Design of a 120 m high GFRP Landmark tower

Liesbeth Tromp, Ernst Klamer, Kees van IJselmuijden, Ton Boeters.

Royal HaskoningDHV, Rotterdam, the Netherlands

Contact: Liesbeth.Tromp@rhdhv.com

Abstract

As an entrance gate to Jeddah (Saudi Arabia), at the new separated junction on the King Abdul Aziz Square, an architectural landmark structure has been designed with a record height of 120 m. The structure consists of two tall curved pylons at each side of the road that are connected at approximately 20 m from the top. The cross section has an airfoil shape and is designed as a stiffened shell structure. Glass Fiber Reinforced Polymer (GFRP) is used as the main construction material. GFRP is a durable and sustainable construction material. Using GFRP the weight of the towers is minimised, saving substantial material and costs in the foundation. More importantly, using GFRP rather than steel, the maintenance of the structure is minimized, thus preventing hindrance of traffic. In this paper the reference design of the Landmark is described and choices that have been made in the design and the design method are explained.

Keywords: Glass Fiber Reinforced Polymer (GFRP), tower, low maintenance, CFD, design, lightweight engineering, structural analyses, earth quake.

1 Introduction

As an entrance gate to Jeddah (Saudi Arabia), at the new separated junction on King Abdul Aziz Square, an architectural Landmark structure has been designed with a record height of 120 m. The curved shape of the Landmark is emphasized by illuminating the structure during the night, see Figure 1. The structure consists of two tall, curved pylons at each side of the road that are connected at approximately 20 m from the top. The pylons have an airfoil-like cross section and are leaning backwards in the direction of the road. They resemble pylons of a cable stay bridge, but have no supporting function for the Balanced Cantilever Bridge. The pylons are each connected to bridge deck by seven cables designed to have minimum force interaction with either structure.

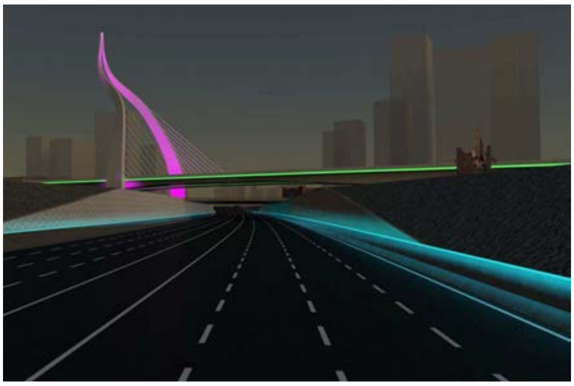

Figure 1. Visualisation Landmark tower by night

The Landmark tower is located in an earthquake sensitive area.

The cross section is designed as a stiffened shell structure. Finite element analysis have been carried out to determine the dimensions and the behaviour under, among others, thermal loads, wind- and earth quake loads, based on analyses including buckling, Eigenfrequency analyses and local plate buckling analyses. In this paper it is demonstrated that the design is feasible. The reference design of the Landmark tower is described and the design approach is explained.

2 Material and structure

The structure has been designed by RoyalHaskoningDHV architect Mari Baauw. He was inspired by the Islamic tradition in Jeddah and the importance of the site as an entrance gate to Jeddah and Saudi Arabia, see Figure 2.

Figure 2. Visualisation Landmark tower, balanced cantilever bridge and underpass (design Royal HaskoningDHV).

2.1 Trade off materials

Materials considered for the structure were concrete, steel and Fiber Reinforced Polymer (FRP). Main aspects for the trade-off were quality, weight, maintenance, ease of installation. Costs were considered a secondary criterion, but it is understood that a good score on these parameters are very determining for the costs of the structure.

The design consists of two leaning towers. Since the cables are non-structural, the foundation of the towers must stabilise the loads of the tower. In the SBC301 tensile forces on the pile foundation are not allowed. The weight of the structure is therefore an important parameter.

Concrete will result in a heavy structure, which requires a heavy foundation and a very complex installation process. Steel will result in a much lighter structure but the expected associated maintenance costs and economic damage due to traffic hindrance is a great disadvantage, and expected to be more severe because of wear of the coating due to sand storms.

FRP is lightweight, durable, chemically resistant and low maintenance and was therefore selected. Glass Fiber Reinforced Polymer was used in the further elaboration of the design. Carbon Fiber Reinforced Polymer (CFRP) stiffeners have been considered in case additional stiffness was needed, but it was found that sufficient stiffness could be realised using glass fiber reinforcement. Furthermore it is expected that the difference in thermal expansion between CFRP and GFRP might cause durability problems especially due to the cyclic nature of the thermal loads. The foot of the tower (lower 5 m) and the foundation are made out of concrete.

2.2 Structural principle

The two pylons are connected at the top and form a portal structure, see Figure 3. The GFRP tower pylons are connected to a moment resistant concrete foundation.

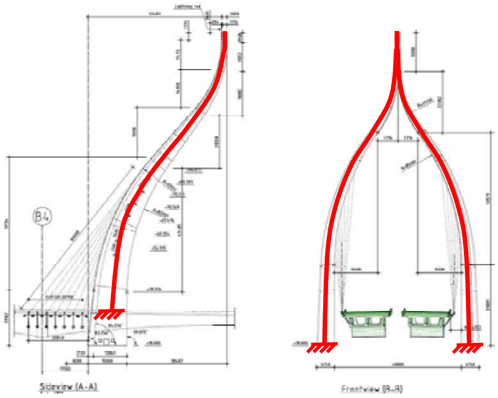

Figure 3. Structural principle

2.2.1 Cross section

The cross section of the towers is depicted in Figure 4. The shape was defined by the architect, but initial calculations on a simplified structure were used to determine the scale. In the materialisation phase the shape and taper of the cross section were further adjusted to fit the structural and functional needs. The cross section evolved from a rectangle into an airfoil shape, making use of the freedom in form provided by the selected material.

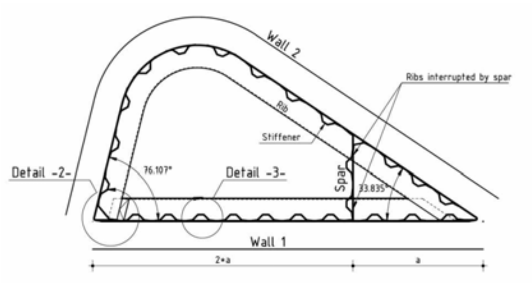

Figure 4. Cross section of the tower

At the base the cross section of the Landmark is 6,75 m x 15 m, at the top 0,9 m x 2,0 m. Dimensions at the top were determined by minimum dimensions for human access and ventilation. The section is stiffened with ribs perpendicular to the direction of the axis of the tower (c.t.c. 2000 mm) and vertical stiffeners (approx. 1500 mm c.t.c.). The skin of the shell and the stiffener consist of an anisotropic GFRP laminate with 62,5% of the fibers in the direction of the tower axis (vertical sweep).

2.2.2 Connection principle

For the manufacturing process of the GFRP pylons vacuum infusion is selected. Technically it is possible to produce 120 m elements, but handling or transport might require divisions. In case the 120 m pylon elements are produced in sections, these parts must be connected by means of laminated or infused scarfed connections, with a minimum slope to create a near continuous structure and avoid eccentricities in the section. The vertical stiffeners can be infused integrally with the shell or laminated in a second step. The horizontal ribs are designed as prefabricated parts that are laminated to the shell by hand lamination.

The connection of Wall 1 to Wall 2 (see Figure 4) is achieved by adhesion.

The connections of the tower to the other tower and of the tower to the foundation have been designed as bolted connections, see Figure 5. Due to the high forces and fatigue loading, at the location of the bolted connections the thickness of the laminate had to be increased to the maximum. Design guidance [3] provides an indicative strength allowable of 100 MPa for the design of bolted connections on fatigue. The applied dimensions of the steel bushes (100mm diameter), bolts (M40, M48) and plate thickness of the FRP (t = 100 mm) are indicative and must be further analysed. In the final design stage the design must be further developed based on strength data derived from tests.

The laminate thickness at the root of the tower at the connections is high, which might induce problems with internal stresses and quality of the laminate, such as fiber misalignment. If possible, in the final design phase the thickness should be reduced to minimise risk on laminate defects. However, the thickness of 100 mm is comparable to dimensions used in wind turbine blades and is therefore considered feasible.

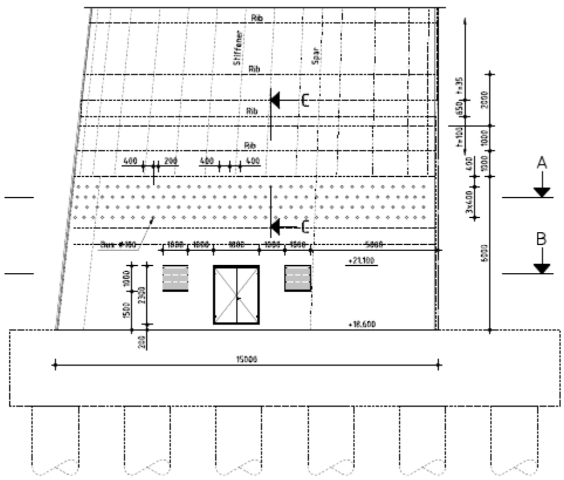

Figure 5. Principle of the connection of the GFRP tower to the concrete lower base and foundation

2.3 Design Standards

The concrete elements have been elaborated according to the Saudi Building Code parts 301 [1], 302 and 303. For the FRP structures use has been made of the Eurocomp Design Code [2]. This code provides guidance on partial material factors for GFRP and rules for the design of connections.

2.4 Structural Analyses

Sofistik version 14.10-30 is used for the analysis of the FRP landmark structure and the concrete foundation. This software is based on the Finite Element Method. The software is used for the analysis of the stresses (ULS) and deformations (SLS) of the landmark structure, including stability analysis. Autodesk Simulation CFD 2015 has been used for the CFD simulations of the wind load.

2.4.1 Wind load and CFD

Wind load has been determined in accordance with SBC301 [1]. Due to the complex section and tapering, standard wind form factors do not apply. The curved rounded corner will generate lift. SBC301 [1] prescribes that wind tunnel tests must be executed and this must be done in the detailed design phase. In the preliminary design stage use is made of Computational Fluid Dynamics (CFD) analysis that is used to derive the resulting wind pressure distribution on the structure. CFD analysis is an in-house expertise and a cost and time efficient way of obtaining the loads due to wind. Also it was used to investigate if there is a risk of undesired vibrations due to flow separation.

From this pressure distribution the wind load is derived for the dimensioning of the Landmark reference design. Four wind directions have been investigated at 0° (frontal), 210°, 270° and 350°, with angles relative to the road axis.

From the wind pressure plot (Figure 6) it is seen that the wind pressure increases at the top of the tower. Where the two towers connect the flow is further accelerated in the narrow opening. So even though the cross section at the top is smaller, wind loads are high over the full height of the structure, causing large bending moments at the foot of the tower.

From the air velocity plot (Figure 7) the acceleration of the flow at the corners of the structure can be seen. Also it can be seen that there are no irregularities in the flow.

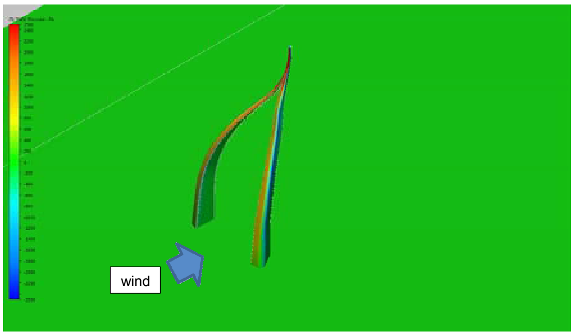

Figure 6. Example of wind pressure plot

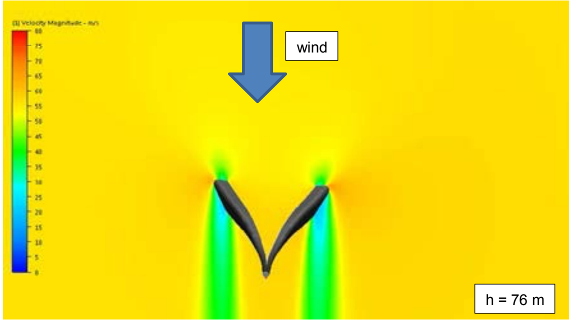

Figure 7. Example of air velocity at 76m, wind at 0°

In the pressure plots the red indicates pressure, the blue indicates suction. It is seen that the curved shape results in an additional lift force. i.e. transverse to the wind direction.

The wind pressures over the height and contour of the towers were tabulated for each element of the CFD model. This was done for each wind direction and translated into average pressures to create the pressure distributions to be applied in the structural analysis of the Landmark structure.

2.4.2 Earthquake load

Earthquake load has been applied in accordance with the SBC301 Site Class C. The principle design response spectrum is seen in Figure 8.

The seismic loads are assumed to act in any lateral direction. For each Eigenfrequency the response is calculated and they are combined with the complete quadratic combination (CQC) method. The analysis includes 35 modes to obtain a combined modal mass participation of 90%.

$$S_a = S_{DS}\,(0.4 + 0.6\,\frac{T}{T_o})$$

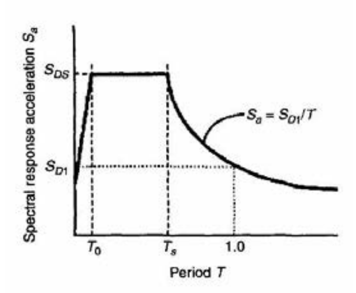

FIGURE 9.4.5
DESIGN RESPONSE SPECTRUM

Figure 8. Design Response Spectrum SBC301

For site class C:

$S_{DS} = 2/3 \cdot F_a \cdot S_s = 2/3 \cdot \mathrm{l}.20 \cdot 0.30g$ = **0.24g**
$S_{Dl} = 2/3 \cdot F_v \cdot S_l = 2/3 \cdot 1.69 \cdot 0.109g$ = **0.123g**
$A_s = 0.4 \cdot S_{DS} = 0.4 \cdot 0.24g$ = **0.096g**

2.4.3 FEA model and analyses

The Landmark structure is analysed with the FEM program Sofistik, using a 3D-model with 2D curved shell elements and 1D beam elements. Two analyses have been carried out: one with a global model of the entire structure and one with a local model of the stiffened plate. The aim of the global model is to determine the global response (deformation, reaction forces, Eigenfrequency, etc.), while the local model is used to determine the local response (strength verification, local plate buckling and effects of imperfections).

In the FEM analyses all materials used are of type anisotropic GFRP625, a balanced, symmetric lay up of glass fiber reinforced polyester with fibers distribution of [62,5%/12,5%/12,5%/12,5%] in orientations [0°/90°/45°/-45°], and a fiber volume percentage of 50%. The strongest and stiffest direction of the anisotropic material is in line with the local x-axis of the element. The main material properties are given inTable 1.

Table 1. Material properties GFRP625, Vf=50%

E1 [N/mm^2]	E2 [N/mm^2]]	G12 N/mm^2]]	α_1 [1/K]
28660	14919	4957	$1.7 \cdot 10^{-5}$

In the next design phase lay ups will be further optimised.

The global model and the local model are seen in Figure 9 and Figure 11.

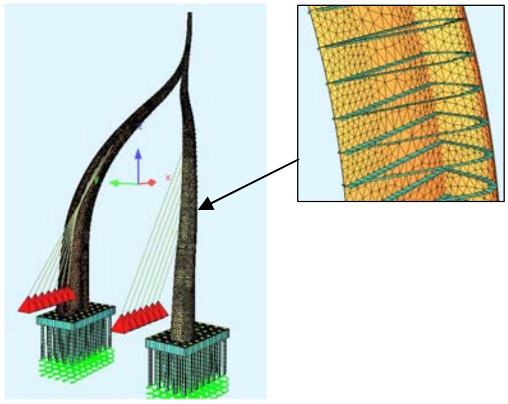

Figure 9. Global model with detail

For the global analysis the stiffened plate is modelled as a shell with geometrically orthotropic properties, which takes into account the effect of the vertical stiffeners. To be able to get both the axial stiffness in x-direction (EA) and the bending stiffness (EI) correct, an equivalent thickness and Young's modulus have been determined which result in the same axial stiffness EA and EI of the stiffened plate.

The horizontal ribs have been modelled as eccentric, curved 1D beam elements. Two different cross sections have been used in the global model: a large rib for the lowest part of the landmark and a smaller rib in the upper part.

Figure 10 shows the 3D and 2D representation of the Sofistik FEM-model of the foundation. The foundation is modelled as a concrete 2D flat plate (t = 3000 mm). The top of the piles is connected to the plate with a rigid link (yellow elements in right hand side of Figure 10), with an infinite stiffness over the height of the slab.

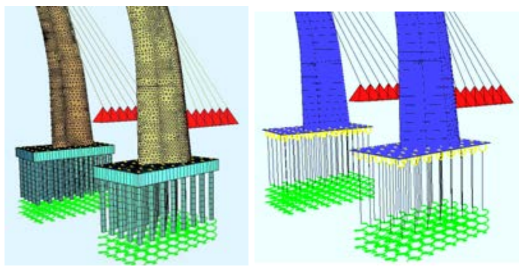

Figure 10 Pile elements and concrete slab

The piles have a diameter of 1200 mm and are supported with a vertical spring (green in Figure 10) and a horizontal bedding based on data from geotechnical investigation.

The local model is seen in Figure 11.

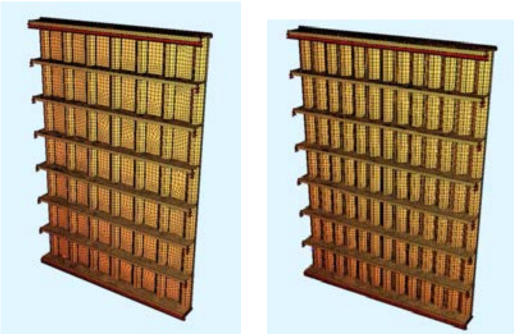

Figure 11 local models; general section (left), corner section (right)

The local model is used to calculate the structure's local response. This model simulates a part of the total structure and is built up with the horizontal and vertical stiffeners as shell elements. The purpose of the local model is to verify the strength and the local buckling behaviour, including effects of curvature due to temperature variation over the thickness, wind and imperfections.

An eccentricity is applied by means of a curvature. The curvature is set to 49.98/km, which is an equivalent of 4 mm initial deflection per 800 mm (L/200). The eccentricity is applied in 4 different configurations:

- Single curvature over the complete model
- Changing curvatures over the height. Each 2.0 m in height a different direction of curvature, with transition point at the ribs
- Changing curvatures over the width, where the curvatures change at each leg of a stiffener.
- Chess board curvatures, which is a combination of changing curvatures over the height and width, as described above.

In the structure near the corners of the cross section, the resulting forces are higher. Therefore stiffeners had to be more closely spaced to each other to withstand the higher loads

The vertical stiffeners are u-shaped. The thickness of the stiffener is 30 mm, the plate thickness is 35 mm. The horizontal ribs at the top and the bottom are L-shaped, 800 mm wide and have a flange of 160 mm height. The material thickness of the horizontal ribs is 30 mm. For the analysis, the top and bottom field of 2.0 m are modelled with twice the thickness for each plate, to prevent them from buckling in these fields. These fields are affected by the boundary conditions. In this way, buckling will occur in one of the inner fields and not directly next to the supports.

2.4.4 FEA results global model

Deformations

An important design criteria, besides strength, stability and reliability, is the deformation of the tower. The tower must appear rigid under normal wind conditions and may not interfere with traffic or cause feelings of discomfort at strong winds. Two deformation limits have been defined and verified for the towers and the cables:

- Clearance: No interference with traffic due to deformation of the tower and cable at maximum wind load including effects of ageing, without load factor (SLS).
- Sense of insecurity: Horizontal deformation of any point of the tower due to 60% of the maximum wind load (SLS), including effects of ageing ≤ L/250 with L = system length
  u ≤ 120000 / 250 = 480 mm

Besides stiffening of the towers, the clearance has been achieved by means of tensioning of the cables and by positioning the lower attachment point of the cables away from the deck of the balanced cantilever bridge. See Figure 12.

Figure 12 Steel cable support structure

The deformations in the direction of the road axis (u_x) under 60% of the wind load, including the SLS material factor of 1,3 for ageing effects is shown in Figure 13.The maximum deformation is 460 mm.

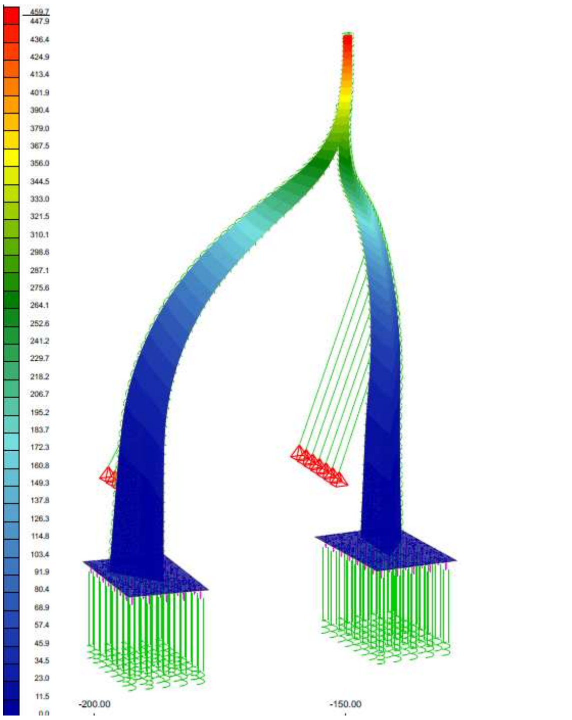

Figure 13. Displacement u_x at 60% of max. wind

The deformation of the structure under wind load perpendicular to the road (u_y) is shown in Figure 14. The maximum deformation at approximately 2/3 of the height is 405 mm.

It is important that the traffic is not distracted, experiences discomfort or feels unsafe by movements of the tower. It is seen that the deformations at the lower base are limited and that the deformations are high above the traffic.

Because this evaluation is made based on loads derived from CFD-analysis and the detailed design must be based on wind tunnel tests, the deformations are a point of attention, especially for the cables. If wind tunnel tests indicate higher wind loads, it is necessary to increase tensioning in the cable.

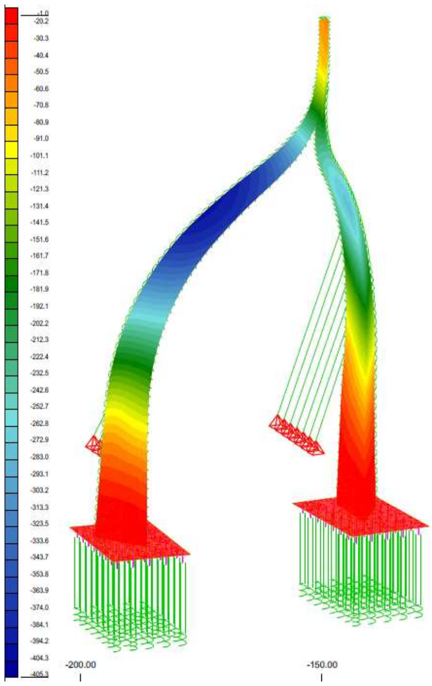

Figure 14. Displacement u_y at 60% of max. wind

Strength and stability

The resistance of the structure is verified with the local FEM model. Forces from the global model will be used as input. The results from the global model are presented in Figure 15. The results represent the maximum forces based on all occurring loads and load combinations in the ULS (envelope). The design determining result is the normal force in the local x-direction (in line with the tower axis -vertical sweep).

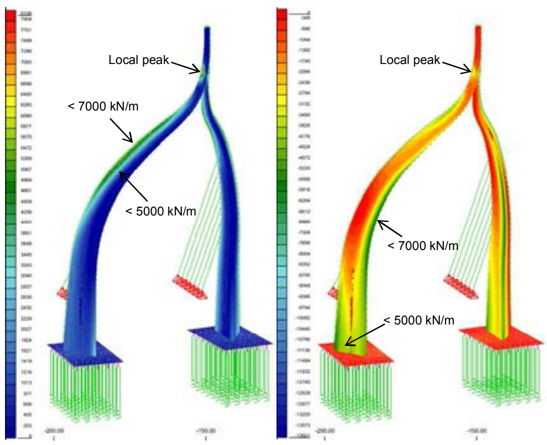

Figure 15. Maximum tensile (left) and compression forces (right) N_{xx} [kN/m]

Buckling evaluation showed that all first 35 buckling modes are local buckling modes of the plate between the ribs. No global buckling was found in the first 35 buckling modes.

2.4.5 FEA results local model

The main forces acting on the panels are the forces as derived from the global model. The normal force in the local x-direction (N_{XX}) is distributed over the plates and the vertical stiffeners. The in-plane shear force (N_{XY}) is mainly taken by the plate and partly by the vertical stiffeners. The other loads are acting out of plane. In plane axial stresses are found to be not critical with reserves of more than 30% below design strength.

Local plate buckling

The design determining criterion is local plate buckling. In the buckling analysis of the global section model it is clearly seen that local buckling of the plate between the ribs and web of the stiffeners is governing, as shown in Figure 16 (left). The local buckling is a combination between shear buckling and compression buckling. It was found that the eccentricity had some influence on the buckling capacity, the wind and temperature load have much less effect on the buckling capacity. The higher buckling modes in the first model and all buckling modes in the second model show global buckling modes of the panel, see Figure 16 (right). The width of the local model is equal to the maximum width measured from the corner to the spar and is thus representative for the structure.

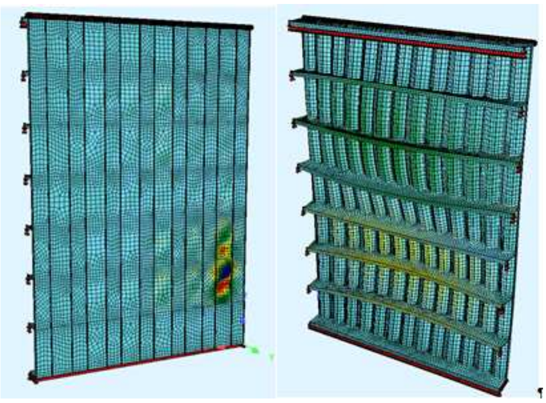

Figure 16. First buckling mode global section (model 1, left) and corner section (model 2,right)

The resulting safety margin for buckling is 1.06 for model 1 and 1.16 for the second model.

The stability of the stiffened skin is the design determining parameter. The evaluation is based on the most unfavorable configuration and the maximum loads. Because of the tapering of the landmark, the panel width in most of the tower is smaller and thus more favorable.

In the detailed analysis further optimization is possible by reducing the stiffener material or increase stiffener spacing in lower loaded sections. Furthermore it is important that the imperfection assumed in the stability analysis is taken as an important geometric tolerance criterion for acceptance of the structure in the realisation phase.

3 Conclusions

It was demonstrated that a 120 m Landmark tower can be built using GFRP composites. CFD analysis has been used to determine the wind loads. All strength, stiffness and stability criteria have been fulfilled in the design presented. The design determining parameters are deformation of the towers and local plate buckling.

4 References

[1] SBC 301 Saudi Building Code 2007 Structural – Loading and forces

[2] John L. Clarke, Structural Design of Polymer Composites; *Eurocomp Design Code and Handbook*, May 1996

[3] Germanischer Lloyd Rules and Regulations, *Rules and Guidelines IV Part 2 Guideline for the Certification of Offshore Wind Turbines*, 2012