SCULPTED HIGH-RISE: THE AL HAMRA TOWER

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ABSTRACT

At 412m tall on completion, the Al Hamra Tower is set to be amongst the ten 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. As one of the few reinforced concrete super high-rise buildings, long-term creep and shrinkage of concrete were carefully studied to account for force redistributions and to develop a program of displacement pre-corrections made during construction. 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.
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
GENERAL DESIGN ISSUES

Fast-Track Construction
SOM’s design work was performed under the most demanding of fast-track conditions due to the unique history of the development of the site. In 2004, the design of a mixed-use complex incorporating a 200m office tower and a retail podium with an associated parking structure by local Architecture & Engineering firm Al Jazera Consultants (AJC) was essentially completed. While the basement excavation and necessary dewatering was well under way, the maximum allowable height of 200m established by the municipality of Kuwait City was increased to 400m. At this point the Client group sought the services of SOM, being a design firm experienced in the complexities of super high-rise towers, to design a landmark tower for Kuwait City. This chain of events resulted in SOM commencing concept design while foundation excavations were already in progress and the Contractor was mobilized on site. By developing an aggressive design schedule, including early and sequential release of piling and mat foundation packages, the construction was allowed to proceed in parallel with the detailed design of the tower superstructure.

Design Codes and Municipal Approvals
Lacking an extensive domestic building code, the Kuwait City Municipality was willing to accept the design of the Al Hamra Tower subject to it being performed in accordance with the provisions of a well-established international building code. The Al Hamra Tower was therefore designed to meet the requirements of the 2003 edition of the International Building Code and all standards referenced therein.

Site Geology
The Al Hamra site is typical for Kuwait City and consists essentially of silty sand with density varying from medium dense to very dense. The formation becomes partially cemented and the degree of cementation seems to increase with depth. Heavily cemented sandstone and siltstone is encountered approximately 75m below grade. The local hydrogeology consists of a phreatic water level at 2m below grade requiring temporary site dewatering during excavation and basement construction. The water level is predominated by rainfall that percolates into the ground with very little runoff to the sea. Evaporation leads to high concentrations of soluble salts, resulting in an aggressive chemical environment for below-grade concrete construction.

Site Seismicity
Kuwait City is known to be located in a region of low seismicity. However, given the low incidence of seismic activity in the area and the short history of significant urban developments, there is little published information about the level of seismicity. When designing a building using the seismic provisions of ASCE 7-02 (referenced from IBC 2003), site specific spectral acceleration parameters are required in order to establish the design seismic base shear. The only reference found for these parameters was TI809-04, Seismic Design for Buildings, published by the US Army Corps of Engineers. This reference is generally used to provide seismic design parameters for in the design of United States government buildings abroad and was also used to determine the seismic loadings for the Al Hamra Tower.
Wind Environment
Due to the relatively low seismicity of the area, the structural design of the Al Hamra Tower was anticipated to be controlled by wind induced forces on the building. However, due to the limited development of super high-rise towers in the vicinity, a suitable wind design criteria was carefully developed. The synoptic wind patterns in the gulf region are the result of the large scale movement of air channeled 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 known 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 measures at the Kuwait airport, the project Wind Engineer BMT Fluid Mechanics Ltd (BMT) established a basic mean hourly wind speed of $23 \text{m.s}^{-1}$ 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.

![Fig. 2 Structure of a Thunderstorm Cell (from ESDU Item 87034)](image)

![Fig. 3 Comparison of Synoptic Wind and Thunderstorm Gust Profile.](image)

![Fig. 4 Wind Tunnel Study Model](image)
FOUNDATION SYSTEM

Pile Phasing
Cognizant of the importance of making design decisions early on in the process, SOM worked collaboratively with the Client (Ajial Real Estate & Entertainment Co.), the Contractor (Ahmadiah Contracting & Trading Co.), the local associate Structural Engineer (Al Jazera Consultants) and the Project Geotechnical Engineer (Consultancy Group Company) to establish the preferred construction type for the project. While still conceptualizing the tower form, knowledge of the plan constraints of the site and that approximately 72 stories of typical office floors would be framed in cast-in-place concrete allowed the team to determine the expected soil bearing stresses under the tower with enough certainty to commit to a foundation system for the tower. Based on these initial calculations and the local knowledge of the Consultancy Group Company (CGC), it was determined that a raft foundation supported on cast-in-place bored piles would be needed. Local construction techniques dictated the maximum pile diameter (1200mm) and soil conditions dictated the closest allowable spacing (3600mm center-to-center), allowing for calculation of the expected pile load demands and for the commencement of a pile load test program.

As the building form took shape and schematic-level superstructure analysis models were developed, it became apparent that the spiraling form of the superstructure would concentrate gravity load to the mat foundation at the west side of the foundation beneath the southwest flared wall, but that very little load would be applied to areas at the north and southwest edges of the mat. This allowed the design team to prioritize the significance of each zone of piles and therefore to release the piles for construction in phases. Ultimately 289 piles were released and constructed in 7 phases working inwards towards the piles beneath the southwest flared wall. The duration of piling works allowed the design of the superstructure to mature and the final mat foundation construction drawings to be completed as work progressed on site.

Fig. 5 On-Site Pile Load Testing

Fig. 6 Pile Construction Phasing
Foundation Analysis and Design

While the local knowledge of the geology in Kuwait City offered by CGC had proven to be invaluable in the preliminary design of the foundation systems for the tower, the design team was concerned that at 400m tall the Al Hamra Tower project might result in geotechnical demands that exceeded the known range that had been established locally during the construction of towers up to 200m tall. Consequently, the Client group contracted the San Francisco office of URS Corporation (URS) to perform a peer review of the recommendations of the Project Geotechnical Engineer in a process including a full three-dimensional non-linear analysis of the soil strata under and around the foundations of the tower. Both analysis approaches were separately used to generate effective soil spring stiffnesses, accounting for the combined effect of mat and pile in each of the zones under the mat. These effective soil spring stiffnesses were used by SOM to analyze and design the mat foundation as well as to verify distribution of load to the piles. Unfortunately, the estimations of effective stiffness recommended by CGC and URS differed significantly, primarily as a result of differing interpretations of the extent to which group action between the piles would develop. Ultimately resolution of this impasse was achieved by SOM designing each component of the mat foundation and piles for the controlling results of four analysis cases: the lower and upper bound effective spring stiffnesses predicted by CGC, and the equivalent values predicted by URS. This approach, though requiring a greater engineering design effort, did not greatly increase the use of materials since between sets of effective stiffness data all values tended to change by a similar ratio. Consequently, no significant redistribution of load occurred and mat forces and pile loads remained relatively constant.

![Fig. 7 Bearing Pressure (MPa) CGC Case 1](image1)

![Fig. 8 Bearing Pressure (MPa) URS Case 1](image2)

![Fig. 9 Deflected Shape (mm) CGC Case 1](image3)

![Fig. 10 Deflected Shape (mm) URS Case 1](image4)
The major differences in results were due to the URS three dimensional non-linear analysis model of the soil predicting significant group behavior. With soil being dragged down, the skin resistance of the piles was lost for all but the perimeter piles. Consequently, the perimeter piles were stiffer and carried greater loads. The CGC analysis approach resulted in all the piles having similar loads and therefore did not show a load concentration at the perimeter. Pile group action also resulted in ‘mat-like’ behavior of the system at the bottom of pile elevation in the URS analysis runs, thus predicting significantly greater values of settlement.

The final design of the raft foundation was for a 4.0m thick raft approximately 70m by 60m in plan dimension with an additional 1.6m thick triangular section of raft approximately 24m by 12m in a region to the north, beyond the footprint of the tower. The tower raft is supported by 289 piles each 1200mm in diameter and varying in length from 20.0m to 27.0m measured from the bottom of raft. The design concrete compressive strength of the raft was 50MPa (cube compressive strength), and varied in the piles from 55MPa to 80MPa (56 day cube compressive strength).

Concrete and Reinforcement Durability
To provide an appropriate level of durability to the sub-grade concrete construction, the effect of the corrosive environment on both the concrete and concrete reinforcement was considered. Moderate heat of hydration and moderate sulfate resistant cement (Type II) was specified for the subgrade construction. This cement was determined to be the most appropriate compromise between the corrosion resistance requirements and the need to control the curing temperature of the 4.0m thick mat in the hot desert environment in Kuwait. The subgrade construction was further protected by a complete waterproof membrane on the external surfaces of the raft and the foundation walls. The membrane was additionally required to mitigate against the infiltration of hydrogen sulfide gas into the completed basement structure. The piles, the bottom layers in the raft, and the outer curtains in the foundation walls were all designed to be reinforced using corrosion-resistant reinforcement manufactured by MMFX technologies corp. The reinforcement was further protected in the cast-in-place augered piles, where the use of a membrane was impractical, by specifying a high-density, low-permeability mix specifically designed to limit sulfate attack and minimize the ingress of corrosive ground water. Clear cover requirements of ACI-318M were also increased to 100mm to further protect the pile reinforcement. As a Contractor substitution the corrosion resistant reinforcement was ultimately eliminated from the project, replaced by an engineered cathodic protection system.

Raft Construction
The raft foundation was poured in 15 separate pours, the size of each limited by municipal traffic regulations restricting the delivery of concrete to the weekends and the local production capabilities of the local concrete batch plants for delivery within each weekend window. As a result, the raft foundation was poured over a period of approximately 4 months. The segmented approach to the raft pour was also beneficial in limiting the peak concrete curing temperature; although insulated to prevent damaging temperature differentials building up near the concrete surface, the increased surface area of the sides in addition to the top of each pour allowed the controlled release heat at a higher rate, nominally reducing the peak curing temperatures. Concrete curing temperatures were further minimized by specifying a high volume fly-ash cement replacement concrete mix.
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 suggested 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 center of 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.
**Torsional Response due 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.

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**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 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 described in the above section. 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 frames are typically 800mm wide by 600mm deep and are poured with the floor framing using 40MPa concrete (cube compressive strength).

Gravity Force Resisting System
The gravity force resisting system for the Al Hamra Tower is significantly more complicated and required more in depth consideration that for a conventional tower design. Reinforced concrete 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 needed to be used that 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).

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.

**Creep and Shrinkage and Construction Corrections**

The design of any super high-rise concrete tower is complicated by the time-dependent shortening of concrete due to creep and shrinkage effects both during and well after the completion of construction. Member internal forces are also greatly affected by the sequence of construction. A traditional finite element analysis of a small building models a one step construction process where the structure is instantaneously constructed and gravity loads are applied only to the completed form. This simplification is inappropriate in the design of a tall building as it can greatly overestimate the internal forces in an element connecting two vertical elements that may experience different elastic shortening. Once gravity is applied to the completed structure, a connecting element between core wall and perimeter column would suddenly be required to deform to maintain displacement compatibility with the elastic shortening taking place over the full height of the core and the column, resulting in flexural and shear forces in the connecting element. In reality all of the elastic shortening of these elements would have occurred at the time of casting the connecting element, so no internal force should be in the connecting element at the time it is cast. To more accurately consider this effect all of the strength design analysis models considered a construction sequence analysis applying the self-weight of the structure in three story increments. The analysis model is initially only three stories tall, gravity loads are applied and internal forces calculated. Then the next three stories are added to the analysis model. The model is now six stories tall but gravity loads are only applied to the newly added portions of the model. Internal forces are calculated for all six stories and added to the forces resulting from the previous analysis step. In this way the element internal forces for each element are ultimately calculated based upon all the elastic demands that are placed on the element after that element is added to the analysis model (constructed). The construction sequence analysis load-case, while taking into account the extent of prior construction at the time of casting an element, still only considers the elastic component of concrete shortening. To study the non-linear creep and shrinkage components a series of non-linear column shortening studies were undertaken, using
theoretical time-dependant models of the creep and shrinkage of concrete and considering an anticipated construction schedule. Following this procedure the absolute shortening of various perimeter columns and zones of the shear wall core were established for a series of time steps from the start of construction until approximately ten years after completion of construction. As creep is a phenomenon of strain at constant stress, the force levels in the vertical elements under study do not change as this process takes place, however, this information is important in considering the impact that differential shortening may have on any connecting elements and evaluating if the final condition of the structure meets the dimensional requirements for the project. In the case of the Al Hamra Tower, a construction corrections program was established which included two major components. Vertical corrections were established both for the core and the perimeter frame. Typically the core at each story is poured to a target elevation which although taller than the design story height at the date of pour, is calculated to shorten down to the design story height at a target date approximately one year after topping out of the tower structure. Additionally the perimeter frame is further corrected and poured higher than the core at each level. This additional offset is to ensure that after the predicted differential shortening occurs, the floor slabs are level in the final condition. The second major part of the corrections program was a program of rotational corrections. Even the simplest of correction programs, building each level to the design elevation and rotation, would require some amount of clockwise twisting of the formwork to correct for the counterclockwise twist of the partly constructed tower resulting from the previous pour. Therefore it was determined that a capacity needed to be built into the formwork system to allow for rotational corrections, and given that capacity, it should also be leveraged to preemptively correct for the anticipated elastic and a portion of the inelastic rotations to result in a final condition as close to the design condition as possible. The rotational corrections program was communicated to the Contractor as a target story rotation relative to the design axis of the tower at the time of casting each level. The rotational preemptive correction has the additional benefit of reducing the second order P-delta effects in the tower, likewise reducing the internal element forces.

Element Design
All structural elements in the Al Hamra Tower were designed in accordance with the 2003 edition of the International Building Code and all codes referenced therein. ACI 318-02M was used for concrete design detailing, and AISC LRFD (1999) for steel design. All column and wall elements were designed to take into account their three-dimensional moment-axial interaction surfaces, adjusted to account for slenderness and buckling effects through moment magnification. Torsion and shear were designed in conjunction with axial demands to give more representative design strengths. Automated spreadsheets were used for the design of each element type, liked to databases containing the element internal force results of the parallel analysis models. Programming scripts were routinely used to perform element design checks on thousands of elements at a time. This combination of traditional engineering and computer programming skills allowed the design team to maintain an updated set of substantially completed calculations as the design of the tower evolved – essential considering both the complexity of the tower design and the overlay between design and construction of the tower that was required by the fast-track schedule.
LOBBY LAMELLA STRUCTURE

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 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.

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.

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.

Fig. 14  Lamella Buckling Analysis Models
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.

**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.

**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.
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Structural Engineer: Skidmore, Owings & Merrill LLP; San Francisco
MEP Engineer: Skidmore, Owings & Merrill LLP; Chicago

Architect of Record: Al Jazera Consultants, Kuwait
Engineer of Record: Al Jazera Consultants, Kuwait

Geotech. Engineer: Consultancy Group Company, Beirut
Wind Engineer: BMT Fluid Mechanics, London

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