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Iconic Campus of the Zayed University Abu Dhabi

Christian Böttcher
Matthias Frenz
Henning Kaufmann

The Emirate of Abu Dhabi is planning for the post-oil era, so education plays a central role in its Master Plan 2030. The new building complex of the Zayed University, intended for 6,000 students, is located in the future Capital District directly on the important connecting road between the Abu Dhabi international airport and the old town peninsular. Hamburg architects BRT's (Bothe Richter Teherani) symbolism-rich design moulds the 100.000 m² GFA ensemble of buildings in the central area into a huge sculpture. The linking element is a jointless free-form roof of 8,000 tonnes of steel with aluminium cladding whose shape and lightness echoes the traditional Arabic chador.
A very short time for design and construction required close cooperation within the international team set up by a main contractor. The project required mastering of numerous engineering challenges, different design philosophies, difficult interfaces between design and construction works on a huge construction site with up to 8,000 workers running in parallel to the design works.

1 Introduction

The Emirate of Abu Dhabi is planning for the post-oil era, so education plays a central role in its Master Plan 2030 [1]. The new building complex of the university, named after the late national founding father H.H. Sheikh Zayed bin Sultan Al Nahyan [2], is located in the future Capital District directly on the important connecting road between the Abu Dhabi international airport and the old town peninsular.

Up to 6,000 students will be housed and taught on the 75 ha grounds (Figure 1). Accommodation, sports facilities and stores are available on a gender-segregated basis, located in the east campus for men and in the west campus for women. The central campus area includes a conference centre and a library as well as administrative buildings, faculties and cafeterias. These facilities are also gender-segregated using structural and scheduling arrangements. A total of approximately 213,000 m² in gross building area (GFA) will be used, with ca. 100,000 m² of this located in the central area.

Shortly after the end of 2008, when the contract for the entire building complex was awarded to an Arabian-South African-Australian general contractor and the construction site set up, the design was reworked in a small invitation-based competition, searching for an improved iconic and symbolic design value. The design submitted by architects BRT Bothe Richter Teherani from Hamburg was selected and became the basis for assignment in April 2009. It moulds the ensemble of buildings in the central area into a huge sculpture. The linking element

Fig, 1. Central Campus with Feature Roof is a jointless free-form roof of 8,000 tonnes of steel with aluminium cladding. The design intent for the shape of the building ensemble and the sculptural roof was the shape and lightness of the chador, a semi-circular cloth worn by Muslim women as a mantle that exposes only the face or parts of it.

Due to the loss of time caused by the change of design and design team, only 27 months of design and construction time remained as of April 2009 until the contractually required handover date of the complete campus in July 2011.

BRT architects were assigned mainly up to design development and interior design. Subsequent to this, Pascall + Watson architects were comissioned for the detailed documentation of the buildings. For the free-form roof, however, the detailed design including all coordinating tasks and clarification of interfaces was done jointly by the structural engineers, the main contractor's project managers, the steelwork contractor and cladding contractor. Because of the complexity, the structural engineers were fully assigned from the initial idea until completion of the roof.

The structural design had to be completed during the remaining 8 months of 2009 and was divided into three design steps: concept design, design development and detailed design.

During form finding of the concept design, 16 different primary shapes and a corresponding number of secondary variations of the geometry and the structural design were developed and tested in close cooperation with the architects. Only 12 weeks after assignment and developed from a total of 80 variations, the final and efficient form was agreed. This was only made possible by the use of a holistic, fully parameterised 3D-architectural model of the buildings and the free-form roof in Rhinoceros® in conjunction with a new software developed in-house by the structural engineers that generates the structural elements within the architectural model. The development, programming and verification of this software were carried out during the project period in parallel with the individual design steps. This BIM-equivalent approach is documented more in detail in [6].

The first invitations to tender for the steel structure of the free-form roof were placed even before finishing the concept design. The subsequent early assignment of the steelwork contractor allowed local market characteristics of material availability, fabrication and assembly methods to be included in the further design process.

Sixteen weeks after the start of the design phase and before completion of the design development phase, the procurement for the gross steel tonnage had to be placed because of an expected significant increase in steel prices in the autumn of 2009. Also at this time, the individual cross sections had to be designed and agreed for the upcoming pre-fabrication. Selected simple parts of the steel structure were identified and fully designed so that these parts could be fabricated in advance.

The structural design mainly had to be completed by the end of 2009 with all corresponding connection forces and the full three-dimensional geometry to be submitted digitally to the steelwork contractor. Finally, the structural engineers had to check the workshop design provided by the steelwork contractor in parallel with already ongoing fabrication and also had to provide a full set of calculations regarding assembly planning and presetting.

2 Buildings in the Central Area

The central campus includes the following buildings on a north-south axis: Convention Center (CON) with 33.000 m^2 GFA including huge column free conference rooms and a theatre with 1100 seats, the Administration (AF2) with 16.000 m^2 GFA, the campus, faculties (Interdisciplinary Studies, IS), dining halls (DH) totalling 17.000 GFA and a Library (LIB) with 20.500 m^2 GFA accomodating up to 500,000 volumes and four lecture halls. Figure 2 shows the arrangement of these buildings under the free form roof. The roof slabs of the CON (shown blue), AF2 (shown yellow) and LIB (shown red) are used for bearing points of the overlying free-form roof. The faculties (green) are free from loads of the feature roof. A more detailed description of the buildings is given in [5].

3 Free-Form Roof

3.1 Principles of the Design

3.1.1 External Appearance

The sculptural roof was conceived to be a jointless, continuously curved shape of aluminium cladding with a constant overall thickness of only 1,75 m.

The challenges inherent in this were multifaceted. Along with high architectural requirements of evenness, continuity of curvature, clean lines and non-visibility of joints, emphasis fell on cost, ease and time of construction, the height remaining for

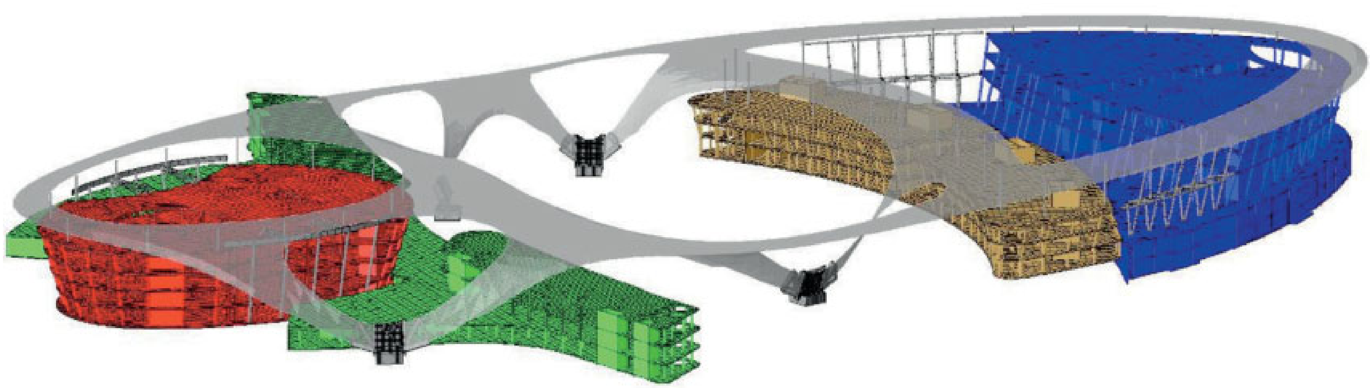


Fig. 2. Overall structural model of the Central Campus

the steel structure and the extreme environmental boundary conditions. Due to the proximity to the Persian Gulf and the high-level, salty ground water, the local dust and sand are slightly saline. With the considerable dewfall in the morning hours, especially in summer and autumn, and the high temperatures during the day, this leads to a „baking" of the dust and sand, to be countered by the especially smooth surface and special joint construction of the cladding. Alternative roof claddings such as a standing seam roof cladding would not serve.

The roughly 25,000 aluminium panels have typical dimensions of ca. 1500 mm × 1500 mm by 3 mm thickness and are mounted on their own substructure at the upper side ceiling and the soffit. This left a structural height of constantly only 1.50 m.

3.1.2 Interaction with the Buildings

In order to preserve the slim appearance of the free-form roof, it was essential to support it from the buildings wherever possible. For the Convention Center (CON), the Administration (AF2) and the Library (LIB), interaction with the roof was thus an additional design requirement that mandated a high level of coordination and the use of full 3D-modelling.

According to the architects' specifications, all structural supports of the roof needed to be nearly invisible. This was achieved by using very slender columns, large spans, special colouring and positioning away from the edges of the building.

The limited space for stiffening elements within the buildings (e. g. cores) required to keep the buildings free from any earthquake loads or temperature related loads from the free-form roof.

3.1.3 Sustainability

Sustainability played a major role in the design. In light of the extreme environmental conditions and architectural desires, the goal was to create a jointless, preferably bearingless steel structure with low steel consumption, low maintenance requirements and the use of local products and workmanship where ever possible.

3.1.4 Materials, Joining Technology, Production and Assembly Capacities

For reasons of time and expense, the particularities and capacities of the local market had to be observed when selecting materials, defining joining procedures and choosing production methods.

So, for example, no steel grades better than S355 J0 were used and no special requirements for through-thickness direction (so-called Z-quality) were imposed. The plate thicknesses did not exceed the market-typical value of 50 mm. On-site welding was avoided where ever possible. Instead, high-strength friction grip (HSFG) bolts were used.

For the assembly of the free-form roof the standard tower cranes from the buildings were used wherever possible. The remaining assembly parts were chosen to be as large as possible and installed with heavy mobile cranes. The number of temporary supports was minimised in order to ensure good accessability on site.

3.1.5 Official Requirements

The municipal authorities in Abu Dhabi have not issued their own technical regulations or standards so far. There are only guidelines that refer to foreign regulations and for example offer additional specifications for wind and earthquake loads. For historical reasons, design in the Arabic world is strongly influenced by British standards. In close cooperation with and after intensive discussion with the authorities and the main contractor, it was agreed to base the structural design on the Eurocode and the British National Application Document.

3.1.6 Structural loads

- **Temperature** was taken into account with 29 °C as medium value and ±22 °C for deviations during the course of the day and an increased value of +30 °C only for the construction time due to directly to sunlight exposed steel.
- **Sand and Rain** loads were covered with a vertical load allowance of $q_k = 0.75$ kN/m^2 in consultation with the owner and the authorities. This covers potential sand accumulations of 3–5 cm thickness on the surface and the relevant waterfilm thickness during heavy rain falls. Accumulations of water and sand in the interior of the roof had to be prevented.
- **Wind** loads for this special geometry were determined via wind tunnel tests at a scale of 1:400. The adopted basic wind speed was taken from bureau of meteorolgy measurements at the Abu Dhabi airport over the past 30 years. As a result of the wind tunnel tests, detailed distributions of the wind loads for the upper and lower side of the structure for 30 degree steps of the wind direction were given (Figure 3). Besides the quasi-static wind loads,

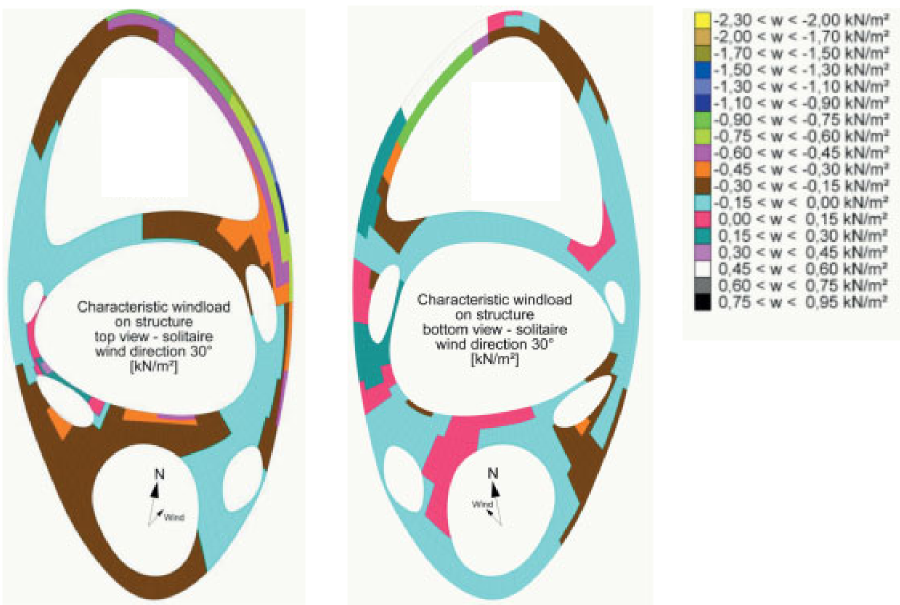


Fig. 3. Example for wind load distributions

the dynamic behaviour of the roof due to wind effects was also relevant, but due to the time constraints, only engineering judgement and calculations could be used in order to address this aspect.
– **Earthquake** loads were taken into account for Zone 2A of the UBC 1997. A ductility of R = 4.2 and an importance factor of I = 1.0 were chosen.

3.2 Design of the Steel Structure

3.2.1 Concept Design and Form-Finding

Initially, the principal load-bearing and deformation behaviour was analysed by simplified estimations and calculations, followed by 3D-shell models, as well as by the development of characteristic sections (Figure 4), and by curvature analysis.

To achieve a cost effective solution within the limited 1.50 m structural depth, the shell-like load-bearing behaviour of the structure, characterised predominantly by normal forces, was optimized in close cooperation with the architectural design. As a result, the global load-bearing behaviour could be achieved and characterised as follows:

In the area of the Convention Center (CON), the free-form roof is ring-shaped and architecturally desired mostly flat, so no shell-like load-bearing behaviour could be activated there. This required a structural ring capable of torsion and bending as well as narrow spaced supports from the buildings. The same applies to most of the area next to the Library (LIB). The curves in the central area (Center), however, permit shell-like load-bearing behaviour.

The three areas above interact strongly positive in case of an overall jointless system, which suits also very well with the design principles as per chapter 3.1.

The various options for geometric data exchange between the architects and structural engineers were checked for suitability early in the design. The architects used Rhinoceros® to develop and coordinate the outer shape geometry of the roof and the buildings in an overall model.

To arrange the complex steel structure within the roof shape, the configuration of the structural grid was first worked out in plane projection, based on the previously identified principles of an optimum global load-bearing behaviour and an empirical grid of 5.00 × 5.00 m.

Then this two-dimensional configuration was projected onto the middle surface of the free-form roof using an in-house developed software. The spatial coordinates of the free-form roof required to do this were acquired from the architects' geometrical data. The spatial orientation of local beam axes was also determined automatically by aligning the strong axes of the beams in the directions of the normal surface vectors of the architectural model.

Starting from these initial structural calculations, the shape of the free-form roof and the spatial configuration of the structure within were interdisciplinary optimised (Figure 5). Additionally, the reliability and applicability of the 2nd-order theory implemented in the structural software was checked [3].

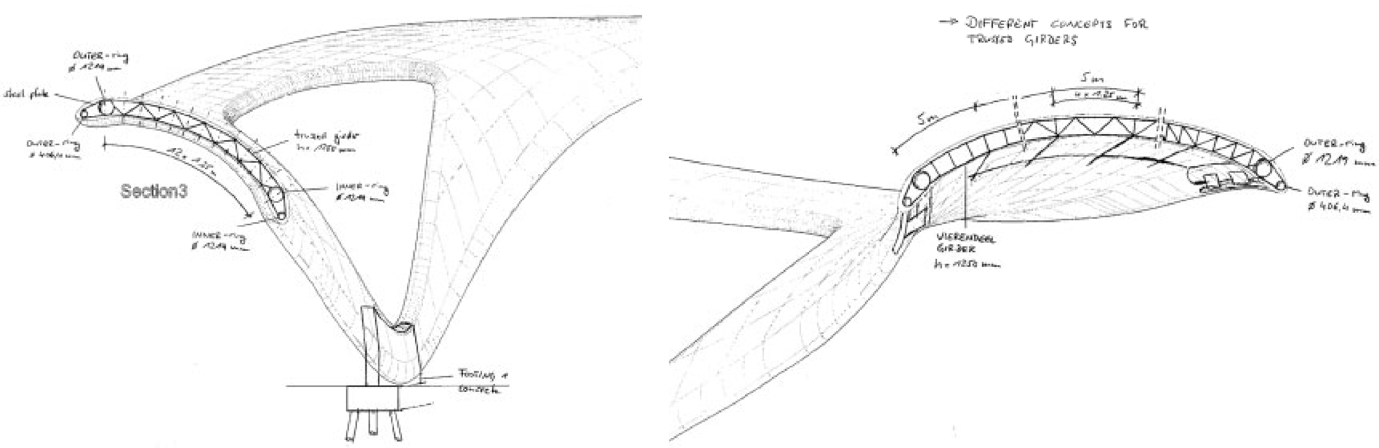


Fig. 4. Development of characteristic sections

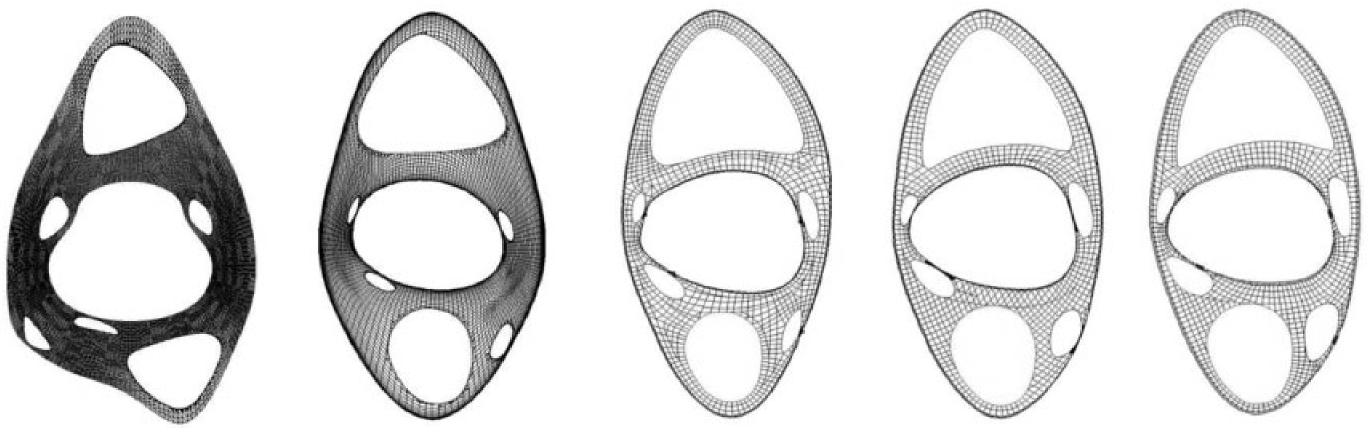


Fig. 5. Evolution of the roof shape during form finding

3.2.2 Design Development

The internal structure of the free-form roof already developed in the concept design phase and refined in several steps during design development is shown in Figure 6 in its hierarchical arrangement.

The primary components include the two rings of the Convention Center and the Library, which were built as heavy, welded rectangular hollow sections of 2000 × 1500 mm due to their governing need to resist torsion. These two rings are connected directly to the central area, whose shell-like load-bearing behaviour is based on an upper tension ring with combined bending capacity and additionally wide-spanning arches. The central tension ring consists of welded quadratic hollow sections of 1500 × 1500 mm. Parts of it are held in position by supporting rings projecting from four integral fix points (so called buttresses). These rings are also made of welded rectangular boxes of 1500 × 1500 mm. The wide span arches, up to 4.0 m deep in cross section, which are directly supported by the integral fix points, mainly consist of welded circular hollow sections of 1500 mm in diameter and tubes of 406 mm in diameter. An overview of the sections used is given in Figure 7.

The integral fix points must simultaneously guarantee the overall stiffening of the roof and also rigid support for the wide-span arches placed on them to prevent their failure by combined bending and buckling. To master the attendant concentrated loads, the 1,50 m thick integral fix points were designed as reinforced concrete composite structures fixed into 2.50 m thick pile caps (Figure 8). In order to avoid large concrete volumes which are difficult to compact and associated laborious three-dimensional formwork, the majority of the forces to be redirected were dealt with in the steel structure in the so-called adaptors. The up to 8 m high and 7 m wide middle area of the integral fix points was made primarily of concrete for the purposes of easier fixing to the pile caps. This was despite the multiple reinforcement layers, the need for coupler–based reinforcement connections and self-compacting concrete C80.

The numerous truss elements of the remaining structure, including internal bracing elements and stiffening edge beams, are called secondary components. Their spatial configuration and detailing in the structural model was optimised step by step. During the concept design phase, all later trusses were modelled as equivalent beams between the primary nodes. From the late design development and for the detailed design, all the 22,000 individual diagonals, posts, etc. of the trusses had to be specified in the structural model (Figure 9).

This could only be done using pre-processing, developed in-house especially for this task. All finite chord and strut elements were automatically generated based on a pre-analysis of stress distributions from dead load. The global orientation of the truss elements was derived from the previously determined normal vectors. Additionally implemented results from independent 2nd order flexural torsional buckling analysis were used to achieve the safe and economic spatial configuration.

CHS-bracing is used to support the shell-like behaviour, as the 5 m × 5 m structural grid is primarily orthogonally oriented for reasons of easier fabrication and assembling.

The outer roof edges were, both for architectural and structural reasons, specially shaped using downward sloped, rounded brims. Pipe truss girders with a structural depth of 3.0 m were used here to provide additional stiffness, following the shape of the brim. The cladded shape of the roof edges is so advantageous that an

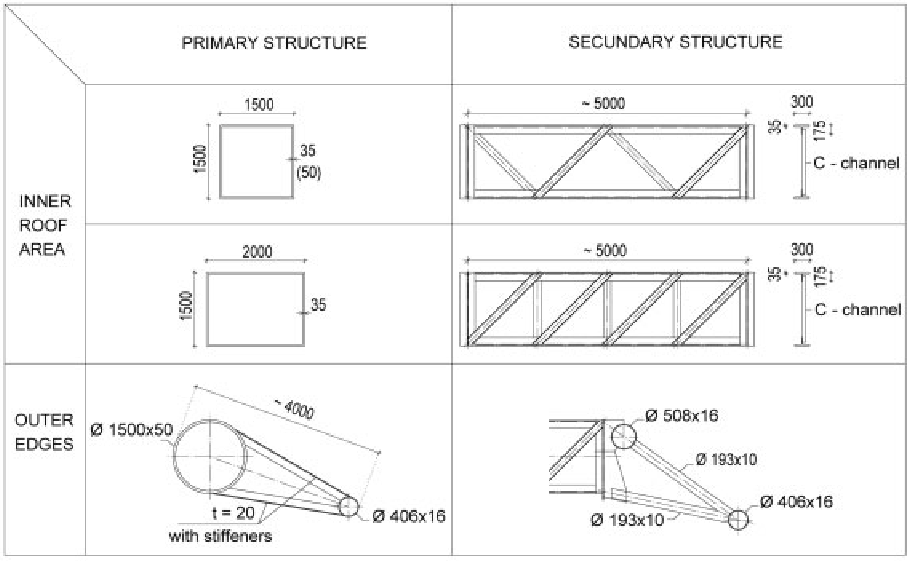


Fig. 7. Overview of main sections

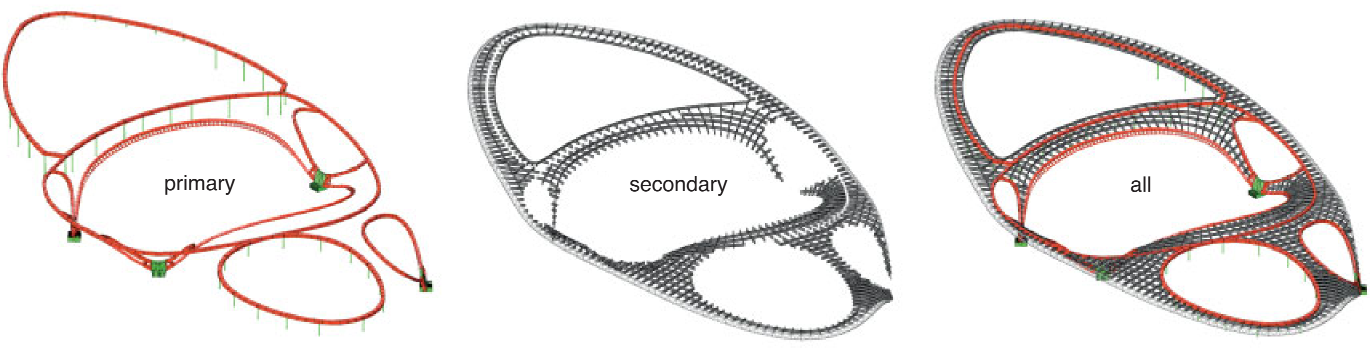


Fig. 6. Hierarchical arrangement of the structural elements

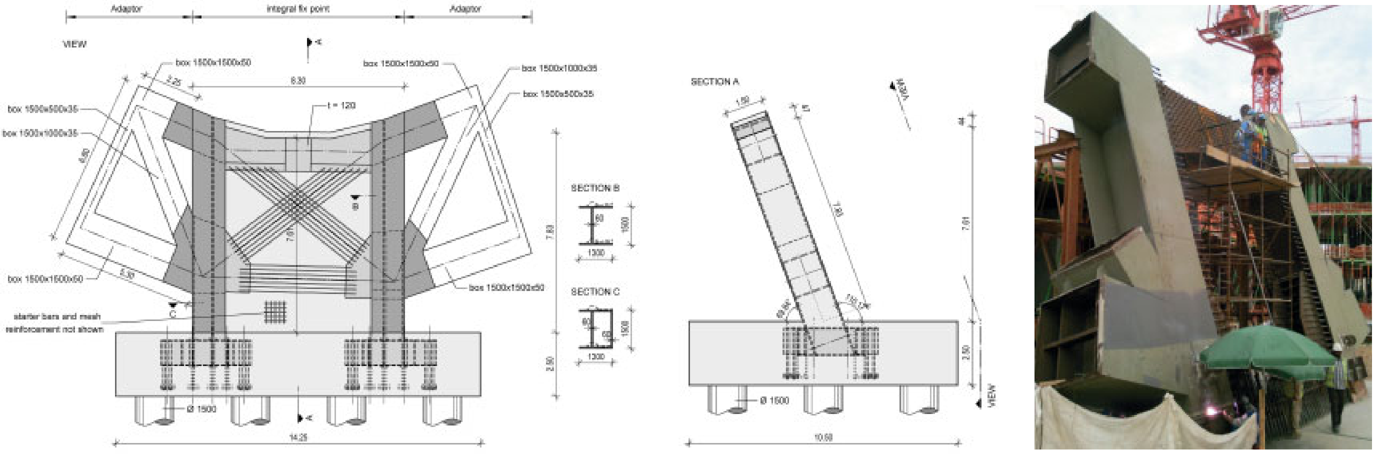


Fig. 8. Drawings and assembly of integral fix point

alternating tear off from air currents and oscillations, feared especially for frequent low wind speeds, will not occur. In general CFD calculations or extra tests addressing the aerodynamic stability were not possible due to the time constraints. Engineering experience and calculations based on Eurocode 1 were used instead.

The so-called tertiary components not further described here include the substructure of the cladding, consisting of cold-formed Z-purlins and hollow sections and do not belong to the main load-bearing structure.

3.2.3 Detailed Design

The spatial structural configuration was finalised in the detailed design phase based on calculations of the full 3D-system according to 2nd order theory. The buckling behaviour of the truss and chords around the weak axis was especially relevant. As agreed, the full set of connecting forces and the final spatial configuration was handed over digitally to the steelwork contractor by the end of 2009 for further procurement and fabrication detailing. The full documentation of the detailed design was issued in February 2010.

3.2.4 Execution Planning and Work Planning

The basis of the steelwork contractors further detailing for fabrication had to be a three-dimensional presetted structure due to large localised deformations under dead load. The presetting of the structure had to be defined also by the structural engineers due to the complexity of the structure and the time constraints and handed over digitally as a presetted spatial configuration to the steelwork contractor.

For this, the highly unequal structural deformations (Figure 10), arising in the ideal structure due to its dead load were repeatedly transferred to the structural model as recursive node displacements until the deformed structure and the architecturally desired form agreed perfectly with each other. It is particularly important to note that horizontal presetting of up to 100 mm became necessary in addition to the vertical presetting in the area of large spans of up to 450 mm. The reason is the asymmetrical position of the four integral fix points and the resulting considerable horizontal thrust.

The presetted and structurally rechecked geometry was transferred directly from the structural model to the steelwork contractor's working model, equivalent to an execution planning. All cross sections were given along with the node coordinates to avoid errors due to the large number of over 22,000 elements.

The steelwork contractor was responsible for the connection design based on the presetted geometry and incorporated the detailing into the 3D model of the work planning. In view of the considerable extent of about 16.000 closely printed DIN A4 pages of connection forces, the structural engineers had to provide considerable assistance in standardising the connections and in preparing the relevant forces.

Shop drawings were then prepared showing the individual construction sections in plan view based on

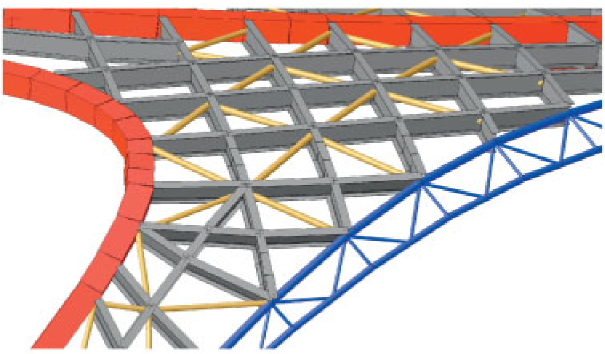
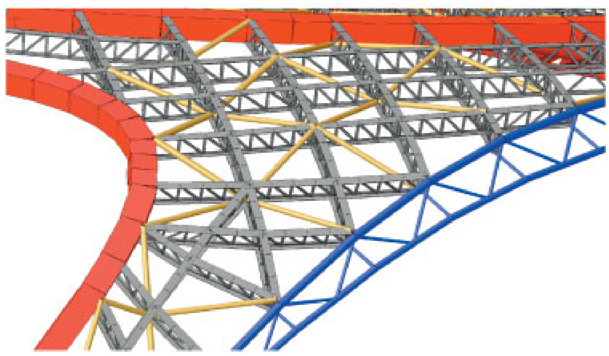
simplified spatial configuration	full spatial configuration
	

Fig. 9. Comparison of simplified and full spatial configuration

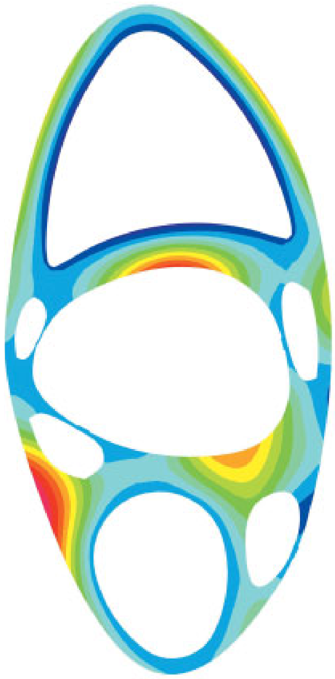


Fig. 10. *Vertical deformations under dead load*

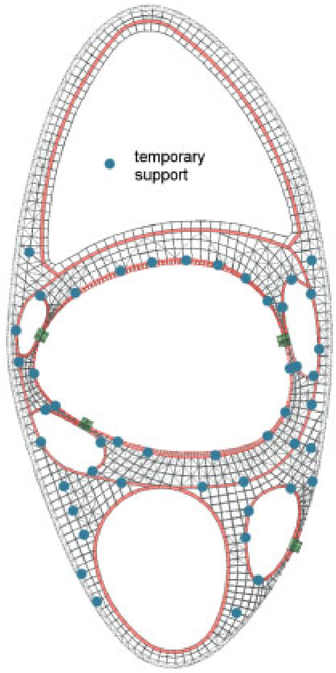


Fig. 11. Arrangement of temporary supports (trestles)

the steelwork contractor's 3D model of the work planning. Additional to the elements displayed, drawings of the important connections and particular construction details were provided in order to allow another check by the structural engineers and their approval for execution. The simultaneously prepared fabrication drawings were not submitted for approval; they contained all the information for the construction of the steel structure. A total of 420 shop drawings and approx. 6,500 fabrication drawings were needed for the project.

3.2.5 Assembly Planning

The accurate planning of the individual erection stages of the roof with related assembly sequences was particularly important due to the simultaneously ongoing complex construction works on the buildings and campus ground and the need to keep access routes open. The required solution needed to have as few temporary supports (trestles) as possible. Another issue was the limited lifting capacity of the available tower cranes, which were supplemented by heavy mobile cranes.

It was also important to investigate that during assembly larger lifting segments can show load-bearing and deformations differing considerably from the final stage. The compatibility of deformations at the final stage was needed to avoid affecting the global load-bearing behaviour due to built-in imperfections. The lifting segments requested by the steelwork contractor were therefore verified by the structural engineers in iterative calculations, including checks of the connection design and the exact erection sequence. The number of trestles was also optimised as part of this process. Besides a number of trestles for sub erection stages, only 47 trestles were necessary by the end of general assembly, mostly in the area of the central ring structures (Figure 11 and 12), which maintained the presetted form of the structure. Work on the roof cladding commenced immediately after the assembly of the secondary elements within each area making it necessary to take into account additional loads due to hanging scaffolds and wind effects during construction.

First, the primary rings were assembled with lifting segments up to 30 m long and 50 t in weight using heavy mobile cranes. Until the primary ring structure was connected to the integral fix points, the pin-ended columns were replaced by braced trestles in the area of the buildings to ensure sufficient horizontal stability during construction. Even with only partially built primary rings, the secondary structure could be gradually assembled in a consistent sequence from south (LIB) to north (CON) (Figure 13).

The depropping of the structure from its temporary supports using hydraulic jacks was another critical assembly step. The planning and supervision involved was correspondingly extensive. First, measures were taken to avoid local eccentricities at the support points. Four symmetrically placed jacks with up to 100 t capacity were used per support point. Next, it was checked whether all trestles could be lowered at the same time or whether parts of the roof would have to be lowered step by step. As a result, 11 sufficiently independent trestle groups were identified using the deformation figures under dead load and taking into account loads on the trestles – see Figure 14. The jacks within these groups were lowered simultaneously by a pre-defined amount until the roof

Fig. 12. Trestles under central ring structure

Fig. 13. Erection of the secondary structure subsequent to primary ring structure

was self supporting. Some areas were sequentially lowered by only 50 %; they were then lowered the remaining 50 % of the way in a second run-through.

3.3 Quality Control

3.3.1 Mock-up

A complete segment of the Convention Center roof was used as mock-up to check the appearance, colouring, functionality and installation methodology before beginning the erection work (Figure 15). Flaws identified here were corrected in further planning and the installation methodology adjusted.

3.3.2 Inspection and Supervision

Due to the rough assembly conditions in the desert, often only basically qualified workers, and considerable time pressure, a three-step quality control process was introduced. First, the steelwork contractor carried out agreed inspection and quality checks in the shop and on site. This was supported by the structural engineers checking the documents submitted by the steelwork contractor and by the main contractor providing comprehensive supervision of execution works on site. Finally, the structural engineers carried out a spot check inspection for the erection of critical building elements. This three-step system has proven reliable on site.

To check the prestressed non-slip in service screwed connections during assembly, a DTI (Direct Tension Indicator) procedure with separate washers was used and the preload forces were verified by random spot checks.

Where joints with butt welds were necessary, e. g. on the primary ring structure (Figure 16), these were checked both in the shop and on the site 100 % visually and non-destructively by ultrasonic and magnetic tests. Fillet welds, used only at a few secondary points, were checked 100 % visually and 10 % non-destructively.

To verify the exact position of the structural elements during assembly and recognise any settlement of trestles, permanent measurements of the structure at narrow spaced measuring points were carried out [4]. Errors due to temperature effects were taken into account and compensated for by multiple measurements.

The considerable – up to 450 mm – horizontal and vertical deformations of the structure occurring at the trestles during depropping correlated very well with the values calculated in advance.

During the assembly and production of the smooth aluminium cladding (Figure 17), various watering tests were run at different points to ensure the impermeability of the upper surface.

During lifetime regular inspections relying on the German DIN 1076 and VDI-guideline 6200 will be carried out in the future. Individual panels of the roof cladding will be used as entry points for this purpose.

4 Summary

The new Zayed University Campus in Abu Dhabi is a successful example of

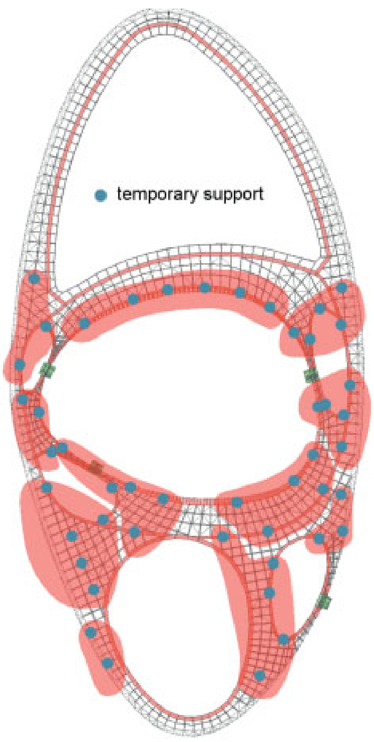


Fig. 14. Trestle groups for simultaneous depropping

Fig. 15. Mock-up for steel structure and cladding

Fig. 16. Prefabrication of welded segments in the shop (left: compression ring, right: crossing point tension ring)

Fig. 17. Finished roof in the campus area

international cooperation between designers and builders from multiple continents, demonstrates the potential of the main contractor philosophy and showcases the benefits from BIM-equivalent design approaches [6]. Apart from the extreme time constraints, the project required mastery of numerous engineering challenges. It required an understanding of different design and construction philosophies, management of difficult interfaces between design and construction works and consideration of the demand of a construction site with up to 8,000 workers running in parallel to the design works. Together with the employees and students of Zayed University, the main contractor and the architects, we are happy that our shared goal could be achieved in such a short time. We would like to thank Mubadala under the patronage of H.H. Sheikh Nahyan Bin Mubarak Al Nahyan, UAE Minister of Higher Education and Scientific Research and President of the Zayed University, for trust and confidence they extended to the whole team.

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Selected Project Participants

Owner:
Mubadala, Abu Dhabi, UAE
Client/Main Contractor:
Al Habtoor & Murray Roberts JV, Abu Dhabi, UAE
Architect:
BRT Engineering (Bothe Richter Teherani), Hamburg, Germany
Roof: up to design development, Buildings: up to design development and interior design
Pascall + Watson, Abu Dhabi, UAE
Buildings: detailed documentation
Structural Engineer:
Consulting Engineers Dr. Binnewies, Hamburg, Germany
Roof: full design service, Buildings: up to design development
Robert Bird Group, Dubai, UAE
Roof: peer review of concept design/temporary support design,
Buildings: detailed documentation
Project Control:
Parsons, Abu Dhabi, UAE
Steelwork Contractor:
Cleveland Bridge & Engineering Middle East, Dubai, UAE
Roof Cladding:
CNYD Shenyang Yuanda Aluminium, Dubai, UAE
Wind Assessment:
IFI, Aachen, Germany

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Figure 3: IFI

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Authors:
Böttcher, Christian, Dr.-Ing. and IWE,
Managing Partner
Frenz, Matthias, Dr.-Ing., Managing Partner,
former project director
Henning Kaufmann, Dr.-Ing.,
Team leader of software development
All: Consulting Engineers Dr. Binnewies,
Ingenieurgesellschaft mbH, Hamburg
Alsterterrasse 10a, 20354 Hamburg
www.dr-ing-binnewies.de
mail@dr-ing-binnewies.de
+49(0)40-415200-0