The Sail Tower, Haifa, Israel

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Introduction

The Government Office Tower is a part of the new urban development, with a total floor area of more than 150,000 m^2 presently under construction at the downtown of Haifa, near the Haifa Port.
The 37 stories Tower, with about 140 m above ground, was completed in February 2002.
The Tower consists of (Fig. 1):

- An underground floor containing offices archives.
- Two stories for commercial use.
- A double-height entrance floor at the level of the Main Plaza (+13.00) which is located on the southern side of the building, and connects it to the other buildings of the complex, such as the Treasury Building, the District Court Building and others.
- 25 office stories that are designated for the government departments. The departments which require easy access as they provide services directly to the public are located on the first six floors. The building's cafeteria and kitchen are located at the tower's mid-height -the 16^{th} floor.
- Floors 28 and 29 have been designated for the machinery room, water reservoir, transformer room, etc.
- Above the top technical floors five mock floors were built in order to complete the architectural look of the two double curved structures.

The Tower floor area is 850m^2 at the lower and upper floors of the building, and gradually increases up to 1250m^2 at the center floors. The floor-to-floor distance is 3.7m.

The development has three floors below the Plaza level, with about 150 parking spaces.

The Energy Center of the building is located within the basement floors, and attached to the parking floors. It includes air conditioning units, generators, transformers, a water reservoir, a diesel oil reservoir and more.

The underground area and the superstructure are made of reinforced concrete, cast in-situ.

The whole building, which is similar in appearance to a huge boat with inflated sails, is covered with curtain walls combining aluminum and glass. The Tower concrete columns are covered with aluminum cladding panels.

The aesthetics of the building is strongly enhanced by the structural solution, which was conceived by means of an intense architecture-structure interaction.

The total floor area of the project is approximately 37,000m^2.

The Structure of the Office Tower

Superstructure

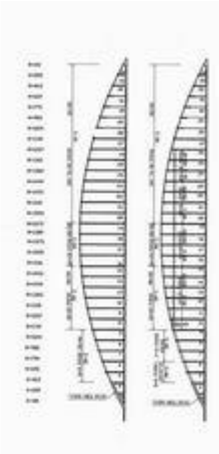
The structure of the Office Tower consists of a core and external façade columns. The main service core is located at the center of each floor, and is surrounded by four wings (Fig. 2).   Two opposite wings are shaped as a quarter of a cylinder with radius of 16.5m, and the other two are shaped as double-curved structure , with a radius of up to 16m, that looks like a quarter of a cigar.

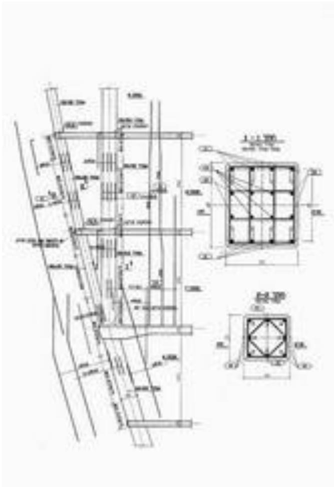
Flat slab & Columns

The Tower is destined for office use; therefore the floors are mostly columns free, with the exception of the two curved structures. The entire office area at the cylindrical wings is columns free, and is supported by 20÷23cm thick cast in-situ flat slab with spans varying from 6m to 9m. The flat slab at all the floors area is fixed to the core walls on one side and simply supported on the façade columns on the other side. A finite-elements analysis of the slab was performed in order to optimize the mesh & mild reinforcement layout.

The façade columns at the cylindrical wings (2 corner columns with fixed size 0.4m x 3.1m and 3 other columns 0.5m x 0.55m that are reduced up to 0.35m x 0.35m at the upper floors) are separated by 6.25m and connected by 0.6m x 0.2m flat beam.
At the spatial wings, the façade columns (2 corner columns with fixed sizes 0.4m x 0.6m and 4 other columns 0.65m x 0.68m that are reduced up to 0.35m x 0.35m at the upper floors) are separated by 5m- at the biggest floor (Fig. 3).

The radius of the spatial wings increases rapidly from floor to floor. At the 8th floor, when the floor radius becomes 11.3m, 2 more interior columns are added in order to reduce the slabs span , which would have increased up to 16m without them (Fig. 4). These columns start from the intersection point of the sloped columns, which follows the curved line of the building, and the round edge beam (Fig. 5).



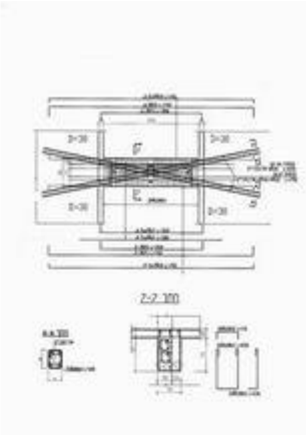


The façade columns that are located on the edge of the spatial wing floor are getting closer to each other as the floor radius reduced toward the lower and upper parts of the Tower. These columns are supported by a massive concrete base, shaped like a quarter of a cone, which is attached to the building's main core.
The design loads at the cylindrical and spatial wings columns are up to 6,000 kN and 11,000 kN respectively.

The absence of beams provides ample space for the office space (275cm) and for all the mechanical systems, running through the ceiling, with no structural interference. The lowered ceiling of each floor consists of 15cm for the acoustic ceiling, and 60cm under the concrete slab for air ducts, sprinklers, water and electrical systems and lighting fixtures.

The main core

The main service core, made of reinforced concrete (18.5m x 20m), is designed to withstand all the lateral forces due to earthquakes and wind. The core is formed mainly by combining the following: 4 elevator shafts, 2 secure staircases, a safe room (air raid protected, 22m^2), service elevator, 2 washrooms, main & service corridors, electrical and communication rooms, kitchenette, smoking rooms and vertical shafts for the building piping and ducts.
The main elevator corridor (300cm wide) and the service corridor (140cm wide), divide the core into four sections. These sections are connected together by four beams (0.5m x 0.9m and 0.5m x 1.2m) in each direction, which are designed to carry out the shear forces, and make these four sections work together as one big core. The reinforcement arrangement in those shear beams is based on two groups of rebars (connected together by very close ties) in each direction, forming a scissor shape, according to the Israeli Earthquake design regulations (Fig. 6, 7).



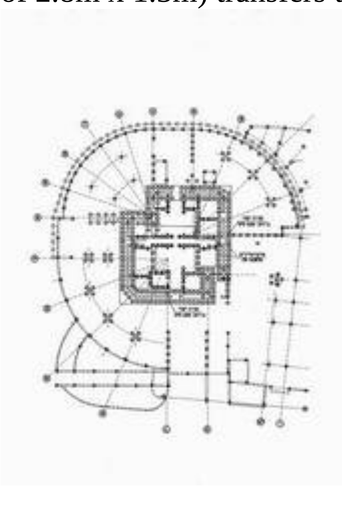
Due to the proximity of the Tower to a geological fault, which crosses at the Haifa bay vicinity, it was necessary to consider an extra cautionary measure and to double the seismic zone coefficient (Z=0.2*g instead of Z=0.1*g). As a result of this measure, the overturning moment due to the earthquake loadings was relatively big for a building of this size. As for the big bending moment, tension stresses at the outer walls of the core had to be considered, and alternately vertical reinforcement bars were used.

Concrete & Steel

The concrete that was used has characteristic strengths of 50 MPa for the columns and 40 MPa for the floor slab and core walls. These values are reduced at the upper floors columns and core walls to 40 MPa and 30 MPa, respectively. The yielding stresses are 500 MPa for the mesh steel, and 400 MPa for the mild steel.

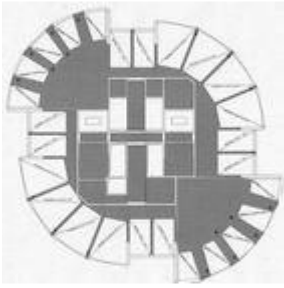
Foundations

The service vertical load of the core is 350,000 kN and the overturning moment due to the earthquake loadings is 1,420,000 kNM. Due to the fact that there was not enough mass to overcome the great overturning moment, the tension forces at the base of the building core were given to tension piles. The 60cm diameter tension piles, that penetrated the dolomite bedrock 10÷17m deep, were located below the perimeter wall of the core and arranged in pairs each 90cm. A continuous foundation beam (pile cap of 2.8m x 1.3m) transfers the vertical forces from the core to the piles (Fig.8).



Construction

A “Peri” flying formwork system was adopted at all the repetitive floor locations. At the irregular floor areas the contractor used the “Peri-Multiflex” formwork system (Fig. 9).



The forms were dismantled after 3 working days, when the concrete attained a strength of at least 25 Mpa, and re-shored again only by “Peri” supporting poles for 18 more days. A cycle of one 850÷1250m^2 floor per week allowed the structure to be completed about one year after the completion of the foundations.

Wind Loads

Only the main core resists the wind actions. This system proved to be quite effective, limiting the drift to 4cm (at a height 113m). The fundamental period of the structure is 2.5 s, and the maximum acceleration at the corner of the uppermost-occupied floor (at a height 113m) for a10-year return period is 7.8 mg.
Wind loading was determined by a model study performed at the Boundary Layer Wind Tunnel Laboratory of the University of Western Ontario, Canada. The Office Tower was modeled accurately using 1:300 scales, according to the detailed architectural drawings (Fig. 10).

Simultaneous time histories of the pressures at 300 points of the façade for 36 azimuths were recorded for one hour in the laboratory tunnel, which simulated a real wind gradient and turbulence. A detailed proximity model of the surrounding city built in block outline from Styrofoam for a radius of 370m. The upstream terrain was modeled coarsely using roughness representative of the actual site topography for each azimuth (Fig. 11).

Applying the time histories of the simultaneous pressures to a dynamic mathematical model that reproduces mass distribution and the first nine vibration modes then performed a random dynamic analysis of the structure. Means, root-mean-square deviations, and maximum probable responses in terms of displacements, accelerations

and base moments were obtained for all the azimuths for a return period of 50 years. A set of equivalent static lateral and torsion loads was also computed. A complete structural CAD model was then loaded with an appropriate number of combinations of these static loads.
The curved surface at the upper and lower part of the spatial wings of the Tower introduced significant horizontal thrusts in these floors. These thrusts are equilibrated by the overall structure stiffness through spatial action, and introduce a slight permanent horizontal displacement at the top.
The overall overturning moment due to the wind loading was 322,000 kNM and 343,000 kNM at X and Y direction respectively, and the torsion moment was 16,100 kNM. These values are given for return period of 100 years and damping value of 1% of critical.
The largest differential pressure and suction on the curtain walls were 1.5 kPa at the western façade and 1.8 kPa at the northern façade respectively.

Credits

Owner:
Joint Venture of: Ashmoret Tichona Ltd. – Israel
(Founded by: Ashtrom Properties Ltd. & Secom – Israel Ltd.)
Industrial Buildings Corporation Ltd. - Israel
Architects:
Dina Ammar-Avraham Curiel, Architects, Haifa, Israel
Yaakov Ben Avraham, Avigail Sachs (Arch. In charge)
Structural Design:
Eng. Rami Ballas
Project Management:
Eng. Yoram Ganor, Ashtrom Properties Ltd.
Project Site Manager:
Eng. Yossi Lazar, Ashtrom Group, Israel
General Contractor:
Partnership:
Ashtrom Engineering & Construction Ltd, Israel
Sollel Bone Ltd, Israel
Curtain Wall Contractor:
Seralwall – Italy

SEI Data Block

Project area (m^2):…..……...……..… 37,000
Concrete (m^3):……………....………22,500
Steel (ton):………...……… ………… 2,300
Total cost (USD Millions):…………...... 50
Construction dates: June 1999- February 2002

Date: 10.12.02

List of Drawings:

Fig. 1: The Sail Tower.

Fig. 2: Typical floor structure.

Fig. 3: The Eastern spatial wing at construction.

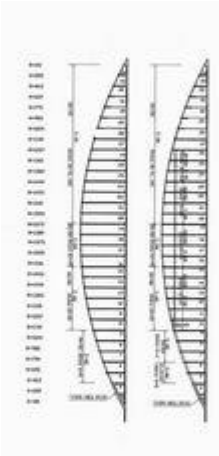


Fig. 4: The spatial wing columns layout.

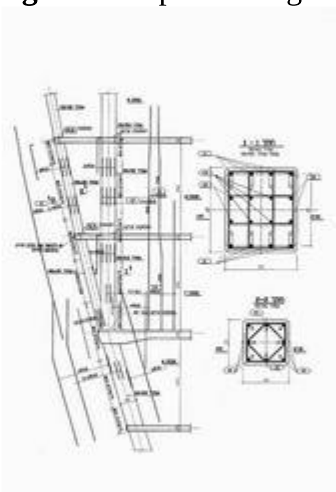


Fig. 5: The intersection point of the sloped columns, the rounded edge beam and the new interior column which is added at the 8^{th} floor.

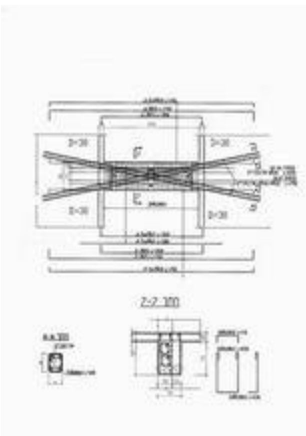


Fig. 6: The reinforcement detailing drawing of the core shear beams.

Fig. 7: The reinforcement arrangement of the core shear beams at construction.

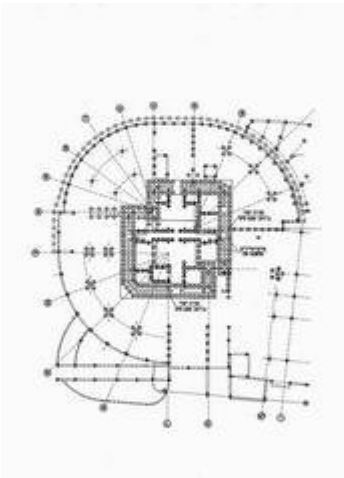


Fig. 8: The Tower's foundations layout.

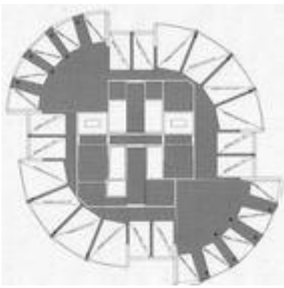


Fig. 9: The typical floor flying formwork system.

Fig. 10: The Tower pressure model for the wind studies (at 1:300 scales).

Fig. 11: The Tower and the detailed proximity models at the Boundary Layer Wind Tunnel Laboratory.