Aspire Tower, Doha, Qatar

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At 300m, Aspire Tower is currently the tallest building in Qatar, its design symbolizing a hand grasping the torch for the December 2006 Asian Games.

Introduction
The 15th Asian Games, at the Sports City athletic complex, Doha, Qatar, on 1-15 December 2006, was the largest yet in the event’s 55-year history. Its centrepiece structure was the Aspire Tower, shaped to represent a colossal torch, which for the duration of the Games held its symbolic flame within the lattice shell that forms the topmost section.

Arup became design team leader for Aspire Tower (at the time named Sports City Tower) in January 2005, on the basis that it would be designed and built in 21 months for the Games. Although the original joint venture partnership was replaced in April 2005, excavation for foundations had already started the previous month. Piling began in May, and the raft foundation works by August. This posed a considerable challenge to the Arup team, as it meant that the design of the substructure, superstructure, and building services had to proceed in parallel.

As had been the case with the nearby Khalifa Stadium, also engineered by Arup\(^1\), the design and construction team was international. The main contractor was the same joint venture between Belgian and Qatari companies, Midmac-Six Construct, that built the stadium. The architectural concept chosen by the client was developed by the Qatari architect Hadi Simaan, the executive architect, Arep, is based in Paris, and the interior designer, Ecart, is also French. Arup was the structural, mechanical, electrical, fire, acoustics, lift, and wind consultant, working with Belgian façade designers and contractors and both Italian and Belgian steel manufacturers. The design was developed under the watchful eye of the Sport City Projects Director and his team, whose aspiration was for this to be a unique landmark building. To make this a reality required dedicated effort and technical excellence by the whole team in close collaboration with the client.

As well as functioning as a support for the Asian Games flame, the tower includes a large reception and public area on two floors for 3000 guests; restaurants and business centre; 17 floors of five-star hotel accommodation; a sports museum on three floors; a health club on three floors with a cantilevered swimming pool 80m above ground; presidential suites; and a revolving restaurant and observation deck about 240m above ground. A 62m high lattice shell structure on top of the reinforced concrete central core frames the 15MW flame cauldron.

2. The completed Aspire Tower, prior to the Asian Games. Part of the Khalifa Stadium can be seen on the left.
Building summary

The concrete core has a minimum external diameter of 13m, a maximum diameter of 18m, and walls 1.2m thick. It reaches 238m above ground and is surmounted by a cone that supports the flame cauldron, shrouded by the lattice shell structure. The core forms the building’s backbone, and supports the clusters (modules) of accommodation floors and the external envelope. Each cluster cantilevers up to 11.3m from the core independently, with no columns to the ground. The health club’s cantilevered swimming pool at the 19th floor extends beyond the floor plate by over 12m.

The building envelope wraps around the core to achieve maximum efficiency, and rises as a sheer structure clad in an energy-efficient outer glass skin, with environmental systems that achieve comfort levels in the occupied spaces even when outside temperatures exceed 40°C. High-speed lifts shuttle guests to the observation deck, bar, and revolving restaurant. In addition, 140 tonnes of tuned mass damper (TMD) at the top of the tower ensure comfort to patrons enjoying Arabic hospitality in the revolving restaurant, against the dry, hot Khamsin winds.

The tower is designed to carry over 71,000 tonnes of load, and is supported by a 37.3m diameter raft up to 7m thick, working in conjunction with 77 straight shafted piles.

5. Aspire Tower at night, showing the cantilevered swimming pool.
Foundations

A geotechnical desk study, which included information from the Khalifa Stadium, indicated sound limestone near to ground level, so the initial design was based on a raft foundation. While this progressed, a site investigation was made of the deeper geology and rock types - particularly important in view of the magnitude and concentration of load arising from the building form - and this confirmed the presence of weak Rus Chalk between the limestone strata, with its potential for voiding and relatively poor rock quality. In view of this and the assessment of soil/rock stress in this stratum based on the raft design already under way, the team decided that piling to the raft was needed to ensure adequate load capacity. Bored cast in situ piles were used, limited to a maximum 1.2m diameter to suit local practice, and extending through the Rus Chalk to the better quality limestone below. The piles were cast in grade C32/40 concrete for the necessary design strength.

The 37.3m raft diameter was determined by the need to spread the entire tower load delivered by the core, so as to limit bearing pressures to appropriate levels under the raft, in the Rus Chalk, and on the piles. In addition to gravity loads, the raft provides resistance to overturning effects under lateral wind and seismic loads. The arrangement provided also ensures that there is no tension in the piles or uplift, as the tension effects of overturning are balanced against the vertical load from the tower’s self-weight.

The raft thickness was derived on the basis of the same loading parameters, with the central area 7m thick to deal with the total core load, and a reduced thickness of 4m towards the perimeter where load is shed into the ground.

Piled raft analysis

A family of analysis models was used to determine the force effects for the raft structural design, with the Oasys interactive soil/structure program, GSRaft, being employed to analyze the foundation system.

This uses an iterative analysis system between two models, one for structure and one for soil. The structure model comprises a GSA grillage model (Fig 7) supported on springs at the underside of the raft and also at pile positions at the depth of the pile load transfer length. The soil model comprises a representation of the soil strata, with loaded areas corresponding to spring location in the structure model. At each stage of iteration soil settlements are calculated by another Oasys program, Vdisp, for the spring loads in the structure model. The settlements are then used to reset the spring stiffnesses in the structure model. The process is continued until vertical movements and load distributions match, providing a unique analysis for each load condition.

To assess local effects such as load spread beneath the core walls, the through thickness flexibility of the raft over an annular sector was modelled axisymmetrically, using loads and spring stiffnesses output by GSRaft. A lower/upper bound design approach allowed for the range of parameters involved - soil, concrete, design loads, etc.
The core
The core's internal diameter at the base is 14m, reducing to 11m at the restaurant level. The full core terminates at the viewing gallery level, above which a concrete frame transfers lateral and vertical loads from the steel lattice shell back to the top of the core. This was analyzed for serviceability and strength under vertical and wind loading by a full dynamic method based on wind engineering data from the advisory body ESDU (originally Engineering Sciences Data Unit). The Ritz method was used for the modal analysis to calculate the natural frequencies. The along-wind response of the tower was critical, as the crosswind effects due to vortex shedding are reduced by the presence of permeable mesh cladding rather than solid perimeter cladding.

A seismic analysis based on the 1997 Uniform Building Code, Zone 1, showed that seismic loading was not critical to the stability design, and so this was determined by the wind loading.

The vertical reinforcement in the core walls, designed in accordance with British Standard BS8110, generally varies between 0.4-0.9% of the section area. This was based on the tower's aerodynamic properties investigated through wind tunnel testing. The following effects were considered as the design progressed:

- moments induced due to local bending arising from steps in the core profile
- radial loads/moments from transfer systems to hotel, museum, health club, and restaurant
- anchoring the steel lattice for the top of the building
- restraint to the perimeter cladding system
- openings in the core and coupling beam arrangements.

The predicted peak displacement at the restaurant level from a 50-year wind was 454mm, or the height of the core from foundation level divided by 472. The predicted building accelerations from wind with return periods between one and 100 years are not within “acceptable” limits for 0.7% damping, which was assessed as the natural damping inherent within the building’s structural stability system. A TMD was therefore proposed to achieve a total of some 2% damping and reduce accelerations to acceptable levels.

Superstructure floors
The floors that cantilever out from the central core comprise steel beams supporting concrete slabs acting compositely with metal decking. The general arrangement has the primary beams spanning radially between steel columns and the core, with circumferential secondary beams. Steel columns in each module are supported by transfer arrangements cantilevering from the core. The presidential apartments, museum, and restaurant floors are supported off the core by steel cantilever brackets at the base of each accommodation block; these brackets also support the external cladding.

The lower viewing platform floor is similarly supported, while the upper viewing platform floor is in reinforced concrete cantilevering directly from the top of the core. The hotel, by contrast, is supported off the core by a system of vertical trusses located within the partition walls between the hotel rooms. The inner lines of the vertical trusses are in turn supported at their bases by a reinforced concrete corbel.
ring to the core located below level 4 in the hotel lobby area. The vertical trusses are typically between levels 5 and 10. An additional level of wall bracing (between levels 10 and 11) was needed under the lift lobby and the swimming pool in order to deal with the greater loads at these locations.

The tower is entirely clad in stainless steel mesh, including the "voids" between the accommodation modules, so as to provide a unifying surface for the entire building. The mesh acts in catenary and is prestressed within individual frames that span vertically between horizontal ring trusses at approximately 8m vertical spacing. The cladding is horizontally restrained either directly by the structural floors, or by an arrangement of struts connected to the floors. In the areas between floors, the cladding is restrained by an arrangement of struts connected directly to the core.

The weight of the perimeter cladding is supported by the same transfer structures that support the main floors. The cladding is both bottom supported and top hung, with horizontal movement joints provided between modules of cladding to accommodate differential vertical movement.
Swimming pool

The swimming pool is elliptical on plan: 11m long, 6m wide, about 50m² in area, and 1.5m deep. It is a reinforced concrete box with 0.3m thick walls, supported on a substantial steel truss structure some 4m deep, to correspond to the storey height containing the pool. The plan geometry of the truss is straight from the core to the column locations, and then elliptical with approximately the same shape as the pool. The truss is connected to the core and supported on two columns that continue down through the levels below, which are supported on the vertical trusses previously described.

As the pool and deck area extend about 12m from the tower perimeter, the support positions for the truss are as close to the perimeter as possible to minimize the distance to be cantilevered and maximize the extension. The truss system was designed to allow the pool itself to extend 8m from the support columns with the deck area protruding a further 4m.

The steel structure was erected first, to provide primary support and a frame within which the concrete for the pool itself could be cast in situ. The steel structure and supports were designed to resist lateral loads and therefore required bracing to transfer loads back to the core. The pool weighs about 300 tonnes in total, of which the steel supporting structure represents just above 10% (approximately 35 tonnes).

Internal core

Inside the central core are access stairs, lift shafts, landings, and service risers (Fig 18). The primary internal walls are of reinforced concrete, with reinforced concrete stairs cast in situ. The design allowed for open-mesh access floors to the services risers, and additional secondary support steelwork was required to support lift guide rails, etc.
Connections to the core

The Arup team discussed with the contractor how the steel beams should be connected to the core, which was slipformed throughout its height, including the corbel section at low level and the frame at the viewing gallery level. Steel plates were cast into the wall at floor beam locations and anchored back into the body of the core. These embedded plates were surveyed and connections for the floor and transfer beams were welded on site.

Damping

Assessment of the building’s performance indicated that additional damping would be needed to reduce lateral accelerations at the top under wind loading and so improve comfort levels. A feasibility study of various options for a total of 2% critical damping indicated that a TMD directly below the highest viewing level was the most practical solution for the range of predicted frequencies, and so a 140 tonne (active mass) folded-pendulum TMD with a steel mass was installed within the tower core. Fine tuning to the tower’s measured natural frequency was achieved by adjusting the pendulum lengths. The “fold” in the pendulum reduced the height of the required envelope by about half, but needed a rigid frame to transfer the tension between the first and second stage members. Shaping the TMD mass in this way facilitated fitting the damper within the circular plan shape of the core (Fig 20).

The detailed design, manufacture, testing and tuning of the TMD container was undertaken by a specialist contractor. Energy dissipation of the TMD is achieved by “pot dampers” which also incorporate bumper stops to prevent excess movement of the mass in extreme situations.

Cladding support systems

As already noted, the whole tower is clad in stainless steel mesh of varying permeability, apart from the bottom few metres. The hotel lobby, up to some 63m, is fully glazed within the outer mesh surface; this glazing is subject to wind load, and has significant thermal performance requirements as the upper levels of the hotel lobby get hot. Apart from the mesh and the hotel lobby glazing, the rest of the cladding is fairly conventional. It typically spans floor-to-floor (typically 4.05m apart), either directly or with supporting mullions.

JAP, the Belgian cladding supplier, proposed supporting the lobby mesh cladding and glazing in panels typically 8.1m high (ie two hotel storey heights), with six panels for each 20° of circumference giving a total of 108 panels on plan. The even number of subdivisions gives greater planning flexibility as it allows the use of the half grid. Above the lobby, the number of mesh panels was reduced to three for each 20° segment, ie 54 in plan for the total circumference. In the hotel lobby zone the external mesh and the glass are separated by about 1m. JAP proposed to link the frames carrying the mesh and glass to form a truss supporting a maintenance walkway. To reduce the number of struts here, the truss was designed to span one ninth of the circumference. Where the large cladding panels or the lobby trusses connect to the struts, movement joints are provided to avoid thermal stresses.
The top of the building

Structural elements

The 62m high steel diagrid frame tops out at 300m above the reference external ground level. The diagrid forms the lateral stability system for this part of the building, and also supports the cladding down to the restaurant floors. The diagrid springs from a substantial concrete frame: a 1m wide and 1.5m deep circumferential ring beam supported by nine concrete columns each approximately 1m by 1.5m arranged radially on top of the concrete core wall. The top of the concrete core wall is approximately 233.3m above lobby level, the top of the frame some 4.5m above that.

The primary loadbearing elements are the circular hollow sections (CHS) that form the diagrid shell, which vary from 610mm diameter near the base to 457mm diameter at the top. The shell is restrained laterally by a series of horizontal trusses outside the “petal” at 8.1m vertical centres, spaced to coincide with the cladding system’s horizontal support elements and sized to accept horizontal loads from it. The upper levels of trusses are in 500mm x 200mm rectangular hollow sections (RHS) and 300mm x 300mm square hollow sections (SHS), while the lower levels have 300mm x 300mm SHS.

The vertical loads in the outer plane of the structure, just inside the cladding line, are carried by 18 hangers, 120mm x 120mm SHS, equally spaced around the building perimeter. The outer booms of the trusses span between them for vertical load, whilst they restrain the truss booms against buckling out of plane. At the head of the outer plane there is an “eaves trimmer” - the rim of the “petal” - formed by 610mm diameter CHS. It carries the vertical load in the hangers, spanning between the heads of the diagrid elements and acting in bi-axial bending to resolve the forces in the diagrid elements that do not node out regularly at the ring. In addition the eaves trimmer carries wind load and/or vertical load from the cladding connected directly to it.

The diagrid is a tall, relatively light structure that is subject to significant lateral loads. As a result, substantial tension forces are generated within it under some load situations. The steel structure is well able to deal with these, but this behaviour also gave rise to the potential for physical uplift and considerable movement of the base of the shell from the supporting structure. To cater for this effect, the base of the lattice shell is attached to the top of the core by clusters of vertical prestressed bars extending down into the body of the core. These clusters consist of either four or six prestressed bars, each stressed up to 3200kN, and anchoring 300mm thick baseplates of the lattice shell nodes to the supporting concrete. Installing these bars and the associated anchor plates, ducts, and anti-bursting reinforcement was a considerable challenge for the contractor.

Structural action

Vertical loads

At the diagrid rim, the hanger loads are collected by the eaves trimmer. This is curved in plan and elevation, so there is no direct line-up between the hangers and the diagrid members. The eaves trimmer must therefore carry torsion as well as bending moments in two directions, shear in two directions, and axial force. The splice connections in the eaves trimmer are bolted connections offset from the diagrid members. The diagrid CHSs then carry the loads in compression to the head of the core where they are resisted by the bearing of the connection nodes onto a grout layer on the head of the core walls.

The building’s asymmetrical shape means that the cladding and self-weight loads on the high side are significantly greater than on the low side. In addition to the overall compression, this generates an overall bending moment in the diagrid shell system, carried in the same way as the moments generated by wind load in the wide direction.

Horizontal loads applied in the wide direction

This is the critical direction for wind loading because (a) the widest face area is exposed to the wind, and (b) the structural depth available to resist the loads is at a minimum.

The wind applies pressure to the cladding system, which spans 8.1m vertically between horizontal trusses. The horizontal trusses collect and redistribute the horizontal component of the wind load and transfer it to the petal diagrid, which resists by push/pull action in the inclined CHS members and transfers it down to the connection at the head of the core. Each level of horizontal truss contributes to the push/pull in the diagrid CHSs and so the magnitude of the forces in the diagrid increases down the structure until it reaches a maximum in the level immediately above the head of the core. The forces distribute themselves elastically through the grid structure so that there are compressions on the downwind side of the shell structure, tensions on the upwind side, and opposing pairs of compressions and tensions in between.

This is analogous to a vertical cantilevering action in an idealized beam element.

Axial shortening of the compression elements and extension of the tension elements lead to an overall downwind deflection of the top of the structure - its most significant deflection mode. Elements were sized to limit this deflection and keep its effect on the racking of the cladding panels within reasonable limits.

Horizontal loads in the narrow direction

The structure works similarly for narrow direction wind but, as the loaded profile is narrower, the overall loads on the diagrid are less. Also, the width of diagrid perpendicular to the load direction is greater, so the push/pull action resists loads more efficiently. However the eccentricity of the centre of action of the load does give rise to an overall torsion in the diagrid in addition to the overall bending. This is resisted by the diagrid just as it resists the other shear forces, by push/pull action in the diagonal CHSs.

24. The lattice diagrid, August 2006.

23. Structural elements of the diagrid frame and b) ring beam support for diagrid.
Wind loading and wind tunnel testing

Meteorological data on wind speeds enabled an analysis of extreme winds to be undertaken. This analysis indicated a gust reference design wind speed of 38m/sec to CP3: Chapter V: Part 2\(^5\), which was the reference code specified by the client. This analysis was accepted as the design basis for the tower and allowed a reduction to the normal reference design wind speed in Doha.

Wind tunnel studies were undertaken at a facility run by the specialist company BMT Fluid Mechanics. These provided aerodynamic design data on which to base computation of the tower’s response, as well as assessing the potential of a vortex shedding resonant response.

Overall aerodynamic coefficients were derived from wind tunnel testing, and were used to review the values adopted in the initial design stage. This permitted a more accurate assessment of overall structural loads and accelerations at the top occupied level. Forces were measured on 1:100 scale sectional models of the top of the tower and a middle section (these being most relevant to the overall behaviour of the tower). A high-frequency force balance at the base of the models was used to measure forces on the model. Modelling of the surface mesh was particularly important, and full size samples were tested together with a scale representation of the mesh so as to provide appropriate representation on the wind tunnel model. Aerodynamic coefficients were derived from the measured force values and the geometry of the tower.

CAD

The tower superstructure was modelled in 3-D using Tekla Structures software. This provided valuable co-ordination and representation of complex elements, including the diagrid, where 2-D CAD would clearly have been inadequate.

Building services design

To design and build the 300m tower in 21 months was a huge challenge. Qatar’s desert climate, with temperatures as high as 50°C in summer, plus Doha’s high humidity levels, make it vital that the plant and equipment perform to their design parameters without failing. The building envelope has to achieve maximum thermal efficiency and comfort levels in the occupied spaces whilst giving guests panoramic views of the city. Detailed studies were carried out, including building physics with simulation software, to select the best glazing and mesh properties to reduce overall cooling load and energy consumption.

The MEP systems were designed to relevant international standards but incorporate Qatar codes and regulations. The main considerations in Arup’s design of the systems included comfort, reliability, life safety, energy efficiency, space for plant, and speed of construction.

The tower’s estimated electrical demand was 7MVA with a peak cooling load of 7MW; two independent 11kV power supplies are connected to the network and operate simultaneously to share the load. Each, however, can supply the full load if the other fails, to give high resilient power supply for life safety and hotel operations. 11kV switchrooms at the basement and revolving restaurant levels act as nodes for the local 11kV distribution network.

A 350mm diameter chilled water connection links to the main energy centre with flow and return temperatures of 6.5°C and 14.5°C respectively, and a chilled water flow rate of 223 litres/sec. Stand-by generators serve the life safety equipment, security systems, commercial operations, and data/communication systems.

As the tower comprises discrete blocks of accommodation connected to the central core, the extent of the MEP services running within the core was limited so as to speed its construction and maximize the accommodation space. Only the main lifts and the chilled water, electrical, and water installations were located in the core to connect the health club, museum, and restaurant levels to the plant in the basement. Each of these includes air intake and exhaust points for the air-handling units (AHUs) serving each discrete block.

The chilled water system comprises series of sealed pressurized circuits, operating on variable flow to match the anticipated high diversity factor and reduce energy demand. The building is divided into four pressure zones: the lowest serving the basement only, the second up to health club level, the third up to museum level, and the fourth the revolving restaurant and observation deck.
To reduce pressure ratings, a plate heat exchanger at health club level serves the upper floors. All terminal units and AHUs were designed with two-port control valves to maintain the required enthalpies. Because of high ambient water temperatures, the swimming pool water is also cooled by plate heat exchangers to maintain a temperature of 30°C.

To predict air movement and comfort level, especially for the ground floor lobby and restaurant areas, a computational fluid dynamics (CFD) model, STAR-CD, was used to calculate the air temperature distribution and air movement in the atrium. The light ray tracing software Radiance was used to calculate the direct and diffuse solar radiation distribution as inputs to the CFD model. This included the complex transmission, absorption, and reflection properties of the external shading elements and glass façade combination.

Arup’s innovative approach to the mechanical design was to create a two-zone environment with air supply nozzles mounted around the inner core at level 4 and a series of binnacles in the lobby area. The low zone was predicted to be well mixed within the target air temperature range, whilst the high zone had stratified conditions - less critical as it was outside the occupied zones. The two-zone approach results in an effective distribution of temperatures.

Both the nozzle and binnacle supplies were optimized (throw angle, flow rate, supply temperature, location) to give an acceptable balance of air temperatures and speeds in the occupied zones, whilst the external mesh and façade glazing combined to reduce direct solar transmission to acceptable levels. The complex annular flow of high-level air between the unshaded and shaded sides of the atrium and its impact on the nozzle system was understood through this high resolution approach.

In addition, comfort levels (air/radiant temperature, air speeds) in the occupied zones could be assessed.

To minimize plantroom space and improve the efficiency of the mechanical systems, terminal cooling units and AHUs were selected to control cooling and dehumidification. The minimum ventilation rates were based on ASHRAE and CIBSE guide recommendations. Terminal units, especially for the triple-height ground floor lobby and restaurant areas, were selected to be integrated with the interior design, and cool only the occupied areas.

Heat recovery systems are used to recover coolth from the exhaust air to pre-cool the hot outside air entering the AHUs, all of which were designed for minimum fresh air to reduce the overall cooling load and energy consumption.

**Acoustic design**

Apart from the usual aspects of acoustic design, two were particularly interesting. As the tower is clad with mesh to visually alter the profile, its height and the wind climate indicated high levels of wind-generated noise. Once the wind magnitudes and frequencies were established, the mesh profile had to be adjusted for the presidential suite area. This was confirmed by the specialist cladding suppliers who tested the mesh at different velocities and frequencies using a wind turbine. Arup used 3-D animation to develop the acoustic analysis of the large atrium area at ground level, the results of which aided the architects and interior designers with their design and choice of materials.
Fire engineering

A highly fire-engineered approach was necessary for the tower’s fire safety design, making use of its architectural features to achieve the expected level of safety and at the same time minimize costs. In the absence of specific local codes for high-rise buildings in Doha, this fire strategy was based mainly on the approach described in various appropriate British Standards, chosen for their good guidance on tall buildings and provision for fire-fighting activities.

The fundamentals of the escape route design were largely determined by the cylindrical structural core design: two exits from each storey into opposite sides of the core, from a racetrack corridor on the hotel levels, served by two separate stairways in the core. The designed floor-to-floor height allowed the scissor stair arrangement. The very robust core wall provides a high degree of fire protection, and pressurization keeps smoke out of the escape stairways.

As the various accommodation sections are spaced up the tower, this separation between groups of floors enables staged evacuation in the event of fire, reducing disruption from unwanted fire alarms and making better use of the available stair capacity.

The automatic sprinkler installation was designed in accordance with NFPA14, for the protection of all areas, including fire hydrant system throughout the building in accordance with NFPA13, for a Class I standpipe system. Gaseous flooding systems were specified for the electrical rooms and data processing rooms. analogue addressable, intelligent fire detection systems are networked, together with “fire survival” bi-directional communications to a master control and monitoring panel. The fire alarm system operates on a two-stage principle and is linked with a voice-alarm system consisting of alert and evacuation alarms in public areas. Emergency lighting is provided along all escape routes and egress points.

Conclusion

All the main structural elements were completed within the 21 months specified for the tower’s design and construction, enabling it to fulfil its designated function for the 2006 Asian Games. Work on the interior spaces continued after the conclusion of the Games, and were completed during the first half of 2007. The Aspire Tower has proven to be a momentous project, of which the entire concept, design, and construction team is very proud.

References

(2) http://eom.springer.de/R/r082500.htm