The Sail Tower, Haifa, Israel

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Introduction

The Government Office Tower is a part of the new urban development currently under construction in downtown Haifa, near Haifa Port. The structure is a 37-storey tower (approx. 140 m high) with a total floor area of more than 150 000 m^2. It was completed in February 2002.

The tower consists of:

- an underground floor containing office archives
- two stories for commercial use
- a double-height entrance floor at the level of the Main Plaza (+13,00 m) 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 floors of office space designated for government departments. Departments providing 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 16th 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 850 m^2 at the lower and upper floors of the building, and gradually increases up to 1250 m^2 at the center floors. The floor-to-floor distance is 3,7 m (*Fig. 1*).

The total floor area of the project is approx. 37 000 m^2. There are three floors below Plaza level, with about 150 parking spaces.

The building's energy center is located in the basement, and attached to the parking floors. It includes air conditioning units, generators, transformers, a water reservoir and a diesel oil reservoir. The underground area and the superstructure are made of cast in-situ reinforced concrete.

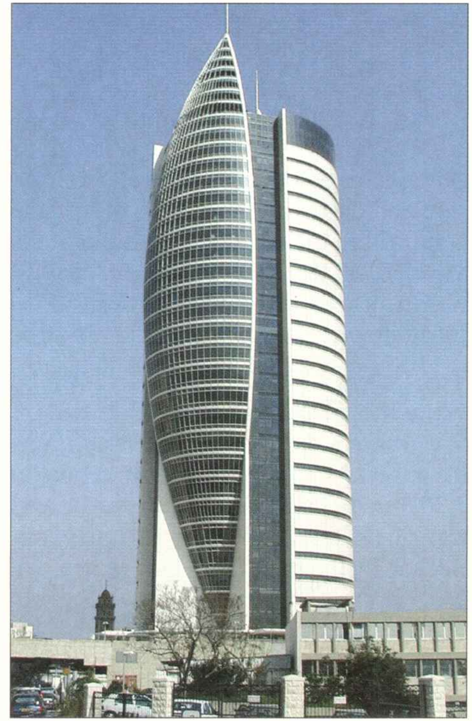

Fig. 1: The Sail Tower

The whole building, resembling a huge boat with inflated sails, is covered with curtain walls combining aluminum and glass. The tower's concrete columns are covered with aluminum cladding panels.

Office Tower Structure

Superstructure

The structure of this 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 a radius of 16,5 m, and the other two are shaped as double-curved structure, with a radius of up to 16 m, resembling a quarter of a cigar.

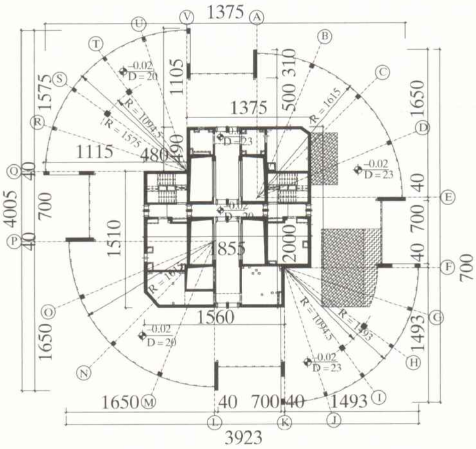

Fig. 2: Typical floor structure

Flat Slab and Columns

As the tower is intended for office use, the floors are mostly column-free, with the exception of the two double curved structures. The entire office area at the cylindrical wings is column-free, and is supported by 200 – 230 mm thick, cast in-situ flat slabs with spans varying from 6 m to 9 m. The flat slab on all floors is fixed to the core walls on one side and simply supported on the façade columns on the other. 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 walls with fixed size 0,4 m × 3,1 m and 3 other columns 0,5 m × 0,55 m that are reduced up to 0,35 m × 0,35 m at the upper floors) are separated by 6,25 m and are connected by 0,6 m × 0,2 m flat beams.

At the spatial wings, the façade columns (2 corner columns with fixed sizes 0,4 m × 0,6 m and 4 other columns 0,65 m × 0,68 m that are reduced up to 0,35 m × 0,35 m at the upper floors) are separated by 5 m at the largest floor (*Fig. 3*).

The radius of the spatial wings increases rapidly from floor to floor. At the

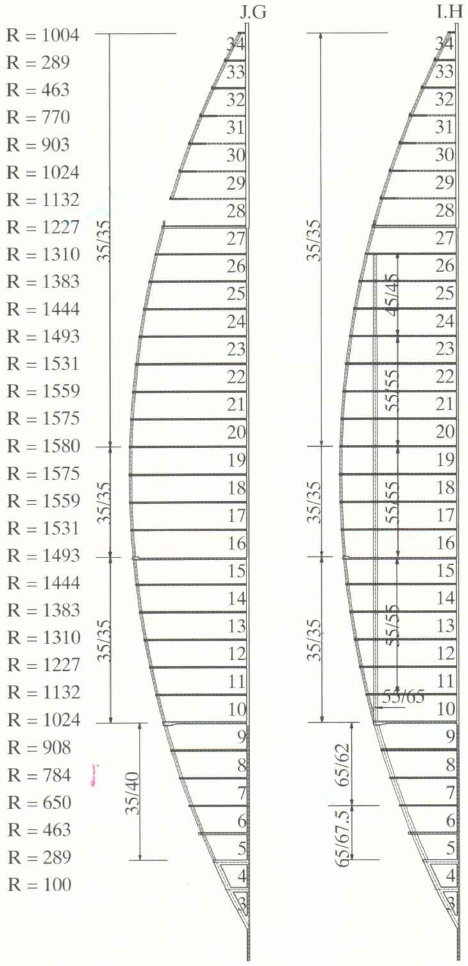

Fig. 3: Spatial wing column layout

8th floor, where the floor radius becomes 11,3 m, two more interior columns are added to reduce the slab's span, which would have increased up to 16 m without them. 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. 4*).

The façade columns located on the edge of the spatial wing floor get closer to each other as the floor radius reduces towards 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 6000 kN and 11 000 kN respectively.

The absence of beams provides ample space for office space (2750 mm) and for all the mechanical systems, running through the ceiling, with no structural interference. The lowered ceiling of each floor consists of 150 mm for the acoustic ceiling, and 600 mm under the concrete slab for air ducts, sprinklers, water and electrical systems and lighting fixtures.

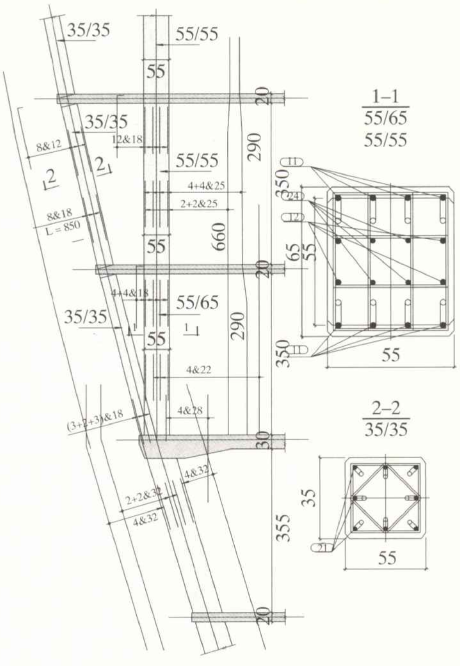

Fig. 4: Intersection point of the sloped columns, the rounded edge beam and the new interior column which is added at the 8th floor

Main Core

The main service core, constructed of reinforced concrete (18,5 m × 20 m), is designed to withstand all the lateral forces of earthquakes and wind. The core is formed mainly by combining the following: 4 elevator shafts, 2 secure staircases, a safe room (air raid protected, 22 m²), service elevator, 2 washrooms, main and service corridors, electrical and communication rooms, kitchenette, smoking rooms and vertical shafts for the building piping and ducts.

The main elevator corridor (3 m wide) and the service corridor (1,4 m wide), divide the core into four sections (*Fig. 2*). These sections are connected together by four beams (0,5 m × 0,9 m and 0,5 m × 1,2 m) in each direction, which are designed to carry the shear forces, and make these four sections work together as one large core. The reinforcement arrangement in these 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. 5*).

Due to the tower's proximity to a geological fault, crossing the Haifa Bay, 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, the overturning moment due to the earthquake loading was relatively large for a building of this size. As for the big bending moment, tension stresses at the outer walls of the core had to be

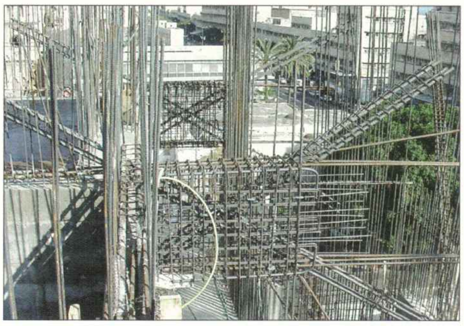

Fig. 5: Reinforcement arrangement of the core shear beams at construction

considered, and consequently, alternated vertical reinforcement bars were used.

Concrete and Steel

The concrete 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 floor 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 loading 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 600 mm diameter tension piles, penetrating the dolomite bedrock 10–17 m deep, were located below the perimeter wall of the core and arranged in pairs each 900 mm. A continuous foundation beam (pile cap of 2,8 m × 1,3 m) transfers the vertical forces from the core to the piles.

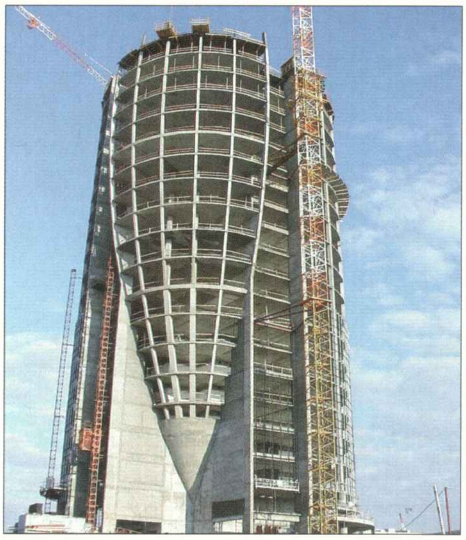

Fig. 6: Eastern spatial wing at construction

Construction

A special flying formwork system was adopted at all the repetitive floor locations. At the irregular floor areas the contractor used a different formwork system.

The forms were dismantled after 3 working days, when the concrete attained strength of at least 25 MPa, and re-shored again only using supporting poles for 18 more days. A cycle of one 850 – 1250 m^2 floor per week allowed the structure to be completed about one year after the completion of the foundations (*Fig. 6*).

Wind Loads

Only the main core resists the wind actions. This system proved to be quite effective, limiting the drift to 40 mm (at a height 113 m). 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 113 m) for a 10 year return period is $7{,}8 \times 10^{-3}$g.

Wind loading was determined using a model study of the tower and using 1:300 scales, according to the detailed architectural drawings.

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 was built in block outline from Styrofoam for a radius of 370 m. The upstream terrain was modeled coarsely using roughness representatives of the actual site topography for each azimuth.

A random dynamic analysis of the structure was then performed by applying the time histories of the simultaneous pressures to a dynamic mathematical model that reproduces mass distribution and the first nine vibration modes. 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 Computer Aided Design 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 structural 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 in both main directions respectively, and the torsion moment was 16 100 kNm. These values are given for a 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.

Conclusion

The aesthetics of the building is strongly enhanced by the structural solution, which was conceived by means of an intense architecture-structure interaction. The chosen structural solution and the construction method were very effective for the relatively complicated geometry of the building. The project foundation costs were relatively high due to the use of deep tension piles instead of a solid raft under the core area laid on the dolomite bedrock. This was an output of the proximity of the tower to a geological fault, which crosses at the Haifa Bay vicinity.

SEI Data Block

Owner:
Joint Venture of: Ashmoret Tichona Ltd
Industrial Buildings Corporation Ltd

Architect:
Dina Ammar-Avraham Curiel, Architects, Haifa

Structural design:
S. Ben-Abraham Engineers Ltd, Tel-Aviv

Project management:
Eng. Yoram Ganor, Ashtrom Properties Ltd.

Project site manager:
Eng. Yossi Lazar, Ashtrom Group

General contractor:
Partnership: Ashtrom Engineering & Construction Ltd
Sollel Bone Ltd

Project area (m^2):	37 000
Concrete (m^3):	22 500
Steel (t):	2 300
Total cost (USD millions):	50
Construction dates:	1999–2002