EFFECT OF WIND LOADS ON NON REGULARLY SHAPED HIGH-RISE BUILDINGS

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EFFECT OF WIND LOADS ON NON REGULARLY SHAPED HIGH-RISE BUILDINGS

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Abstract. *Wind loads have historically been recognized as one of the most important issue in high-rise buildings analysis and design. In particular, in regions of low seismic intensity, a high-rise building lateral design is controlled by wind loads. In wind analysis, Computational Fluid Dynamics (CFD) and/or wind tunnel testing are required to calculate the external pressures acting on a building.*

In this paper, two case studies are presented to show how the wind loads are calculated and applied in design. The first case study is based on the CFD results for the New Marina Casablanca Tower in Casablanca, Morocco. The second case study considers the results from the wind tunnel test studies conducted for the Al-Hamra tower, in Kuwait City, Kuwait.

The New Marina Casablanca tower is a 167m tall concrete building, with a unique twisting shape generated from the relative rotation of two adjacent floors. Sloped columns are introduced in the perimeter to follow the tower outer geometry and to support the concrete slabs spanning between the central core and the perimeter frame. The effects of wind loads on the twisted geometry has been studied in details since the pressure coefficients are not easily identified for such a complex form. In addition, the effect of the wind loads on the structure presented unique challenges that required innovative structural solutions.

The Al-Hamra tower is a 412m tall concrete building with a sculpted twisting form which optimizes the views to the Arabian Gulf while minimizing the solar heat gain. The complex form is realized using sloped walls and vertical columns on the perimeter and a central concrete core. The unique shape of the tower presented several design challenges related to the wind loads on the structure.

This paper will discuss the unique challenges and solutions associated with wind loads effect on buildings of unique form.

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1 INTRODUCTION

In the last century, a sudden increase of high-rise buildings expansion took place. High-rise buildings phenomena started with an American experiment in vertical building begun by Jennings, Burnham, Sullivan and other in Chicago after the Great Fire of 1871 which represents the starting point for new vision in the high-rise buildings in term of sustainability, human settlement and construction technologies.

High-rise structures represent a challenging problem which requires adequate structural solutions to minimize the structural effects due to external loads. On these structures, particular attention is required on the lateral loads effects, such as wind and earthquakes, in order to control the horizontal displacements, forces distribution and external pressure. In particular, in region of low seismic intensity, a high-rise building lateral design is controlled by wind loads (Sarkisian[1], Heiza and Tayel[2], Mendis et al.[3]).

In the aerodynamics high-rise structures analysis, horizontal forces distribution and wake shapes could be investigated by means of Computational Fluid Dynamics (CFD) analysis and wind tunnel tests. A modeling of neighbour constructions is required in both CFD analysis and wind tunnel tests since that variations of the flow may occur, as widely explained by Bungale and Taranath[4] and Cho et al.[5].

As given by Sarkisian[1] and Mendis et al[3], code-defined wind criteria must be used as the basis for all high-rise buildings design however these criteria are generally too conservative for high-rise structures. In fact, CFD analysis and wind tunnel testing may be used when design does not concern conventional structures.

In this paper the CFD analysis and wind tunnel test are applied on two non regularly shaped high-rise buildings. The first case study is based on the CFD results for the New Marina Casablanca Tower in Casablanca, Morocco. The second case study considers the results from the wind tunnel test conducted for the Al-Hamra tower, in Kuwait City, Kuwait. The aim of the work is to discuss the unique challenges and solutions associated with wind loads effect on buildings of unique form. Concerning the loads, in this paper only wind actions are considered.

2 CFD ANALYSIS

This section focuses on the case study application of CFD analysis on New Marina Casablanca Tower regarding the pressures on the façade, the wake shape and the velocity field of the airflow around the tower.

2.1 New Marina Casablanca Tower: architectural and structural viewpoint

The investigated high-rise structure is the New Marina Casablanca tower, designed by Arji Ali. It is a 167 m high-rise concrete building. New Marina Casablanca tower is composed by 42 storeys, having not a constant height. The unique architectural sign of the tower is represented by twisting shape. While the skyscraper’s 42 floorplates are all identical, each is slightly rotated of 3 degrees against the story below it, resulting in a full 140-degree twist over the course of the tower's 167 meter rise. Facades are realized by means of a modular system of triangular glass panels hold together with an aluminium frame.

From the structural viewpoint, the tower structural system consists in a concrete shear wall frame with external columns. Building foundation consists in a solid prestressed concrete slab of 3 metre thickness. Tower floors are designed by ordinary concrete slab and at the slab’s edge a rib will be designed in order to join the façade with the structural frame. Tower’s slabs are 28 centimetre thickness. Circular and rectangular columns together central core represent the New Marina Casablanca tower’s vertical elements. Along tower high, columns are twisted like tower shape. Circular columns diameter is 1.2 metre at the basement and 0.7 metre at the top. Rectangular sections columns is 1.65 x 0.3 metre at the basement and 1.05x0.3 metre at the top. Central core show semicircular shape with a shear wall thickness equal to 0.8 metre at the basement and 0.4 metre at the top. Material strength and properties of the vertical elements varying with tower high.

The influence of creep and shrinkage are not taken into account in the analysis.

In the same tower compound, New Marina Casablanca congress canter will rise. Congress center and tower are divided by a structural joint which allow to consider both structures separately.

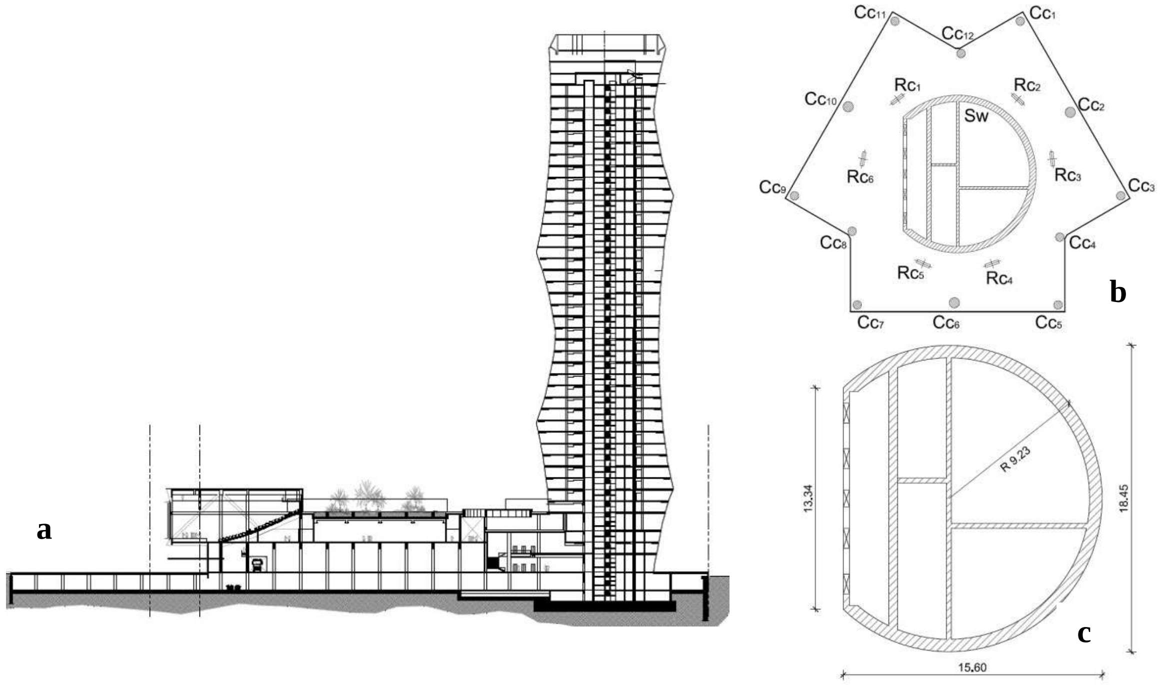

Figure 1: New Marina Casablanca tower cross section (a). Planimetric structural layout of the New Marina Casablanca tower showing the main horizontal resistant members (shear walls-Sw, circular coulumn-Cc, rectangular coulumn-Rc) (b). Shear walls' geometric dimensions, in metres (c).

2.2 CFD model

In order to define airflow distribution around the high-rise buildings and to compute forces acting on the surface of the towers for different wind directions the CFD model is carried out. The fundamental basis of the CFD model are represented by the Navier–Stokes equations, which define any single-phase fluid flow. Geometry (physical bounds), volume occupied by the fluid and boundary conditions represents the fundamental elements to define a CFD model. When these fundamental elements are defined, CFD model is ready to solve the equations iteratively as a steady-state or transient. Finally a postprocessor is used for the analysis and visualization of the resulting solution. In order to solve differential equation the ICON® FOAMpro software has been useful and to plot CFD results the Paraview® software has been used. CFD model parameters are following are represented by wind speed ($U = 30.12$ m/s), steady state simulation, turbulence model (k-ω SST), air density ($\rho = 1.225$ kg/m^3), air dynamic viscosity ($\mu = 1.82 \cdot 10^{-5}$ Pa·s), model number of elements ($n \approx 50$ M hexahedra), tower diameter ($D \approx 40$ m), CFD domain dimensions (17×12×27 D)

Two different wind direction cases are analysed: the first one is represented by wind blows from the congress center to the tower instead the second one is represented by wind blows from the tower to the congress center.

CFD domain is reported in Figure 2.

2.3 CFD results

Static pressure coefficient distribution is the first CFD data analyzed. It represents the static pressure in a generic point on high-rise façade. Static pressure coefficient is given as the ratio between the pressure difference and kinetic pressure and it can be calculated as:

$$C_p = \frac{p - p_\infty}{\frac{1}{2}\rho_\infty V_\infty^2} \tag{1}$$

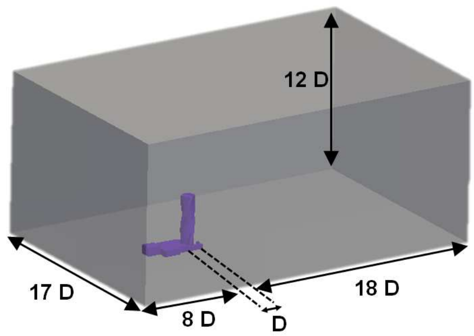

Figure 1: CFD domain

where p is the pressure in a generic point of the CFD model, p_∞ is the pressure in the undisturbed condition, ρ is the air density, conventionally equal to 1.25 kg/m^3, V_∞ is the freestream fluid velocity. Positive C_p values mean a windward façade condition whereas negative C_p values mean a leeward façade condition. Static pressure coefficient for New Marina Casablanca tower is plotted in Figure 3 for two different wind direction cases analyzed.

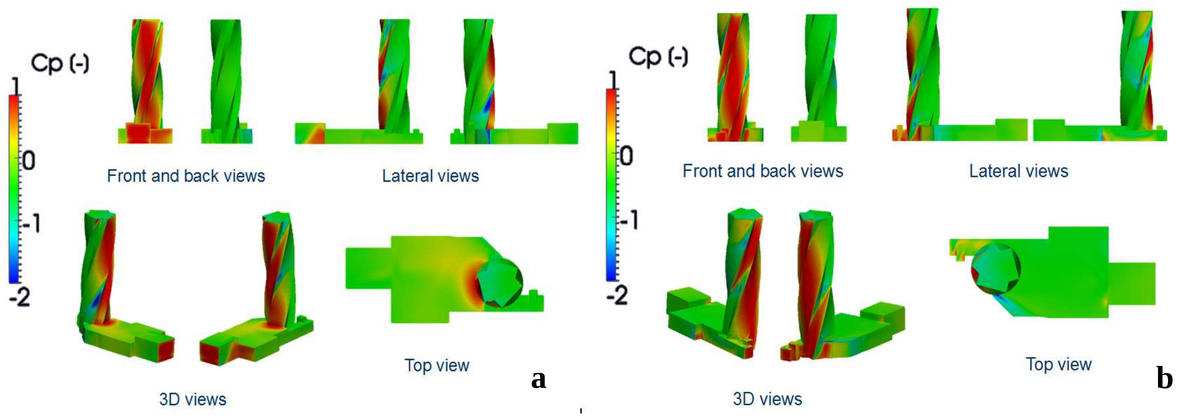

Figure 3: Static pressure coefficient distribution for the congress center to the tower wind direction (a) and for the tower to the congress center wind direction (b).

In both cases, a upwind area along high-rise façade with a positive C_p value is possible to observe when wind load is applied directly on high-rise façade. Contrary, a downwind area along high-rise façade with a negative C_p value is possible to observe when wind load is applied indirectly on high-rise façade. In particular, negative C_p value is due to vortex created when wind interact with the facades corners: in these points, a separation airflow occurs creating air vortex. Due to New Marina Casablanca tower's twisting shape, a positive and negative C_p values is possible to note along lateral sides of the tower.

By means of **wake shape** results is possible to understand the airflow and high-rise building interaction. In Figure 4 the total pressure coefficient zero-isosurface (colored with velocity vector magnitude) are plotted for t two different wind direction cases analysed.

In order to characterize the wake size along the high-rise building, the drag coefficient C_x is defined. C_x coefficient is composed by two drag forces contribution called as friction drag forces and shape drag forces. It can be calculated as:

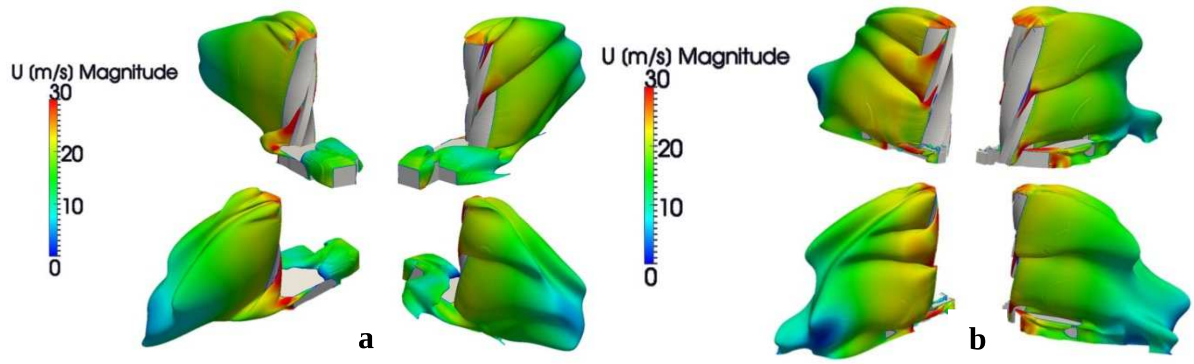

Figure 4: Total pressure coefficient zero-isosurface for the congress center to the tower wind direction (a) and for the tower to the congress center wind direction (b).

$$C_x = \frac{D}{\frac{1}{2}\rho V^2 S} \tag{2}$$

where D is the drag, ρ is the air density, S is the impacting surface, V is the is the speed of the body referred to the undisturbed fluid (or the speed of the fluid referred to the fixed body). In Figure 4, flow separation at the tower facade corners is possible to observe with a gradual velocity decrease around New Marina Casablanca tower. Moreover, a flow separation at the tower top is observed providing downwind condition in roof tower zone, as observed in Figure 3.

Air velocity field is plotted Figure 5, where it is possible to note airflow velocity before and after impact on high-rise building. In vertical tower section, when the distance to the tower increases an undisturbed airflow is possible to observe with a blue color in the velocity maps. Undisturbed airflow means to have "zero velocity" region around the tower and in particular along the high-rise building façade. In "zero velocity" region an air recirculation occurs. On the other hand, when the distance to the tower decrease an airflow and tower interaction occurs with a velocity field variation highlighted with a red color in the velocity maps. In the same figure, the air velocity field in the horizontal tower section is plotted. In both wind direction cases analysed, a unique shape of the velocity fields is observed due to unique New Marina Casablanca tower's shape.

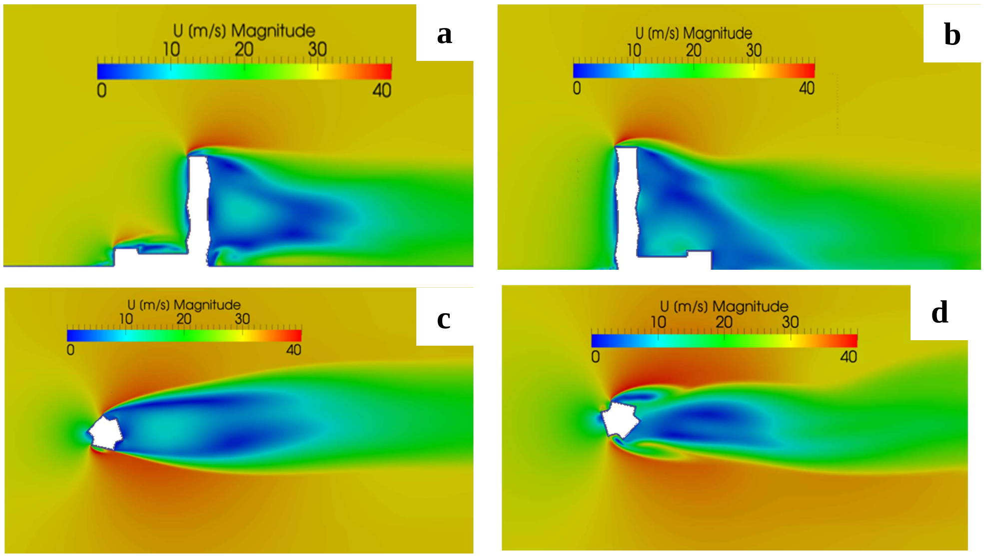

Figure 5: Air velocity field along vertical and horizontal tower section for the congress center to the tower wind direction from (a), (c) and for the tower to the congress center wind direction along vertical and horizontal tower section (b), (d).

3 AL HAMRA TOWER

The second case study considered in this paper is the 412m tall reinforced concrete Al Hamra tower, located in Kuwait city, Kuwait.

Figure 6: Image of the Al Harma tower.

The architectural design of the Al Hamra Tower is a carefully considered response to site specific environmental and urban conditions. Located on a space-constrained site at a prominent intersection in the center of Kuwait City, the Al Hamra Tower is part of a mixed-use complex consisting of a commercial office tower, a retail/entertainment podium and an associated parking structure. At the commencement of Skidmore, Owings & Merrill LLP's (SOM) involvement in the design of the tower, the podium and parking structures were already designed and under construction. The remaining site available for the tower defined both the plan limits and alignment of the superstructure. Located immediately north of the retail podium and east of a major road, a tower geometry which opened up to the retail entrance at the southwest quadrant of the tower site was desirable. However, with the primary gulf views valued by future office tenants to the north, west and east, a form which focused the office spaces in those directions was preferred. To accommodate these seemingly conflicting interests, a spiraling geometry was developed by subtracting a quadrant of a typical filleted square floor plan and incrementally rotating the subtracted portion at each higher level. The surface generated by the cut slab edges is articulated as a stone-clad continuous ribbon which connects the hyperbolic paraboloid shear walls extending from the southwest and southeast corners of the central core (termed the 'flared' walls) and the roof of the tower. This expression of the flared wall and the exposure of the south wall of the central core allowed for extensive glass use on the north, west and east sides of the tower, while providing a measure of environmental protection from the desert sun by presenting a nearly solid stone façade to the south.

The sculpted twisted form of the tower is realized using sloped reinforced concrete walls and vertical columns on the perimeter and a central concrete core.

Due to the relatively low seismicity of the area, the structural design of the tower is controlled by the wind induced forces on the building. The unique shape presented several challenges related to such wind loads on the structure.

3.1 Lateral Force Resisting System

The lateral system for resisting the controlling wind and gravity load combinations consists of a cast-in-place reinforced concrete shear wall core supplemented by a perimeter moment resisting frame. The shear walls at the core were deliberately sized with thicker walls on the outside of the core and thinner cross walls, optimizing the placement of material to maximize the resistance of the core to the gravity load induced torsion. The flared walls which connect back to the core also participate in the lateral force resisting system. Although wind design load combinations controlled the design of the lateral force resisting system, the seismic design loads were not insignificant and accounted for in the design. As the shear wall core resists the majority of the wind induced forces, the most efficient approach to the seismic design of the tower is to designate the same reinforced concrete shear walls only to be the Seismic Force Resisting System. This allowed a full seismic design of the tower without the need to increase the use of materials anywhere in the structure. The reinforced concrete shear walls in the Al Hamra Tower vary from 1200mm to 300mm in thickness, and from 80MPa to 50MPa in cube compressive strength. The moment resisting frames are typically 800mm wide by 600mm deep and are poured with the floor framing using 40MPa concrete (cube compressive strength).

3.2 Gravity Force Resisting System

The gravity force resisting system for the Al Hamra Tower is significantly more complex and required more in depth consideration than in a conventional tower design. Reinforced concrete cast-in-place slabs span circumferentially between reinforced concrete gravity beams that, in turn, span between the core and the perimeter frame. The unusual geometry of the tower resulted in significant loads being transferred between the flared walls and the core through the reinforced concrete diaphragms. Rather than participating only in the lateral force resisting system, the diaphragms are an integral part of the gravity force resisting system. The increased importance of the diaphragms meant that a wider gravity beam spacing and a thicker slab was preferred over a solution with more frequent gravity beams and a thinner slab. By using a 160mm slab spanning between beams at 6.0m on center, only slightly more material needed to be used compared to a solution with a thin slab spanning 3.0m on center, but a greater proportion of the materials used contributed to the diaphragm shear capacity of the slabs. 700mm deep reinforced concrete gravity beams span 10.6m between core and perimeter frames. The perimeter columns vary from 1200mm square to 700mm square. Composite columns are used from mat foundation level to level 29, with embedded W360 steel column sections of varying weights, allowing 1100mm square columns to be used in all typical office floors from level 40 down to level 5. 1200mm square columns are required below level 5 due to the increased story heights within mechanical floors and double height podium levels. Reinforced concrete in the perimeter frame columns varies from 80MPa to 50MPa (cube compressive strength), and beam and slab floor framing is all constructed using 40MPa concrete (cube compressive strength).

3.3 Wind Tunnel Testing

As mentioned in the previous sections, due to the relatively low seismicity of the area, the structural design of the tower is controlled by the wind induced forces on the building. Wind tunnel testing was required to establish a suitable wind design criteria. The synoptic wind patterns in the gulf region are the result of the large scale movement of air channelled along the north-westerly/south-easterly axis of the Persian gulf. The wind climate is further affected by the local topography in each area around the gulf. Very localized and short term wind phenomena are know to exist in the gulf region due to thunderstorms producing strong downbursts close to the ground. These downbursts result from a cold air mass being deflected downward by a moving warm air mass, due to a strong temperature gradient. The incidence on the ground surface of the cold air mass generates short duration high intensity winds. Drawing on their experience working in Kuwait and other cities in the region as well as wind speed data measured at the Kuwait airport, the project wind engineer BMT fluid mechanics ltd (BMT) established a basic mean hourly wind speed of 23m/s at 10m height in open terrain. This value represented the 50 year return period synoptic wind event consistent with the methodology of ASCE 7-02. After extensive study of the non-synoptic thunderstorm wind events it was determined that although these events could generate greater wind speeds than the synoptic events between ground level and an elevation of approximately 150m, the thunderstorm events resulted in significantly lower wind speeds higher than 150m above grade. While the thunderstorm wind profile could prove to be the critical wind event for the structural system of a tower lower than 200m in height, the gross effect of the synoptic wind profile over the full height of the al hamra tower controlled the design in all aspects other than localized cladding pressures on the lower stories.

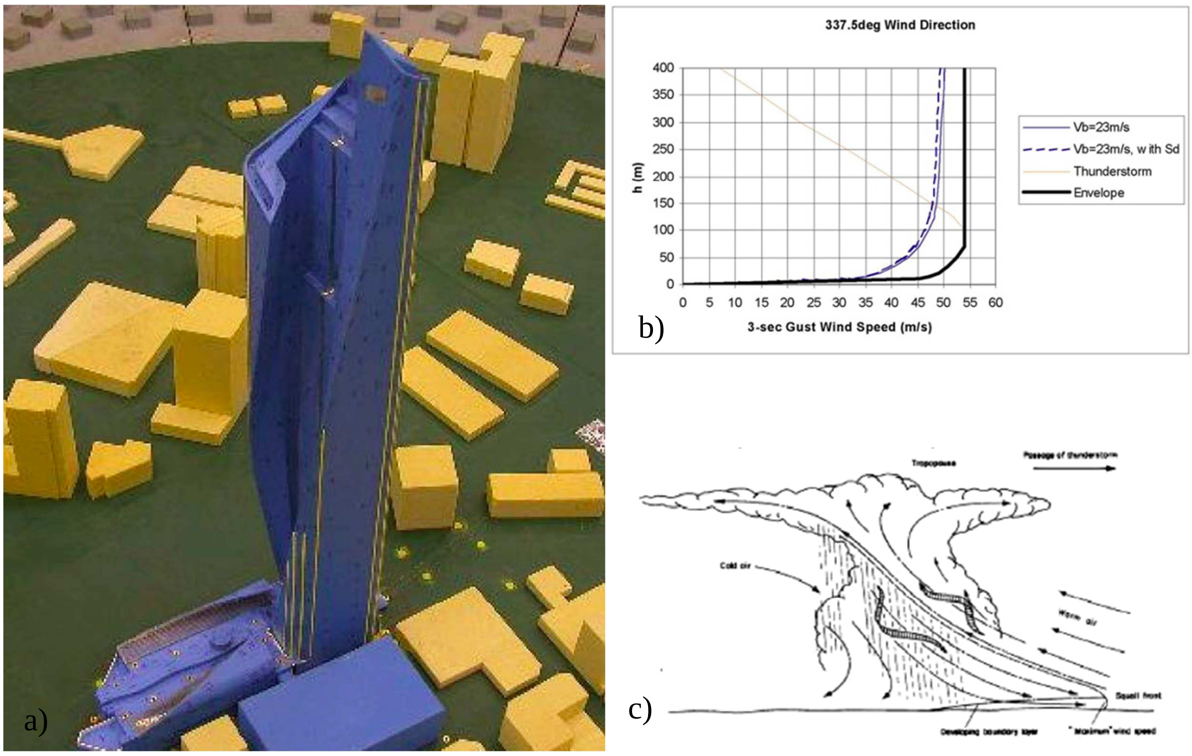

Figure 6: a) Wind tunnel test model; b) Comparison of Synoptic Wind and Thunderstorm profie; c) Structure of a thunderstorm cell (from ESDU Item 87034).

3.4 Tower Analysis and Design

The analysis and design of the tower structure was based on the results of a series of three dimensional finite element analysis models run in parallel. A *serviceability model* was used to establish the fundamental building periods of the structure, used in the calculation of seismic design forces and in establishing the design wind loads through wind tunnel testing performed by BMT. This model was also used to verify that the structure was stiff enough to meet the established wind drift criteria for the project (height/500 for 50-year return period design wind loads). A *wind design model* was used for the design of the shear wall core and perimeter moment resisting frame when subjected to gravity and wind load combinations. Cracked stiffness modifiers were used on elements of the lateral force resisting system in accordance with the provisions of ACI-318M. The shear wall designs were then verified by using a *seismic design model*, which applied all seismic load combinations to an analysis model that had been modified by moment releasing the ends of each of the perimeter moment resisting frame beams. In this way the reinforcement layouts designed using the *wind design model* were verified as being suitable for resisting the seismic loads using only the seismic force resisting system. Lastly, as the building twists elastically under gravity loads it is the walls at the perimeter of the core which are primarily resisting the torsional moment applied to the core through their shear stiffness and their circumferential alignment relative to the center of stiffness of the core (a torsion tube). These shear walls experience elastic shear deformations, but as the applied load is permanent it can be expected that these walls will also creep thus resulting in additional inelastic shear deformations of the walls and therefore twisting of the core. The magnitude of shear creep deformations to be expected is difficult to calculate, but a best estimate of this value was established by using the recommendations for shear deformations due to creep in deep beams made in the report of ACI committee 209, ACI 209R-92 *Prediction of Creep, Shrinkage and Temperature Effects in Concrete Structures*. This procedure describes an approach for the estimation of an effective shear modulus for the deep concrete element that will result in the anticipated long term shear deformation when subjected to a shear stress. SOM used this procedure to estimate an appropriate effective shear stiffness for the shear walls and ran a *torsional creep compatibility model* to investigate the effect of this reduction of effective stiffness on the core and other structural elements. This analysis confirmed that which can be easily predicted - that creep is a strain at constant stress phenomenon. Reducing the shear stiffness of the core resulted in increased gravity-induced twisting of the tower, but little increase in forces in any of the shear walls. To ensure compatibility of the perimeter frame with the possible reduced torsional stiffness of the core, SOM designed the perimeter frame to elastically resist the additional forces observed in the perimeter moment frame resulting from this increased long-term gravity twist.

4 CONCLUSIONS

Two case studies have been presented in this paper showing the unique engineering challenges associated with the development of a structural system to resist the wind forces acting on buildings on unique form. This work evaluates the accuracy and the effectiveness of a CFD and wind tunnel analysis through two case studies regarding New Marina Casablanca Tower and Al Hamra Tower. CFD results have been useful to define static pressure coefficient, wake shape and air velocity field on high-rise building. Wind tunnel testing was required to establish a suitable wind design criteria and to understand Al Hamra Tower's real structural behaviour under wind effect.

Present paper confirms the usefulness of two wind study approaches on buildings of unique form.

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