Title: **Sculpted High-rise The Al Hamra Tower**

Authors: Mark Sarkisian, Director, Skidmore, Owings & Merrill LLP
R. Agarwal
N. Atari
L. Hu
Neville John Mathias, Associate Director, Skidmore, Owings & Merrill LLP
Aaron Mazeika, Associate, Skidmore, Owings & Merrill LLP

Subjects: Architectural/Design
Structural Engineering

Keyword: Supertall

Publication Date: 2007

Original Publication: Structural Engineers World Congress 2007

Paper Type:
1. **Book chapter/Part chapter**
2. Journal paper
3. Conference proceeding
4. Unpublished conference paper
5. Magazine article
6. Unpublished

SCULPTED HIGH-RISE: THE AL HAMRA TOWER

Mark SARKISIAN PE SE
Director
Skidmore, Owings & Merrill LLP
San Francisco, CA, USA
mark.sarkisian@som.com

Mark Sarkisian received his Bachelors Degree in Civil Engineering from the University of Connecticut and his Masters Degree in Structural Engineering from Lehigh University. He joined SOM in 1985.

Neville MATHIAS PE SE
Associate Director
Skidmore, Owings & Merrill LLP
San Francisco, CA, USA
neville.mathias@som.com

Neville Mathias, received his Bachelors Degree in Civil Engineering from Madras University, India and Masters Degrees in Structural Engineering and Architecture from the University of California, Berkeley. He joined SOM in 1985.

Aaron MAZEIKA PE
Associate
Skidmore, Owings & Merrill LLP
Chicago, IL, USA
aaron.mazeika@som.com

Aaron Mazeika received his Bachelors and Masters Degrees in Engineering from the University of Cambridge. He joined SOM in 2000.

Keywords: Free Form Design; Sculpted Tower; Twisted Tower; Hyperbolic Paraboloid; Super High-Rise; Mixed-Use; High-Strength Concrete; Creep and Shrinkage; Parametric Modeling; Buckling Analysis

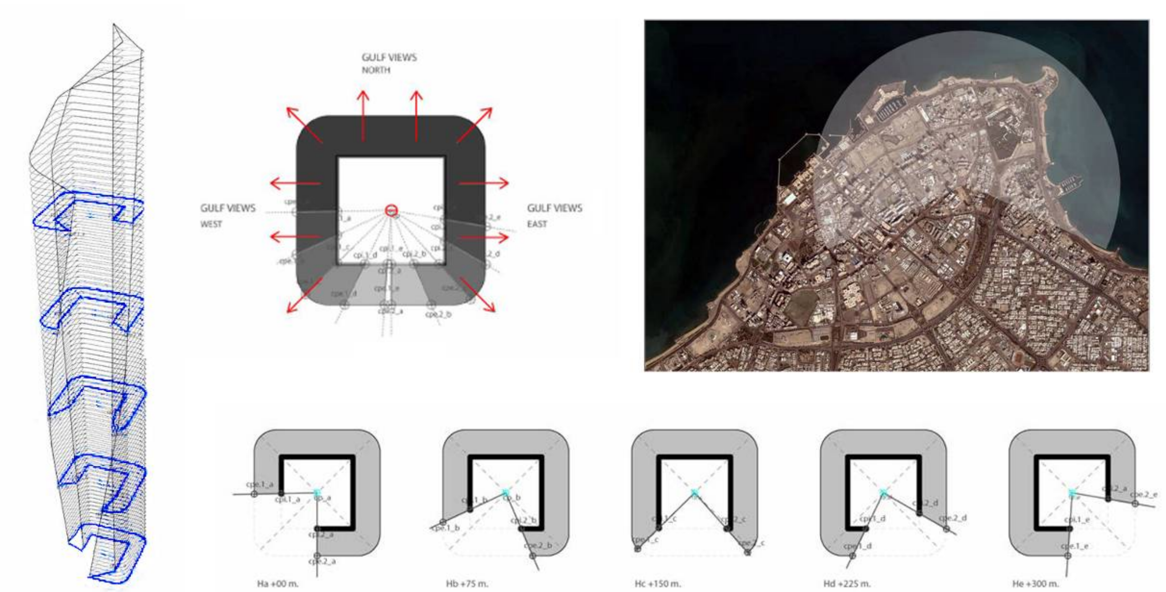
Extended Abstract

At 412m tall on completion, the Al Hamra Tower is set to be amongst the tallest buildings in the world. Setting it apart from other super high-rise buildings is its unique sculpted form. An example of architectural expression through structural form on a grand scale, the structural system and exterior form developed together in a process of symbiotic evolution. The building geometry is generated by a spiraling slice subtracted from a simple prismatic volume. The two resultant surfaces are hyperbolic paraboloid reinforced-concrete walls, which extend the full height of the tower and participate in the lateral and gravity force resisting systems.

The design of the Al Hamra Tower required consideration of challenging engineering issues complicated by both the height and form of the structure. The spiraling hyperbolic paraboloid

‘flared walls’ required for gravity load support of the cantilevered wing of the building apply a torsional gravity load to the building core that necessitates consideration of both the long-term vertical and torsional deformations of the building structure.

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.



Presently under construction in Kuwait City, the Al Hamra Tower will be an impressive addition to the skyline of this fast-growing city. As part of a mixed-use development combining world-class office space, a high-end retail mall, and an entertainment center, the Al Hamra Mixed-Use Complex is set to become a major destination for the city.

SCULPTED HIGH-RISE: THE AL HAMRA TOWER

Mark SARKISIAN PE SE
Director
Skidmore, Owings & Merrill LLP
San Francisco, CA, USA
mark.sarkisian@som.com

Mark Sarkisian received his Bachelors Degree in Civil Engineering from the University of Connecticut and his Masters Degree in Structural Engineering from Lehigh University. He joined SOM in 1985.

Neville MATHIAS PE SE
Associate Director
Skidmore, Owings & Merrill LLP
San Francisco, CA, USA
neville.mathias@som.com

Neville Mathias, received his Bachelors Degree in Civil Engineering from Madras University, India and Masters Degrees in Structural Engineering and Architecture from the University of California, Berkeley. He joined SOM in 1985.

Aaron MAZEIKA PE
Associate
Skidmore, Owings & Merrill LLP
Chicago, IL, USA
aaron.mazeika@som.com

Aaron Mazeika received his Bachelors and Masters Degrees in Engineering from the University of Cambridge. He joined SOM in 2000.

Keywords: Free Form Design; Sculpted Tower; Twisted Tower; Hyperbolic Paraboloid; Super High-Rise; Mixed-Use; High-Strength Concrete; Creep and Shrinkage; Parametric Modeling; Buckling Analysis

Abstract

At 412m tall on completion, the Al Hamra Tower is set to be amongst the tallest buildings in the world. Setting it apart from other super high-rise buildings is its unique sculpted form. An example of architectural expression through structural form on a grand scale, the structural system and exterior form developed together in a process of symbiotic evolution. The building geometry is generated by a spiraling slice subtracted from a simple prismatic volume. The two resultant surfaces are hyperbolic paraboloid reinforced-concrete walls, which extend the full height of the tower and participate in the lateral and gravity force resisting systems.

The design of the Al Hamra Tower required consideration of challenging engineering issues complicated by both the height and form of the structure. The spiraling hyperbolic paraboloid

‘flared walls’ required for gravity load support of the cantilevered wing of the building apply a torsional gravity load to the building core that necessitates consideration of both the long-term vertical and torsional deformations of the building structure.

Presently under construction in Kuwait City, the Al Hamra Tower will be an impressive addition to the skyline of this fast-growing city. As part of a mixed-use development combining world-class office space, a high-end retail mall, and an entertainment center, the Al Hamra Mixed-Use Complex is set to become a major destination for the city.

1. Sculpted form

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.

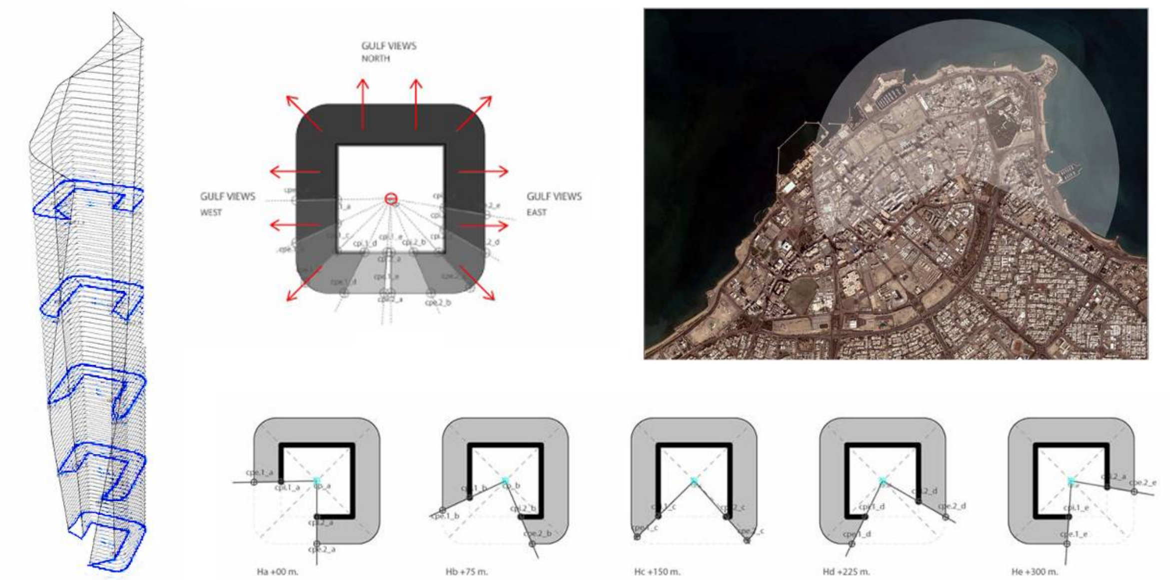


Fig. 1 Tower Defining Geometry

2. Tower Primary Structural System

2.1. Geometry Considerations

Early studies of the twisted shape of the south side of the building structure were able to predict the global behavior of the tower structure and suggest areas in the structure which would require careful consideration during the detailed design process. Initial studies of the center of mass of each diaphragm indicated that the effect of the rotating quadrant removed from the floor plate was that the center of mass was offset to the east for the lower third of the tower, was approximately aligned with the geometric center of the tower through the middle third of the height, and was offset to the west for the upper third. However, these two offsets cancelled each other out and the center of mass for the overall tower was well aligned in the east-west direction with its geometric center. In the north-south direction the center of mass of the tower was approximately 7.0m to the north of the geometric center of the tower. Fortunately, these conditions fit the tower program within the existing excavation at the site. The tower columns needed to be built to the edge of the mat on the south, east and west sides, however, there was an extension of the excavation to the north beyond the tower footprint, allowing the mat geometry to be biased to the north to match the tower mass.

An analysis of the load paths through the structure also highlighted the great difference between the behavior of the southeast and southwest flared walls. At the southeast flared wall, the hyperbolic paraboloid shear wall leans in on the building structure. Therefore only the small areas of floor slab that frame directly into the wall at each level can add gravity load into the wall. In fact at approximately every 7 stories the wall intersects a perimeter column and a load path exists where gravity load can divert out of the wall into the perimeter column. Consequently the southeast flared wall is relatively lightly loaded up the full height of the wall. Conversely the southwest flared wall leans away from the building structure. This means that as well as the small areas of floor load that are applied to the flared wall at every story, approximately at every 7 stories a perimeter column exists vertically above the flared wall, but not below it. This means that at these locations the full gravity load in these interrupted columns is transferred to the flared wall. The resulting gravity loads in the southwest flared wall are very high - in fact the full gravity load of every area of framed structure in the southwest quadrant of the tower and south of the location of the flared wall at ground level is carried by this wall. The impact of this load concentration is apparent in the raft bearing pressures and pile loads diagramed in the previous section, as well as in the flexural and shear demands considered in the design of the raft foundation in this area. Early recognition of the importance of this load path allowed the structural team to influence the functional use planning of the southwest quadrant of the tower. Most notably at each atypical floor (mechanical floors, sky lobbies, refuge floors), careful consideration was made in the location of zones requiring high floor load capacities. This effort included placing water storage tanks and heavy mechanical equipment away from the southwest quadrant, and when the available space on a mechanical or refuge floor exceeded the required floor area, designating the southwest quadrant as unoccupied, allowing this zone to be designed using nominal floor load capacities.

2.2. Torsional Response to Gravity Loads

While the load paths flowing through flared walls were easily understood, the impact of those load paths on the base building structural system required careful consideration. As with most structures with inclined columns and walls supporting floor framing, a horizontal force is applied to the slab at the intersection of the inclined element and the slab, The slab adds gravity loads to the inclined element, and the vertical load in the inclined element is increased accordingly. To maintain an axial load path the horizontal component of force in the inclined element must increase along with the vertical component, and to satisfy static equilibrium at the slab interface, the slab must apply a horizontal load to the intersection. If the inclined element slopes away from the slab, the slab goes into tension, if it slopes towards the slab it goes into compression. For the specific conditions of the flared walls in the Al Hamra Tower, the direction of lean of the flared walls is always predominantly circumferential and counter-clockwise when viewed from above, therefore for resolution of static equilibrium a counter-clockwise circumferential force is applied at each slab to flared wall intersections. These forces each impart a counter-clockwise torsional moment on the lateral force resisting system of the tower. The cumulative effect of these torsional moments applied to each floor diaphragm is a net torsional moment applied to the lateral system of the structure that increases from zero at the top of the structure to a maximum at the base of the building. This torsional moment causes a twisted displaced shape clearly observed in the results of the finite element analysis model when subjected to gravity loads only.

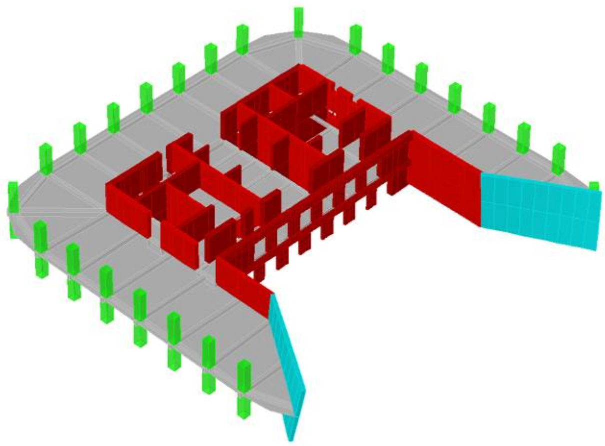


Fig. 2 Analysis Model at Typical Floor

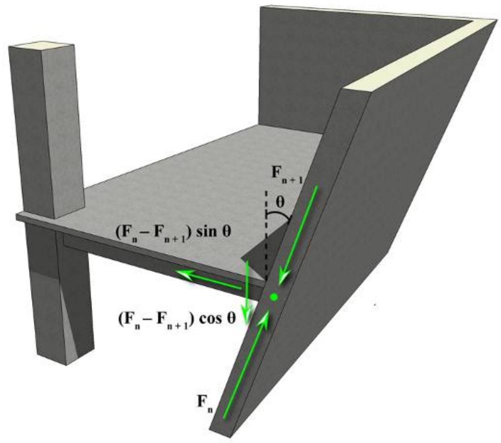


Fig. 3 Static Equilibrium at Flared Wall

2.3. 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 wall core was designed with thicker walls on the perimeter of the core, 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. Therefore a full seismic design of the Al Hamra Tower was performed on the designated Seismic Force Resisting System. As the shear wall core was resisting the majority of the wind induced forces, it was determined that the most efficient approach to the seismic design of the tower would be to designate only the reinforced concrete shear walls to be the Seismic Force Resisting System. This allowed a full seismic design of the tower to be performed without needing 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 compressive strength (cube compressive strength). The moment resisting frame beamss are typically 800mm wide by 600mm deep and are poured with the floor framing using 40MPa concrete (cube compressive strength).

2.4. Gravity Force Resisting System

The gravity force resisting system for the Al Hamra Tower required more in depth consideration than that for a conventional tower design. Cast-in-place slabs span circumferentially onto reinforced concrete gravity beams themselves spanning between core and perimeter frame. However 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 only participating 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 thicker slab was preferred over a more conventional 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 was used than 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 frame. 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).

2.5. 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.

3. Lobby Lamella Structure

3.1. Lobby column bracing

At the north side of the building is the main lobby of the office tower. The Lobby is a 24m high space that extends from the building core to the perimeter frame. To increase the area of the lobby the north columns of the tower, which a are vertical from level 12 to the tower roof, slope away from the building core following a circular arch. This results in these columns intercepting the ground level slab 7.6m further to the north. The result of this movement is that the main tower columns passing through the lobby are 24m tall and curved, developing large bending moments in the columns. To address issues of slenderness in these otherwise unbraced columns, a lamella bracing scheme was devised which reduces both the unbraced length of the primary tower columns and the load demands through load sharing with parallel members. The primary load-bearing structural components of the lobby lamella structure include the building perimeter composite columns (the “A” elements) and bracing elements within the plane of the north façade (the “B” elements). Out of plane bracing of the “A” and “B” elements is provided by the “C”, “D” & “E” elements, which brace the north façade back to the building core. At grade level, collector elements were provided to distribute the net horizontal thrust from the column base to the adjacent slab.

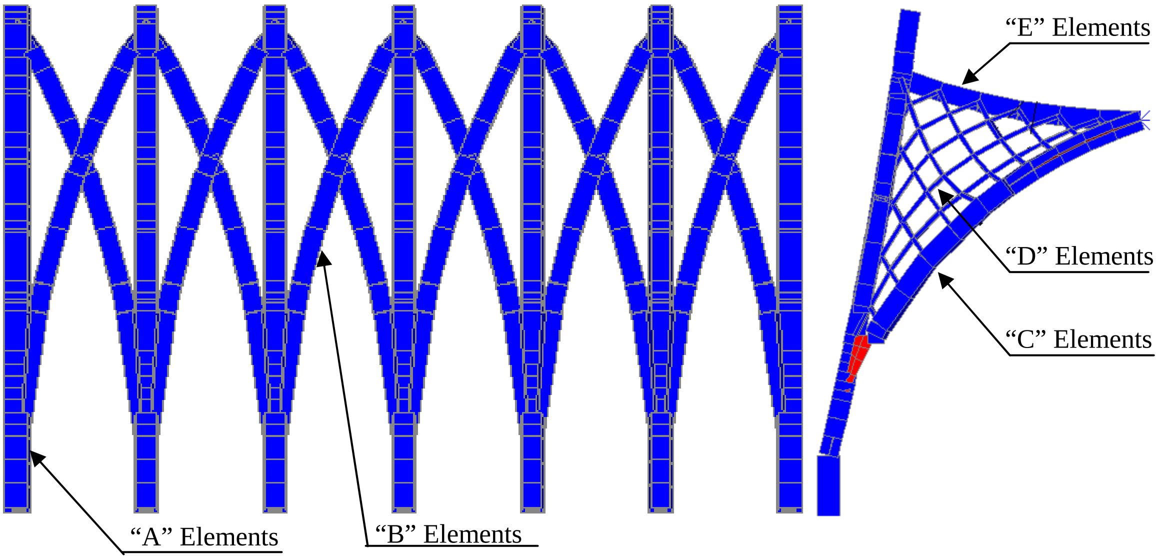


Fig. 4 Lobby Lamella Naming Convention

3.2. Stability Analysis

A complete three-dimensional finite element analysis model of the lobby lamella structure was built to study the effectiveness of the bracing scheme that had been developed and to guide the architectural design of these elements. A series of non-linear buckling analyses were performed on the lamella scheme, each model adding the next layer of bracing elements. The models analyzed included “A” elements alone, “A” and “B” elements, all elements except “D” elements and finally all the lobby elements. The buckling mode of the first model was weak axis buckling of the “A” elements. The addition of the “B” elements reduced the weak axis buckling length of the “A” so that strong axis buckling controlled and reduced the proportion of the applied load that was carried by the “A” elements, due to the “B” elements sharing the load. The product of these two factors was that the buckling load increased by a factor of two. The addition of the “C” and “E” elements reduced the strong axis buckling length of the “A” elements, nominally increasing the buckling load further. However, up to this point the buckling load of the “A” elements was still slightly lower than their load demand. The addition of the "D" elements had the single biggest impact on the buckling load of the system. By tying together the "A", "B", "C" and "E" elements, buckling failure of any of these elements is effectively prevented and the critical buckling mode became the buckling of the "A" elements at the first conventional story above the lobby. This study confirmed the concept of the lamella bracing scheme and demonstrated the structural importance of all the elements in the lamella.

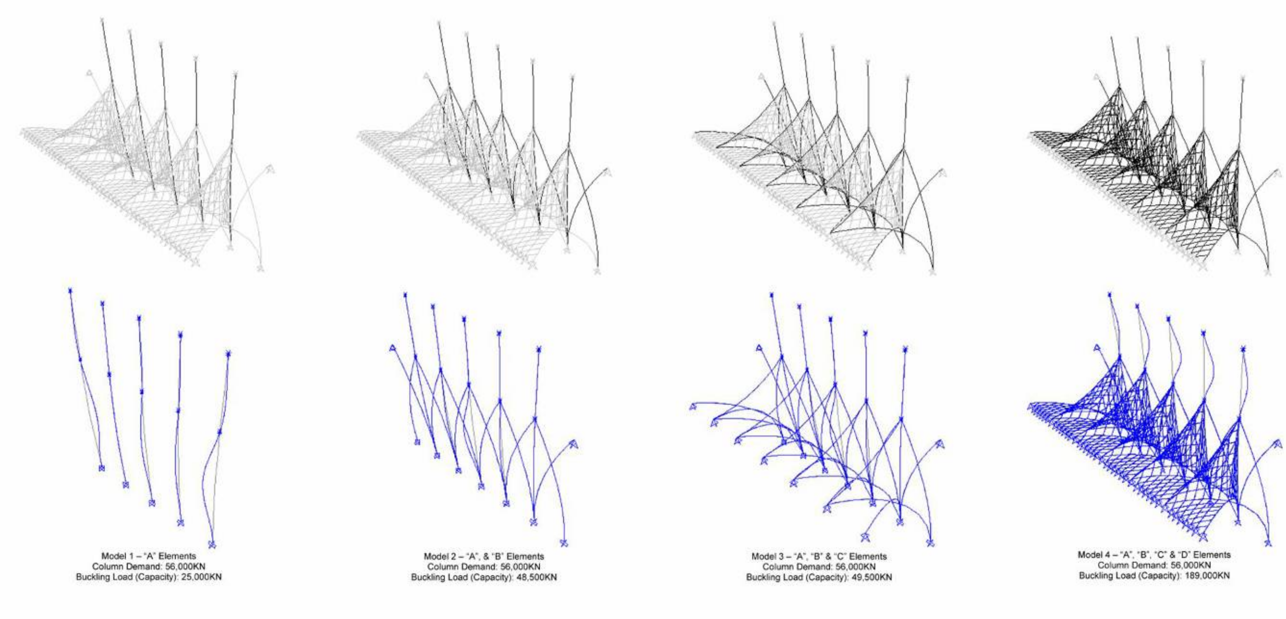


Fig. 5 Lamella Buckling Analysis Models

3.3. Element Design

The strength design of the lamella structural members was performed as part of an evolutionary form-finding exercise performed hand in hand with the architectural design team. A parametric three-dimensional model of the lamella structure was modeled using Gehry Technologies' Digital Project software. All elements in the model were tied to a few controlling geometrical parameters such as the overall radius of the north façade of the tower, the maximum allowed spacing between the "D" elements, or the section size at each end of the elements. At each step a centerline model of the lamella was exported and brought into the structural analysis software, and structural feedback from the analysis results guided improvements to the form of the lamella structure. The final layout of the "D" elements was determined by replacing them with shell elements in the analysis model and reviewing plots of the direction of action of principal stresses to determine the most effective alignment for these members. The detailed design of the lamella elements was based on non-linear second order static analysis carried out with appropriate stiffness reduction modifiers assigned to all composite and concrete elements. The lobby lamella was modeled separately from the overall superstructure analysis model, with force and displacement demands on the lamella structure determined from the overall superstructure model and applied at the boundary conditions of the lobby model. To avoid the "C" elements acting as coupling elements transferring vertical loads between the core and the perimeter frame, a sliding connection between the "C" elements and the core was introduced.

As the design of the "A" and "B" elements depended on load sharing between the two, careful consideration was made to determine if long-term response of the system would result in the proportions of load carried by each element varying over time. A creep and shrinkage analysis was carried out using a similar procedure to the overall column shortening procedure described above. The elements were subjected to elastic, creep and shrinkage deformations under long term loads (vertical loads only). It was observed that "A" and "B" elements had different rate of long term shortening. Due to strain compatibility, this differential shortening led to a redistribution of forces between the “A” and “B” elements. Thus the internal forces in these lobby elements were re-calculated based on the results of the creep and shrinkage studies. Design forces in members which reduced over time were not modified whereas design forces in members with increased internal loads were corrected accordingly.

3.4. Lobby Construction

To prevent the duration of construction of the lobby lamella from having a negative impact on the overall construction schedule, the observations of the incrementally beneficial effect of adding bracing elements was incorporated into the design schedule. While the “A” and “B” element must be in place prior to the construction of the floor slab above, construction of the typical floors above is allowed to proceed as work continues on the lobby lamella. A limit on the number of floors that may be poured prior to installation of the “C” and “E” elements as well as another limit prior to installation of all the “D” elements. Shop drawings for all the work were generated from three-dimensional models of the lamella structure, with fiberglass formwork moulds being fabricated directly from these models.

Fig. 6 Architectural Rendering of Lobby

Fig. 7 Lobby Construction in Progress

4. Conclusions

The design and construction of the Al Hamra Tower is a significant step forward both in terms of architectural design form and process. Blending the conventional tools of the engineer and the computer programmer and by leverage the latest three-dimensional parametric modeling software, SOM has brought together the realms of free-form design and the super high-rise skyscraper. The result is an exciting, dynamic architecture, representative of the increasing design freedoms afforded to us in this digital age.