριστά και συνδέονται επί τόπου με τις τεγίδες. Επίσης, γίνεται αναφορά στους ελέγχους των διαφορετικών προσωρινών στατικών συστημάτων που προκύπτουν στη

φάση της κατασκευής, για τα φορτία που ασκούνται στην περίοδο αυτή (ίδιο βάρος και ανεμοπίεση). Τέλος, γίνεται μελέτη των επιπτώσεων που έχει η τμηματική κατασκευή και διερευνώνται οι αποκλίσεις που εμφανίζονται εξ αυτής στις μετακινήσεις και στα μεγέθη έντασης σε σχέση με τα αντίστοιχα μεγέθη που προκύπτουν με τις συνήθεις παραδοχές (δηλ. ότι το ίδιο βάρος ασκείται στην ολοκληρωμένη κατασκευή).