DMCC Al Mas Tower - Structural Design

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ABSTRACT:

DMCC (Dubai Metal & Commodities Center) Al Mas Tower is a 360m high slender office tower located in the Jumeirah Lake Towers Development in Dubai, UAE. Significant structural design challenges were presented by the architectural form and the client's requirement for floor efficiencies of 80%.

This paper discusses the structural system adopted; wind tunnel testing done to assess structural loads, cladding pressures and building acceleration; assessment of short and long term lateral movements of the building arising from the asymmetric form; column shortening effects and resultant mitigating measures; construction aspects and the connection between outriggers and the peripheral columns; and vibration and fatigue assessment of the feature spire.

1 DESCRIPTION OF THE PROJECT

The tower comprises 5 basements, 3 podium levels, 60 commercial floors and 3 mechanical floors. The client is Dubai Multi Commodities Centre (DMCC). Figure 1 shows an artist's impression of the building. A typical tower floor plan is in the form of two diagonally offset ellipses, the maximum floor length of which is approximately 64 m compared to its width of 42 m (Figure 2). From level 53 to 64 the floor plan consists of only one of the two ellipses. Due to the iconic nature of the building, there is an 81 m slender spire at its top.

Construction of the structural frame is currently well under way.

Figure 1 –Artist’s impression

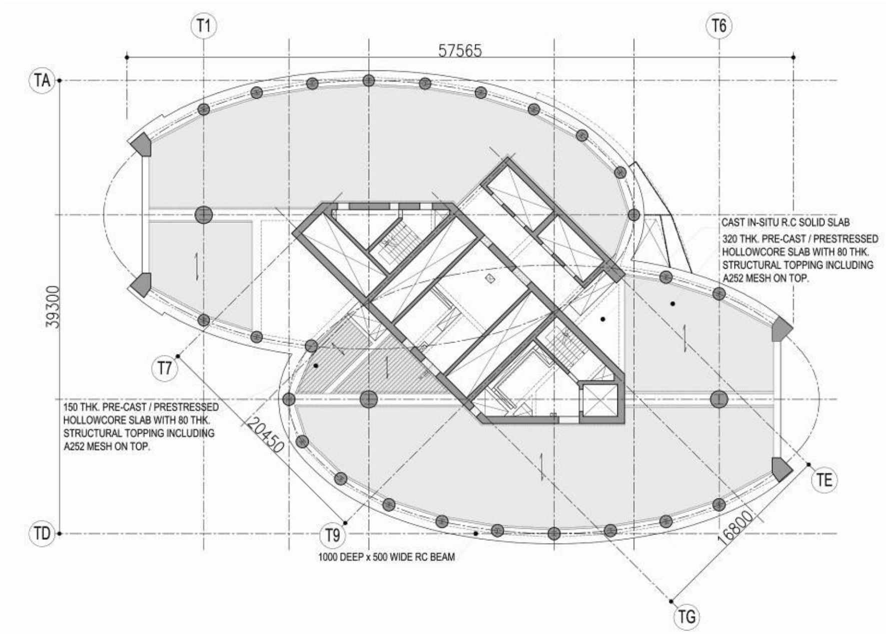

Figure 2 – Typical structural floor plan

The following constraints were considered at the concept stage design:

- An efficiency of not less than 80% for the office floors
- Flexible column/wall free office space
- Each office floor plate to be capable of supporting a safe weighing 2.5 tonnes at any location within each office.

2 STRUCTURAL SYSTEM

The main structural form consists of a reinforced concrete peripheral frame and a central core wall. These are connected to each other by central spine beams on each floor and outrigger walls at service floors.

The typical floor slab comprises 320 mm thick hollowcore slab with 80 mm thick structural topping (Figure 2). The floor is designed to act as a diaphragm to transfer lateral wind and seismic forces to the central core and peripheral frame. Precast slabs were chosen because they were quick to build, comparatively light-weight and provided uninterrupted space for services.

The peripheral frame comprises 1000 mm deep by 500 mm wide beams supporting the precast units, spanning on peripheral columns at a maximum spacing of 5 m. The columns are of composite construction (Figure 3) in the lower half of the building in order to minimise column size.

Three service floors are provided above the 9th, 28thand the 47th floors. Service floor slabs are 450 mm thick solid RC slabs with floating slabs provided below certain equipments to control vibration & noise. Ceiling slabs (except the uppermost 47th floor) are 400 mm thick solid RC so as to provide an acoustic barrier to the floor immediately above.

The design was performed according to British Standards, BS 8110 Part 1[1] & BS 5950 Part 1[2] generally and UBC-97[3] was used for seismic load assessment.

A range of concrete grades from 45 MPa to 70 MPa was used with Grade 460 reinforcement. Structural steelwork was S355 to BS EN 10025-2:2004 – “Hot rolled products of structural steel”, Part 2.

Lateral stability is provided by the reinforced concrete central core and external peripheral frame together with outrigger walls plus reinforced concrete peripheral belt walls to improve lateral stiffness.

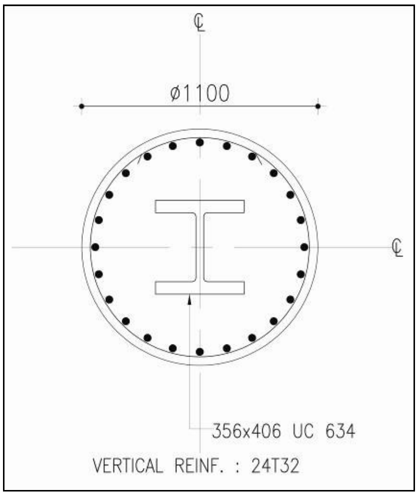

Figure 3 –Typical composite column detail

A three dimensional finite element analysis was carried out using ETABS. The model included raft slab and spring supports to simulate piles. Allowance was made in the section properties for cracking under ultimate limit state based on UBC-97. It was assumed that all loads will be transferred to the ground through the piles. Raft weight was not considered for the purpose of assessing seismic base shear. Spring stiffness for piles was based on the pile working load capacity and the theoretical settlement of the pile under that load. The effect of the podium on lateral movement was considered by modelling lateral springs at various levels based on the stiffness of the podium structure.
In order to study the efficiency of the peripheral frame, belt walls and outrigger walls, a parameter study was carried out (Table 1)

Parameters	**Core Wall**	**Core wall + peripheral frame**	**Core wall + peripheral frame + beltwalls + outriggers**
Natural Period (s)	14.6	12.2	9.6
50 yr wind sway (mm)	1785	1258	771

Table 1 – Parameter study on the effectiveness of the structural system

The belt walls include large openings to allow for air intake and discharge. The outrigger walls also include large service openings (Figure 4) to allow for movement of ductwork and piping.

The outrigger walls, if constructed along with the floors, would have transferred significant dead load from the peripheral columns onto the core. In addition, the outriggers would have attracted forces due to differential axial shortening between the core and the peripheral frame. To mitigate this effect the outrigger walls were not connected to the floor slab above and the peripheral frame until after all floors had been constructed.

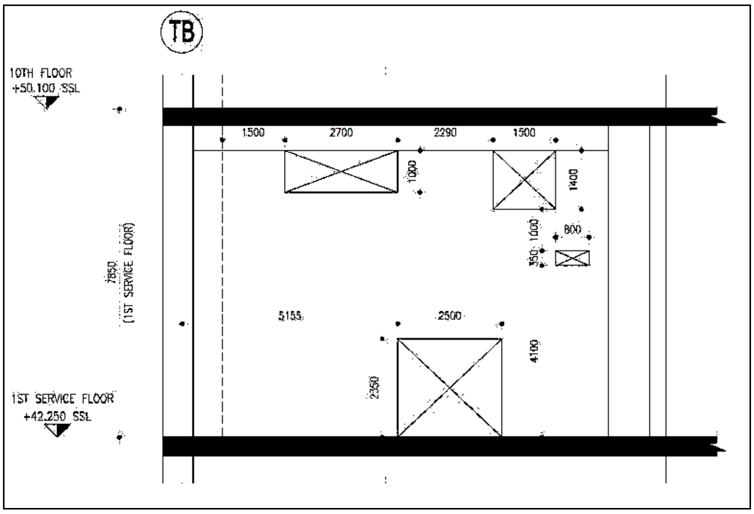

Figure 4 – Typical outrigger wall elevation

The deflection for a 50 year return wind was greater than H/500 (H = height of building) and, therefore, soft joints were provided between the blockwork walls and the structure (in accordance with BS 8110 Part 2[4]) to allow for racking movement between adjacent storeys under wind loads.

3 WIND ENGINEERING

Wind tunnel testing was carried out by RWDI, Guelph, Canada using a High Frequency Force Balance Model (Figure 5). Wind loads were based on a 3 second gust wind speed of 37.7 m/s for open terrain at 10 m height. This was based on wind loads measured at Dubai International Airport between 1983 and 1997. The proximity model was based on a radius of 575 m.

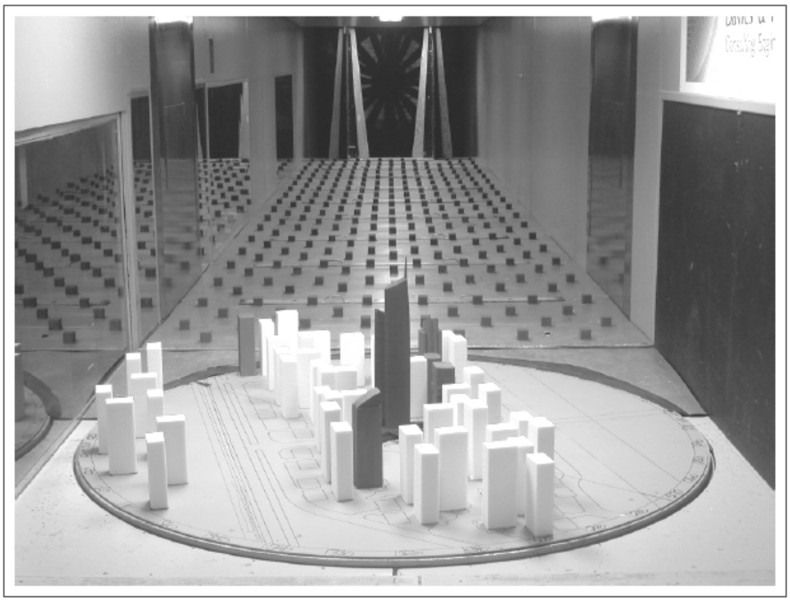

Figure 5 – Model of Al Mas Tower in Wind Tunnel

The model was placed on a turn-table and rotated at intervals of 15 degrees (24 wind directions). Structural properties such as mass, mass distribution, mode shapes and frequencies were obtained from the structural analysis model and input to assess overall structural loads, building acceleration and cladding pressures. A damping value of 2% was initially assumed for the calculations. RWDI provided overall structural loads taking into account directional effects for each sector.

The building's accelerations at the top floor for a 10 year return period was reported to be 18.7 mg which was within acceptable criteria of 23.4 mg (Figure 6). A further check for a 1.5% damping resulted in an acceleration of 21.6 mg (increased by a factor of $\sqrt{[2/1.5]}$) which was still within acceptable limits. The maximum localised cladding pressure was reported to be 4.5 MPa.

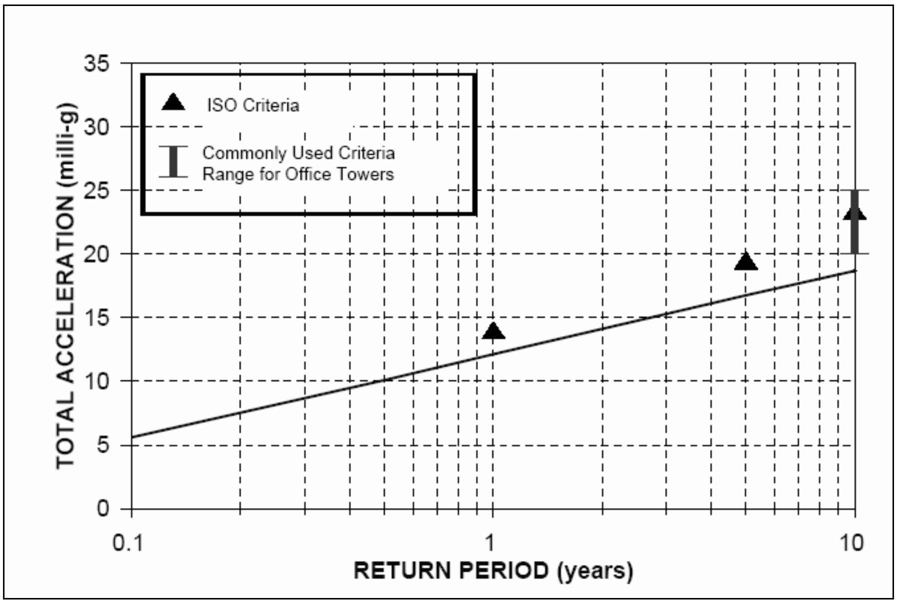

Figure 6 – Building acceleration results from wind tunnel testing

4 SEISMIC DESIGN

Seismic loads were based on Zone 2A of UBC-97 as per local authority requirements. Response Spectrum Analysis compliant with UBC-97 was carried out with appropriate scale factors to obtain member forces and associated drifts. Section modifiers as per UBC-97 were applied to the design i.e. 0.7 for uncracked walls and columns, 0.35 for cracked walls and 0.35 for beams. Ductile detailing for coupling beams with diagonal reinforcement was used as per UBC-97, although it is strictly not required for Zone 2A.

5 FOUNDATION SYSTEM

The foundation for the tower is a 3.0 m thick piled raft supported on 1200 mm diameter friction piles, approximately 40 m deep. Cement replacement using 50% GGBFS (Glass Granulated Blast Furnace Slag) was used to mitigate heat of hydration in the raft as well as to enhance durability of the raft.

The columns and walls in the podium area are supported by pile-caps resting on piles with slabs spanning between the pile-caps. Because of the high water-table, there is uplift in the podium basement slab and tension piles are employed to limit the slab thickness.

6 VERTICAL ASYMMETRY

The tower floor plan and the corresponding stiffness are such that the construction of the tower does not cause any lateral deflection up to level 53. However, as one portion of the tower rises a further 13 floors, the resulting eccentric load makes the lower part deflect laterally and thus pull the upper part with it. The theoretical maximum deflection under gravity load for an instantaneously applied load would be 225 mm at the highest floor level i.e. floor 64. However, there were a number of factors that had to be allowed for to estimate the maximum long term deflection.

1. Vertical elements built plum at each floor rather than following the slope of their supporting floor.
2. The actual strength and stiffness of the concrete.
3. The construction method and construction sequence.
4. Long term effects of creep (shrinkage does not normally add to this parameter).

There is no specific limit mentioned in the standards for the lateral deflection under gravity load. However, from an aesthetic viewpoint and to remove this as an issue for the secondary elements in the building a limit of H/1000, where H = height of building, was considered acceptable.

The main contractor was using a 'targeting' construction method to make sure that the key points in the core were constructed at specified coordinates. This meant that from level 53 upwards, whenever a deflection due to eccentric loading was introduced, an automatic correction was made in the alignment of the floor above. However, as the effect up to level 53 was cumulative it meant that by the time the construction reached its final level, floor 53 had moved sideways to its maximum and this would increase with time due to creep.

Calculations were carried out to estimate the lateral deflection at level 53 for the scenario where the final floor was constructed. This was relatively simple and was carried out with a single analysis by activating all the vertical permanent loads in the floors above level 53. The results were then corrected for the following effects:

1. Actual strength and stiffness of the concrete as measured by cube test results:

   The influence of the actual strength and elastic modulus of concrete was simply catered for by reducing the calculated lateral movement by about 30%.

2. The time dependent creep effect:

   The effect of the time dependent creep was allowed for by a reduction in the effective elastic modulus. Calculations were carried out based on the principle of area moment to work out a multiplier on the lateral movement. This resulted in a net increase of 110% on the calculated value of lateral movement due to the load from level 53 above.

3. Any additional structural alterations:

   The structure was also modified by increasing the thickness of certain walls in the core in order to reduce the eccentricity of the unbalanced loads hence reducing lateral movement. The effect of this was a reduction of 15%.

The above effects were combined to give a calculated final deflection value of 170 mm at level 53, which is equivalent to H/1350 (where H = height of building).

6.1 *Long Term Axial Deformation of Vertical Elements*

As for all tall buildings of this magnitude, it was necessary to estimate the axial deformation and differential shortening between the core and the columns as this would impact on the design of connecting elements. Adjustments were also required to make sure the floor levels were kept within acceptable tolerances.

Two approaches were considered for the calculation of creep and shrinkage effects. The first was based on the methods in EC2 – BS EN 1992-1-1[5] and the other was based on the formulae in the ACI [6].

The ACI approach considers the relaxation of creep due to the presence of reinforcement. This was a significant factor for this building given the high level of reinforcement in columns and walls. A computer program to calculate long term axial deformation was developed by Mehrkar using procedures published by Fintel, Khan and Iyengar[7].

Fintel's approach assesses the long term axial shortening of columns and walls by considering elastic shortening, creep and shrinkage and allowing for the fact that floors will be cast horizontally to the design level indicated on the drawings. The calculation was carried out for the core walls and columns. The difference in the shortening was then allowed for in the construction of the floors between the cores and the columns. A typical difference between the core walls and one column is shown in Figure 7.

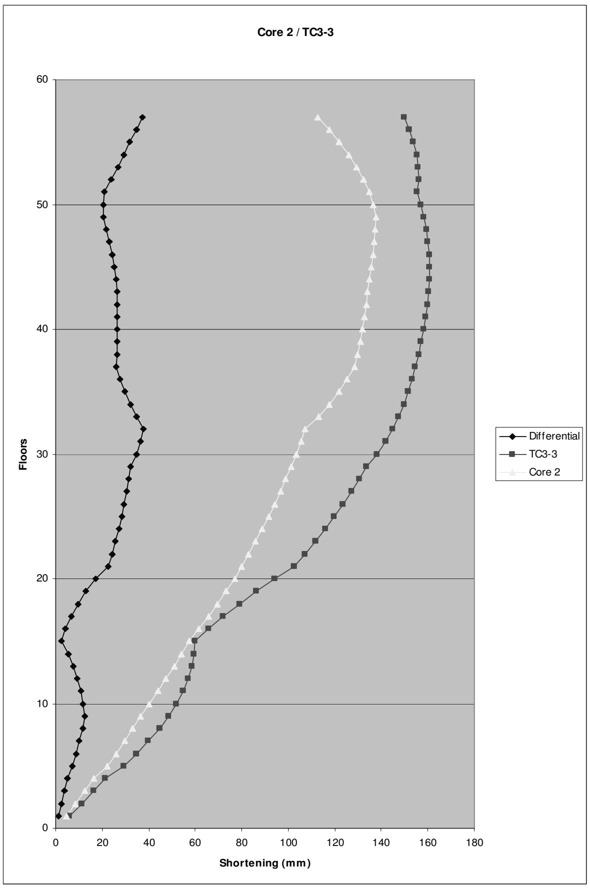

Figure 7 – Long term vertical shortening of core wall and a typical column

Survey points were located at each floor and were monitored (using a laser surveying instrument - Leica TPS700) for lateral drift against a fixed benchmark located at ground level outside the building. Points were also located at the core and periphery of each floor to monitor movement due to axial shortening.

7 CLADDING

The building façade consists of a unitised cladding and curtain walling system, which was manufactured as a complete module incorporating aluminium, glass and insulation.

The vertical spacing is based on the floor-to-floor height of 4.0m, while the horizontal spacing is based on a combination of the structural grid and the requirement to achieve a maximum number of vision bays within the office layout. A 10 mm horizontal gap is provided between each panel for thermal and seismic movement as well as the long term movement of the concrete frame due to creep and shrinkage. A 200 mm zone at the slab edges has been allowed for the cladding. The full depth of the mullions, including the glazing or cladding is 150mm. This allows for a 50mm structural tolerance of 25 mm either way.

8 PODIUM

The design of the podium was inspired by the inherent angular geometry of a diamond. The podium comprises an array of eight triangular glass petals that radiate from the central core (Figure 8). A Diamond Exchange Center is accommodated in the north-eastern petal that projects out over a terraced water feature stepping down to the lake. The three-storey podium accommodates a variety of retail spaces, food court, business centre, health club and the Diamond Exchange Club.

Each triangular retail petal comprises a steel framed structure with composite metal deck floor slabs. The Diamond Exchange Center sits on a profiled metal deck slab supported on a grid of steel beams. These are supported in turn by exposed steel trusses and steel columns which are vertical or raking (Figure 9). The steel frames are stabilised by portal action and by the central core of the building to which they are connected.

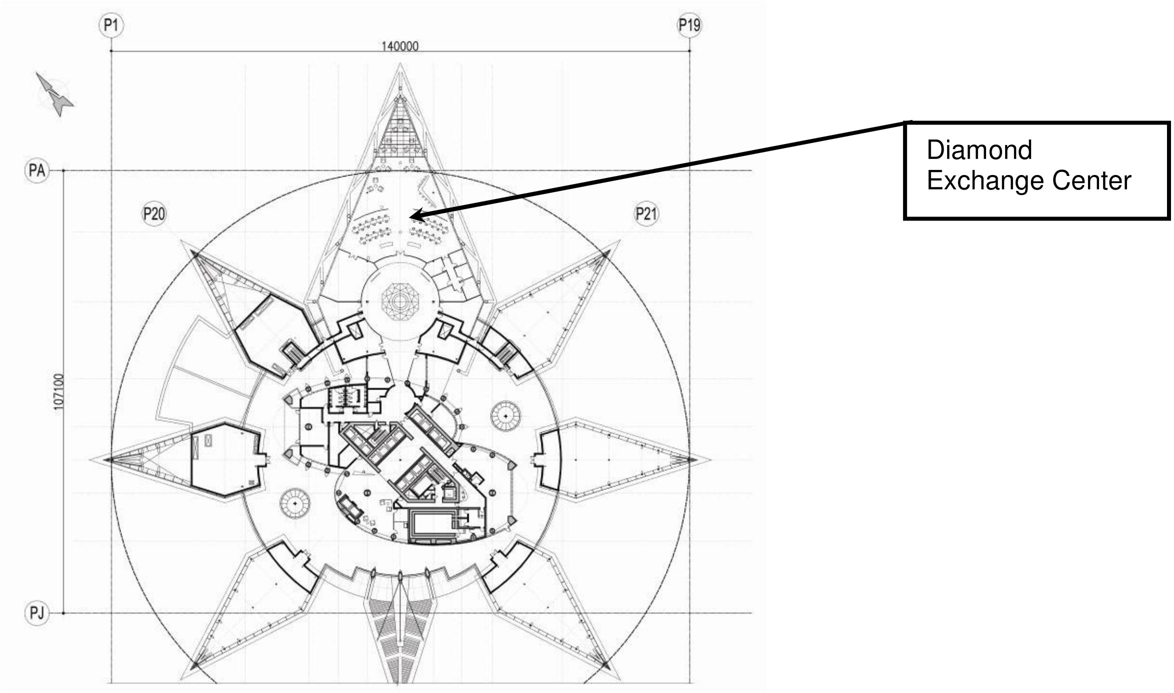

Figure 8 – Podium inspired by the shape of a diamond

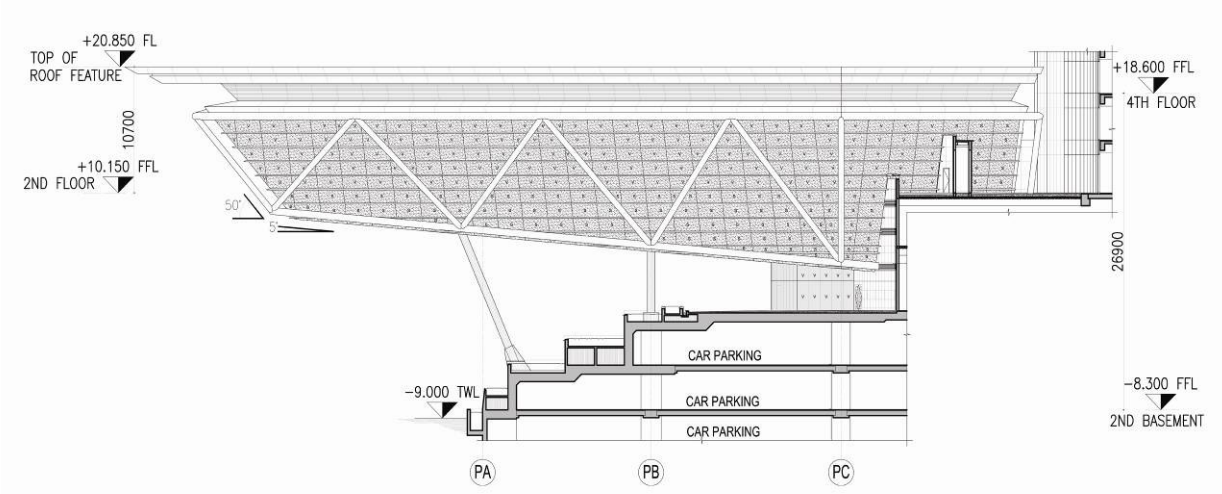

Figure 9 – Diamond Exchange Center Elevation

The ground floor slab is formed by a 140m diameter stepped floor radiating from the central tower to the perimeter. Close to the tower, the podium is at ground level, gradually stepping to basement-1 level towards the tower entrance and basement-2 towards the rear side.

The ground floor slab is a single unit 750mm thick folded concrete plate. All the basement columns outside the tower terminate at the ground floor slab. These columns form a regular grid measuring 17.5m x 8.0m in order to incorporate the Client's requirement of large area, column-free basement parking. The folded slab supports the landscape loading together with the planted columns from the retail units and the Diamond Exchange Center.

A 3D finite element model in ROBOT software was used for the analysis and design of the slab. No expansion joints are provided in the ground floor slab. The ground floor slab is exposed externally and, therefore, designed for constrained stresses due to seasonal temperature variations of $\pm 20^{o}$C. The temperature variation was applied to the model and its effects were combined with the gravity load effects. Podium columns as well as perimeter retaining walls were designed for the additional lateral loads due to thermal movements.

9 SPIRE

The top of the Al Mas Tower features an 81m tall spire reaching to a height of 360m (Figure 10). The base of the spire is connected to the tower through a reinforced concrete upstand wall over a length of 21m resulting in a free-standing length of spire of approximately 60m.

The spire has two distinct sections with a step transition at approximately two-thirds of its height (58m above base). The lower portion is roughly elliptical in shape and is constructed from a triangulated steel frame with aluminium cladding. The major axis of the ellipse is 7.4m at the base and the minor axis is 3.4m. The upper portion of the spire, which is 23m high, is of steel sheet construction. The major axis of the upper portion is 4.3m at its base and the minor axis is 1.5m.

9.1 *Method of Analysis*

Due to the slender nature of the spire, it was realised early on in the design that without additional damping, the spire would vibrate excessively causing unacceptable fatigue stresses leading to structural failure. It was therefore necessary to check the spire to ensure that the wind-induced dynamic fatigue stresses were kept within acceptable limits throughout the 50 year design life of the structure. This was achieved by the introduction of tuned mass dampers.

The spire structure was initially analysed and designed for code calculated static wind forces. The design was then checked for dynamic effects from wind oscillations.

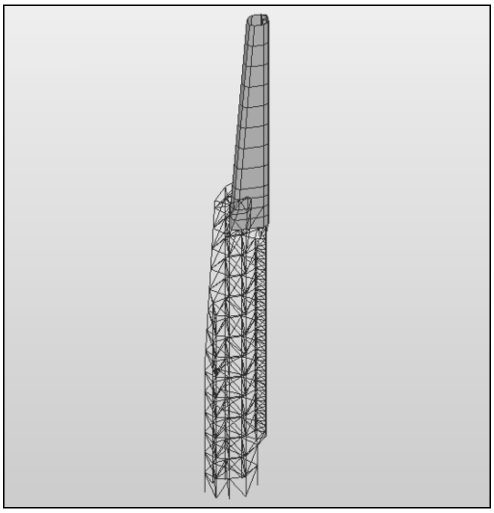

Figure 10 – Robot 3D model of Spire

9.2 *Assessment of vibration of fatigue and means of mitigation*

Analysis was done to assess the susceptibility of excitation of the spire from effects of galloping, flutter, wind turbulence and vortex shedding. It has been found that vortex shedding can produce significant fatigue stresses in the spire. Initial fatigue assessment calculations were performed using an intrinsic critical damping value of 0.5%. The calculations were re-run with increasing levels of damping. Values of 3% damping for the first mode and 2% for the second mode were found to be required to produce an acceptable fatigue life. Four 2 tonne TMDs, located close to the top of the spire were designed to provide the necessary damping requirements.

10 CONCLUSION

The building has been designed to meet international standards for strength as well as serviceability requirements such as occupancy comfort, overall deflection and story drift, which were of primary concern.

Lateral sway and column shortening calculations, which are normally complicated, time dependent and construction sequence dependent, were carried out by using simplified but relevant methods which led to traceable and de-coupled component of each movement.

Measures were taken during design & construction to address issues such as column shortening & dead load sway due to vertical asymmetry.

The construction is well underway & data is being collected to compare the actual movements with the theoretical movements. This obviously needs to be monitored over a relatively long period of time. The results will be published at a later date.

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