CONSTRUCTION OF THE STEEL DOME OF THE MULTIPURPOSE AL-SADD SPORTS HALL IN DOHA, QATAR

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1 SUMMARY

The present paper presents the methodology for the construction of the steel dome of the Al-Sadd sports hall in Doha, Qatar. Certain decisions concerning the construction of the steel roof had as a result the temporary modification of the concrete structural system on which the steel roof is supported. For this reason, an extensive study was performed in order to verify that the unavoidable increase of the design magnitudes remained within acceptable levels. The paper presents in detail the construction methodology and describes the methodology that was followed for the verification, which included staged nonlinear analysis. In order to minimize the impact of the erection methodology, temporary measures were taken (back propping in certain areas and temporary braces to increase the lateral stiffness).

2 INTRODUCTION

The new sports facility of Al-Sadd in Doha is α multipurpose hall for handball, basketball, volley ball, badminton, rhythmic gymnastics and international ice rink sports. The main arena building is on a plot of approximately 16,000 m² and provides seating for 7,686 spectators. The stadium will open in 2014 and is planned to host games for the World Handball Championship to be held in Qatar. The design of the stadium was performed by James Cubitt and Partners and it is constructed by AKTOR S.A. The structural system of the arena building is made of concrete and it has seven main levels (including a raft foundation). The concrete structure consists of 8 statically independent parts, separated by expansion joints. The whole structure is covered by a steel dome which has the shape of a partial ellipsoid with main axes lengths 100m and 120m respectively, which is supported

on the concrete elements of the 5th floor. Figure 1 presents a 3D rendering of the finished building and the complete structural analysis model.



Figure 1. Architectural 3D rendering and the 3D structural analysis model of the stadium

The steel dome is roughly composed by (see Figure 2):

- a stiff perimetric ring supported on concrete
- a central arch having a length of about 100m
- secondary trusses, which are supported on the perimetric ring and on the central arch
- purlins and bracings that ensure the spatial stability of the whole steel structure.



Figure 2. The components of the steel roof

3 THE ERECTION METHODOLOGY

The erection methodology was dominated by the decision for the construction and erection of the central arch. This component (see Figure 2) had a length of roughly 100m and a weight of 270t. Due to the large dimensions and weight and the height above ground that it had to be installed, the erection was not possible in multiple pieces in a propped configuration. For this reason, a large part of the central arch, having a length of 75m and a weight of 185t was constructed as a single piece on the floor of the sports hall and was elevated at the foreseen position using hydraulic jacks. Due to this operation, the concrete structure (including the raker beams) was only partially constructed in order to give the necessary space for the construction and elevation of the central arch (Figure 3).



Figure 3. Parts of the concrete structure that were not constructed so that the main arch could pass through and elevated into the final position.

Preliminary analyses showed that the strength of the concrete structure was significantly affected by the erection methodology, as large parts of the statically independent Sections A and B (Figure 3), contributing to the lateral stiffness, could be constructed only after the finalization of the steel erection. The most significantly affected components were the core walls of Sections A and B. Therefore, specific measures had to be taken in order to reduce the stressing of these walls. To this end, an extended area was propped (see dots in the sketches of Figure 4). The props were positioned in all the floors, starting from the foundation level (back propping), prior to the casting of the 5th floor slab. Moreover, in Section A, temporary bracings were installed between certain concrete columns, in order to increase the lateral stiffness of the concrete structure. The positions where the temporary bracings were installed are shown in Figure 4. Both the temporary bracings and the props were removed after the completion of the erection of the steel roof.



Figure 4. Pack-propping positions in Parts A and B and positions of temporary bracings in Part A.

After the installation of the main arch, the secondary trusses had to be installed. It has to be noted that the secondary trusses, even though their structural system is simply supported,

when installed start receiving loads from the additional roof components installed afterwards, by means of developing compressive forces at the upper chord (in addition to the compressive forces coming from bending). Therefore, the section forces that develop at the secondary trusses depend on the specific installation sequence and this stressing is different from the one that develops if it is considered that the self-weight is applied on the completed structure (normal design assumptions). Moreover, an asymmetric installation procedure that would be advantageous from the viewpoint of minimization of the crane movements (e.g. installation of the east secondary trusses first and then installation of the west ones), concludes to significant asymmetries of the section forces. For the above reasons, an extended study was performed in order to determine a sequence that would lead to overall stressing close enough to the one that occurs under normal design assumptions. Finally, the study concluded to the installation sequence presented in Figure 6, which was optimized also for two cranes operating simultaneously. Together with the secondary trusses, parts of the bracing system were also installed so that the completed parts are always adequately supported laterally.



Figure 6. Installation sequence of secondary trusses.

The complete sequence of the erection is presented in the following.

- 1. Construction of the concrete parts till the 4th floor in Sections A and B, with the exception of the areas indicated with arrows in Figure 3.
- 2. Back propping of the areas indicated in Figure 4.
- 3. Casting of the concrete structure of the 5^{th} floor in Sections A and B.
- 4. Installation of the temporary bracings indicated in Figure 4.
- 5. Completion of the concrete construction till 5th floor, all around the stadium.
- 6. Installation of the perimetric ring steel structure.
- 7. Installation of the main steel arch.
- 8. Installation of the secondary trusses together with the required for stability roof bracings (Figure 6).
- 9. Completion of the concrete construction of the omitted parts in Sections A and B.
- 10. Removal of the back propping and temporary bracings
- 11. Completion of the steel construction (installation of purlins, cladding, etc).

4 STRUCTURAL ANALYSIS - VERIFICATIONS

The erection procedure affected the overall design of the stadium because the measures taken to increase the lateral stability of the partially constructed concrete parts in Sections A and B were not able to completely replace the missing lateral stiffness. Obviously, what was affected was the stress vector due to the self-weight of the structure. All the other loads (cladding, additional dead loads, live loads, seismic loads, etc.) would be applied on the completed structure, therefore no difference existed with the design assumptions. However, the stresses due to self-weight presented large differences with those that occurred under normal design assumptions. For example, the core walls bending moments presented differences of more than 40%.

For the above reasons, it was decided to proceed to a complete verification of the structural members that would take into account all the specific steps of the erection procedure, using nonlinear staged analysis. According to this procedure, multiple structural systems were considered, each corresponding to a real construction stage. The self-weight of the structural elements was applied on a different structural system, corresponding to the actual configuration of the structure. At the end of the procedure, the stress vector that was obtained for the self-weight load case contained the contribution from all the different (temporary) structural systems. Figure 7 presents characteristic snapshots of the analysis process. In Figure 7a (in which some concrete elements have been hidden for clarity) the introduction of the props can be noticed, prior to the casting of the 5th floor slabs. Figure 7b corresponds to the installation of the main arch, after the perimetric steel ring has been completed. Figure 7c displays the structure after the completion of the roof, prior to the removal of the props.

The verification of the structural elements was performed for the load combinations according to ASCE 7-05 [1], in which load vectors stemming from different structural configurations were combined. More specifically, the load vector corresponding to the self-weight of the structure was determined following the above described staged construction procedure, while the load vectors of the rest load cases were determined by applying the corresponding loads on the completed structural system. The calculations were done according to ACI 318-08 [2] for the concrete elements and according to

AISC360-10 [3] for the structural steel ones. The verifications included also the raft foundation of the stadium.



Figure 7. Snapshots of the different structural systems that were considered.

As an example of the extra stressing that the construction methodology causes to the core walls and of the procedure that was followed, the case of the core walls of Section A is presented. Figure 8 presents the bending moments of the core walls due to self-weight as they are calculated applying the normal calculation practice (load applied on the completed structure) and applying the staged construction methodology. It is noticed that there is an increase of the bending moments due to the specific methodology followed, by roughly 45%. Moreover,

Figure 9 presents the enveloping values of the **design** moment that correspond to the two calculation cases. It is noticed that design bending moments are increased by only 16.5%



Figure 8. Bending moments of core walls of Section A due to self-weight.



Figure 9. Design envelope of bending moments for the core walls of Section A.

Moreover, the verification of the core walls (comparison between the reinforcement requirements and the installed reinforcement) led to the conclusion that the highest strength utilization factor was of the order of 85%, therefore, despite the increased design magnitudes, the core walls had a significant margin from failure.

5 CONCLUSIONS

The study yielded the following conclusions concerning the effects of the method statement on stressing that develops at the structural elements of Sections A and B and at the raft foundation:

- It was inevitable to avoid some amount of extra stressing at the structural elements due to the construction methodology. The most affected elements were the core walls of Sections A and B, however, the full design verification showed that the strength utilization factor of these walls remained in acceptable levels.
- The columns and beams were not affected by the construction methodology and no indications of extra stressing existed. The study concluded that the columns were safe, with a significant distance from failure, while the required reinforcement for the beams remained lower than the existing one.
- The initial design of the raft foundation had also reserves and despite the increased loads that occurred due to the erection methodology, no additional reinforcement requirements occurred.

6 **REFERENCES**

- [1] ASCE 7-05 Minimum Design Loads for Buildings and Other Structures, American Society of Civil Engineers, Reston, VA, 2006.
- [2] ACI 318-08: Building code requirements for reinforced concrete. American Concrete Institute, 20089
- [3] ANSI/AISC 360-10, Specification for structural steel buildings, American Institute of Steel Construction, Chicago, 2010.

ΜΕΘΟΔΟΛΟΓΙΑ ΚΑΤΑΣΚΕΥΗ ΤΟΥ ΜΕΤΑΛΛΙΚΟΥ ΘΟΛΟΥ ΤΟΥ ΚΛΕΙΣΤΟΥ ΓΥΜΝΑΣΤΗΡΙΟΥ AL-SADD ΣΤΗ ΝΤΟΧΑ, ΚΑΤΑΡ

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ΠΕΡΙΛΗΨΗ

Στην εργασία παρουσιάζεται η μεθοδολογία κατασκευής του μεταλλικού θόλου που καλύπτει το νέο κλειστό γυμναστήριο της Al-Sadd στην Ντόγα. Το πολλαπλών γρήσεων στάδιο κατασκευάζεται από την εταιρεία ΑΚΤΩΡ. Πρόκειται για μια εγκατάσταση σε έκταση περίπου 16.000 m² και θα μπορεί να φιλοξενήσει 7.686 θεατές. Ο σχεδιασμός του σταδίου έγινε από την μελετητική εταιρεία James Cubitt and Partners. Το στατικό σύστημα περιλαμβάνει οκτώ στατικά ανεξάρτητα τμήματα από οπλισμένο σκυρόδεμα πάνω στα οποία εδράζεται μια ελλειψοειδής μεταλλική στέγη με κύριους άξονες μήκους 120m και 100m αντίστοιχα. Το κύριο δομικό στοιχείο της στέγης είναι μια τοξοειδής κατασκευή (κεντρικό τόξο) μήκους περίπου 100m και βάρους 270t, πάνω στην οποία εδράζονται δευτερεύοντα καμπύλα δικτυώματα. Λόγω των μεγάλων διαστάσεων και του βάρους του κεντρικού τόξου, καθώς και του μεγάλου ύψους τοποθέτησης δεν ήταν εφικτή και οικονομικά συμφέρουσα η τμηματική κατασκευή με υποστύλωση. Αντίθετα, προτιμήθηκε η κατασκευή ενός μεγάλου τμήματος του κεντρικού τόξου (μήκους περίπου 75m) στο δάπεδο του γυμναστηρίου και η ανύψωσή του με σύστημα υδραυλικών πιεστηρίων. Για να καταστεί όμως εφικτή η παραπάνω λύση, έπρεπε ένα τμήμα του σκυροδέματος προσωρινά να μην κατασκευαστεί, γεγονός που οδηγούσε σε μειωμένη πλευρική δυσκαμψία. Στην εργασία περιγράφεται ο τρόπος που αντιμετωπίστηκε μελετητικά το παραπάνω θέμα. Συγκεκριμένα ακολουθήθηκε ανάλυση φάσεων κατασκευής για τη φόρτιση του ιδίου βάρους του σταδίου, στην οποία προσομοιώθηκαν με λεπτομέρεια τα στάδια κατασκευής. Για τη μείωση της καταπόνησης των δομικών μελών, απαιτήθηκε η λήψη προσωρινών μέτρων (υποστυλώσεις, αύξηση πλευρικής δυσκαμψίας). Τελικά, διαπιστώθηκε ότι η αύξηση των μεγεθών έντασης λόγω της τμηματικής κατασκευής ήταν εντός αποδεκτών πλαισίων.