CTBUH
Research Paper

ctbuh.org/papers

Title:	**Jin Mao Tower's Influence on China's New Innovative Tall Buildings**
Authors:	Mark Sarkisian, Partner, Skidmore, Owings & Merrill LLP Neville John Mathias, Associate Partner, Skidmore, Owings & Merrill LLP Eric Long, Associate, Skidmore, Owings & Merrill LLP Aaron Mazeika, Associate, Skidmore, Owings & Merrill LLP John Gordon, Associate, Skidmore, Owings & Merrill LLP Jean-Pierre Chakar, Skidmore, Owings & Merrill LLP
Subjects:	Architectural/Design Construction Structural Engineering
Keyword:	Systems
Publication Date:	2006
Original Publication:	2006 International Seminar of Design and Construction Technologies of Super High-Rise Buildings
Paper Type:	1. **Book chapter/Part chapter** 2. Journal paper 3. Conference proceeding 4. Unpublished conference paper 5. Magazine article 6. Unpublished

© Council on Tall Buildings and Urban Habitat / Mark Sarkisian; Neville John Mathias; Eric Long; Aaron Mazeika; John Gordon; Jean-Pierre Chakar

2006 Shanghai International Seminar of Design and
Construction Technologies of Super High-Rise Buildings
15&16 May 2006

Jin Mao Tower's Influence on China's New Innovative Tall Buildings

Mark Sarkisian, PE, SE, Partner
Neville Mathias, PE, SE, Associate Partner
Eric Long, PE, Associate
Aaron Mazeika, PE, Associate
John Gordon, PE, Associate
Jean-Pierre Chakar, PE

Skidmore, Owings & Merrill LLP (SOM)
San Francisco, California, USA

Abstract

The structural system for the Jin Mao Tower has influenced the design of other tall buildings in China through the use of innovative concepts that carefully integrate architectural and structural systems. The Jin Mao Tower not only incorporated composite design into an ultra-tall structure, but introduced mechanized building concepts into the building behavior and construction. The design approach emphasized integration of innovative structural systems and construction methods while respecting the architectural design intent. These innovations spawned other ideas that maximize building performance when subjected to extreme wind and seismic loadings while reducing material quantities and therefore cost.

Infinity Column

The Jinao Tower, currently under construction in Nanjing utilizes a tube-in-tube reinforced concrete system combined with perimeter diagonal steel bracing. Located outside of the perimeter reinforced concrete frame, the diagonal brace system allows for the core stiffness to be minimized and allows for an open atrium on hotel floors located in the top half of the 232 meter-tall building. The diagonal bracing concept resulted in a 55% reduction in concrete material quantities for the inner core, a 40% reduction in concrete material quantities for the lateral system and a 20% reduction in overall concrete material quantities in the overall structure. Advanced non-linear finite element modeling techniques were used to substantiate the conceived design. The perimeter bracing system is located within a double exterior wall system that is used naturally control heating and cooling temperature demands on the building.

Screen Frames

The Goldfield International Garden Project – Beijing incorporates expressed reinforced concrete stiffening frames into multi-story moment-resisting mega-frames for the 150 meter-tall and 100 meter-tall towers. The stiffening frames or screens provide an unsymmetrical yet efficient lateral stiffening system. Special considerations for construction sequence and tuned lateral stiffness were evaluated. Advanced non-linear push-over analyses were used to define the behavior of the structure when subjected to wind and seismic loads.

The Rocker

The 100 meter-tall composite China Poly – Beijing building project includes the world's largest enclosed atrium utilizing a 90 meter-high by 60 meter-long cable net. A museum, designed to be the centerpiece of the atrium, is suspended above the lobby floor by a diagonal cable-stayed system. This cable system is decoupled from the primary building lateral load resisting frame by using a "rocker" or reverse pulley system. This system prevents lateral load forces due to potentially strong ground shaking from being attracted these diagonal members while providing support of the atrium's cable net.

Sunshades

The China Poly Pazhou Project – Guangzhou, also known as Poly International Plaza, consisting of two 150 meter-tall towers, combine diagonal composite members and reinforced concrete buttress walls to form lateral load resisting screen frames on the south façade of the tower. The screen frames act to control building temperatures by providing shading to the severe south solar exposure. These buttresses are then interconnected with outrigger trusses at two 2-story levels within the tower. Steel diagonal bracing is used along the narrow faces of the towers. The towers are exceptionally slender, with aspect ratios (ratio of building height to width) of over 8 to 1. The towers are designed to resist typhoons winds as well as moderate seismicity.

The Perfect Tube

A fine diagonal structural mesh is incorporated into a rotating tubular form in the Jinling Hotel proposed for Nanjing, China. The perimeter structural mesh, conceived to consist either of structural steel or reinforced concrete combined with the central core to form the structure for this mixed-use 320 meter-tall tower. The form of the structure changes along its height with floor plans directly responding to use. The base of the building is essentially square in office areas with the plan opening on mid-rise and high-rise floors allowing residential and hotel spaces to have perimeter exposure while being located around the central service core. The structural concept achieves essentially 100% tubular efficiency since building deformations are almost entirely controlled by axial deformations of mesh elements.

1 JIN MAO TOWER, SHANGHAI, THE PEOPLE'S REPUBLIC OF CHINA

1.1 *Introduction*

The site for the Jin Mao Tower located in new Pudong development district of Shanghai, The People's Republic of China, is not naturally conducive to accepting a tall building structure, especially China's tallest. Soil conditions are very poor since the site is located in the flood plain of the Yangtze River, the permanent water table is just below grade, typhoon wind conditions exist, and moderate earthquakes are possible. Unique structural engineering solutions were incorporated into the design with the combined use of structural steel and reinforced concrete; solutions which not only overcame the adverse site conditions but also produced a very efficient structure for this ultra-tall building. These solutions were necessary to accommodate the mixed-use program that included a five star hotel, Class A office, parking, and retail uses.

1.2 *Structural System*

The superstructure for the 421 meter-tall, 88-story Jin Mao Tower consists of a mixed use of structural steel and reinforced concrete with major structural members composed of both structural steel and reinforced concrete (composite). Thirty-six (36) stories of hotel spaces exist over 52 stories of office space. The structure was developed by the China Shanghai Foreign Trade Co., Ltd. and constructed by the Shanghai Jin Mao Contractors, a consortium of the Shanghai Construction Group; Obayashi Corp., Toyko: Campenon Bernard SGE, France; and Chevalier, Hong Kong. The structure was topped-out in August 1997 and fully completed in August 1998. The structure is the tallest in China and the fourth tallest in the world behind the Taipei 101, the Petronas Towers in Kuala Lumpur, Malaysia and the Sears Tower in Chicago, Illinois, USA.

The primary components of the lateral system for this slender Tower, with an overall aspect ratio of 7:1 to the top occupied floor and an overall aspect ratio of 8:1 to the top of the spire, include a central reinforced concrete core linked to exterior composite mega-columns by structural steel outrigger trusses. The central core houses the primary building service functions, including elevators, mechanical fan rooms for HVAC services, and washrooms. The octagon-shaped core is nominally 27 m deep with flanges varying in thickness from 850 mm at the top of foundations to 450 mm at Level 87 with concrete strength varying from C60 to C40. Four (4) - 450 mm thick interconnecting core web walls exist throughout the office levels with no web walls on the hotel levels, creating an atrium with a total height of 205 m which leads into the spire. The composite mega-columns vary in cross-section from 1500 mm x 5000 mm at the top of foundations to 1000 mm x 3500 mm at Level 87. Concrete strengths vary from C60 at the lowest floors to C40 at the highest floors.

Structural steel outrigger trusses interconnect the central reinforced concrete core and the composite mega-columns at three 2-story tall levels. The interconnection occurs between Levels 24 & 26, Levels 51 & 53, and Levels 85 & 87. The outrigger trusses between Levels 85 & 87 engage the 3-dimensional structural steel cap truss system. The cap truss system which frames the top of the building between Level 87 and the spire is used to span over the open core, support the gravity load of heavy mechanical spaces, engage the structural steel spire, and resist lateral loads above the top of the central core wall / composite mega-column system.

In addition to resisting lateral loads, the central reinforced concrete core wall and the composite mega-columns carry gravity loads. Eight (8) built-up structural steel mega-columns also carry gravity loads and composite structural steel wide-flanged beams and built-up trusses are used to frame typical floors. The floor framing elements are typically spaced at 4.5 m on-center with a composite metal deck slab (75 mm metal deck topped with 80 mm of normal weight concrete) framing between the steel members. Figure 1a illustrates the components of the superstructure.

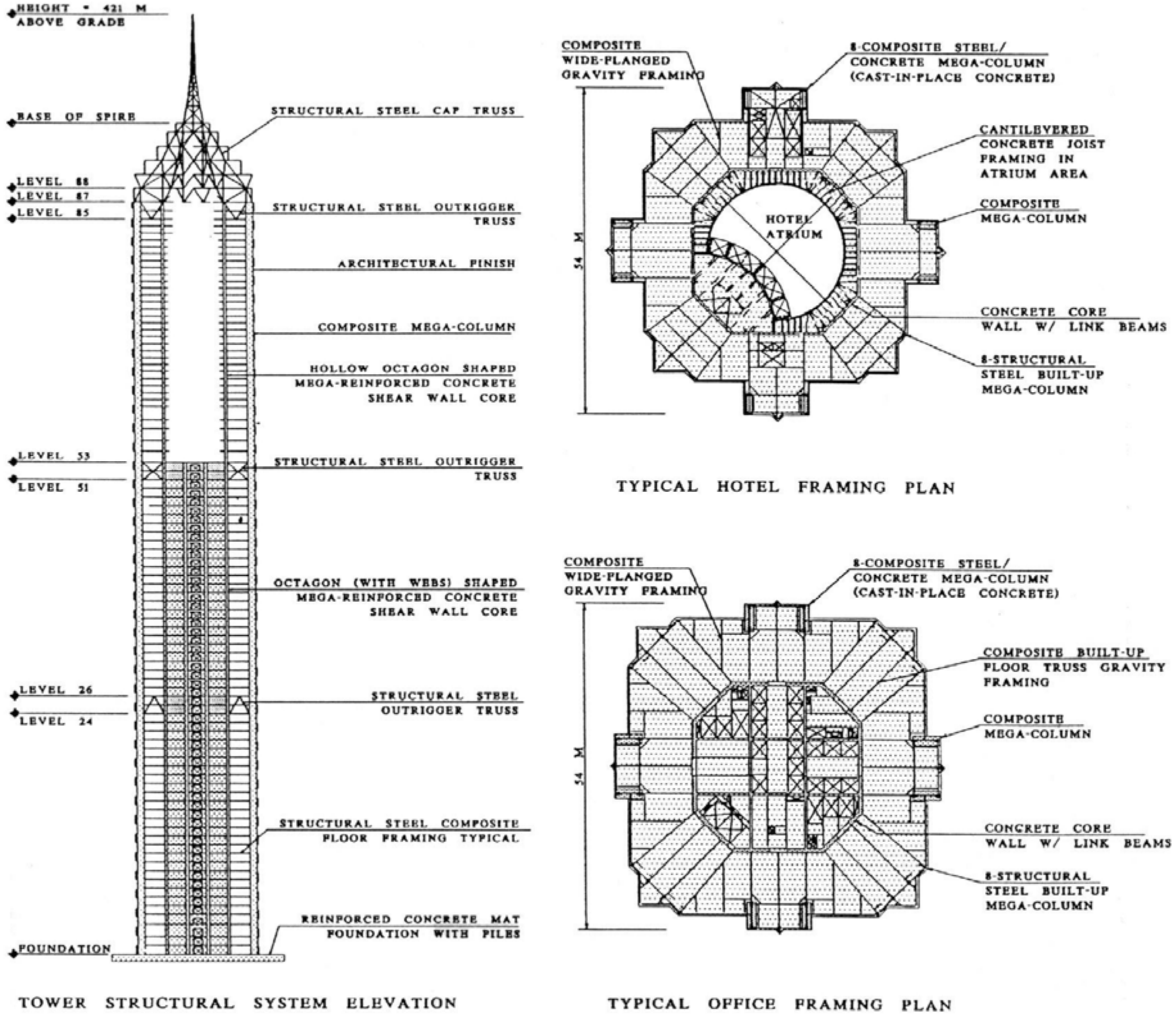


Figure 1a. Structural system elevation and framing plans

1.3 *Soil Conditions*

Because of extremely poor upper-strata soil conditions, deep, high-capacity structural steel pipe piles are required to transfer the superstructure loads to the soil by friction. Open structural steel pipe piles are 65 m long with a tip elevation 80 m from existing grade. The tips of the piles rest in very stiff sand and are the deepest ever attempted in China. Pipe piles were installed in three (3) approximately equal segments, having a wall thickness of 20 mm, and having an individual design pile capacity of 750 tonnes. Piles were driven from grade with 15 m long followers before any site retention system construction or excavation had commenced. The pipe piles are typically spaced at 2.7 m on-center under the core and composite mega-columns with a 3.0 m spacing under the other areas. The piles are capped with a 4 m thick reinforced concrete mat comprised of 13,500 m3 of C50 concrete. The mat was poured continuously, without any cold joints, over a 48 hour period. Concrete temperature was controlled by an internal cooling pipe system with insulating straw blankets used on the top surface to control temperature variations through the depth of the mat and to control cracking.

A reinforced concrete slurry system was designed and constructed around the entire perimeter of the site (0.75 kilometer). The thickness of the slurry wall is 1 m with a concrete design strength of C40 and depth of 33 m. The slurry wall bears on moderately stiff, impervious clay. The slurry wall acts as a temporary retention system wall, a permanent foundation wall, and a temporary / permanent water cut-off system. A tieback ground anchor system was designed and successfully tested to provide lateral support of the slurry wall during construction, however, the contractor chose to construct a locally accepted reinforced concrete cross-lot bracing system for the three (3) full basement levels which extended approximately 15 m below grade. The permanent ground water table is within 1 m of existing grade. Based on the site conditions and the slurry wall depth, a sub-soil drainage system was designed to carry 18.5 liter/sec of water. An overall description of the foundation system is shown in figure 1b.

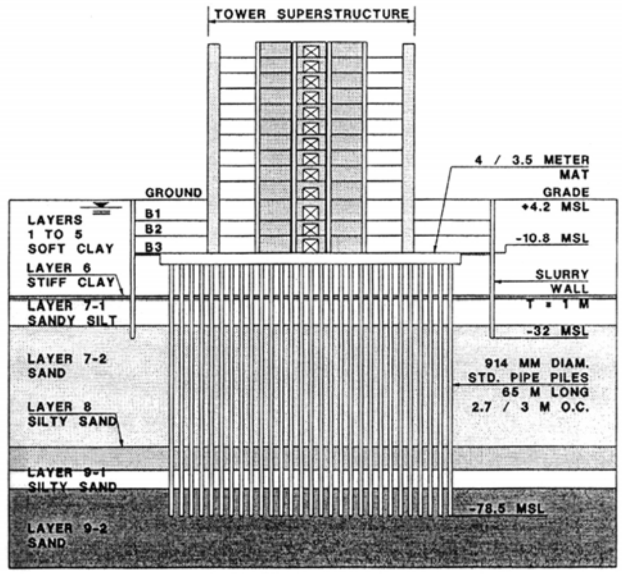


Figure 1b. Tower foundation systems

1.4 *Extreme winds*

Typhoon winds as well as strong extratropical winds exist in the local Shanghai environment. Multiple analytical and physical testing techniques were used to evaluate the behavior of the Tower. Since ultra-tall structures had not been previously constructed in China, the Chinese wind design code did not address structures taller than 160 m. Therefore, code requirements were extrapolated for the Tower and wind tunnel studies were performed to confirm Code extrapolations and to study the actual, “rational” local wind climate. Wind tunnel studies, performed under the direction of Dr. Nicholas Isyumov at the University of Western Ontario in conjunction with the Shanghai Climate Center, were conducted for the building located in the existing site condition and considering the future master plan development termed the “developed Pudong” condition. The existing site context essentially consisted of low-rise buildings (3-5 stories in height) with the fully “developed Pudong” environment consisting of 30 - 50 story buildings surrounding the Jin Mao Tower with two (2) ultra-tall towers located within 300 m of Jin Mao. Wind tunnel investigation included a local climate study, construction of proximity models, a force balance test, an aeroelastic test, an exterior pressure test, and a pedestrian-level wind study. All tests considered both typhoon and extratropical winds as well as the existing and “developed Pudong” site conditions.

The final design of the Tower considered both the People’s Republic of China Building Code as well as the “rational” wind tunnel studies. Strength design for all lateral load-resisting components is based on the Code-defined 100-year return wind with a basic wind speed of 33 m/s for a 10 minute average time at 10 m above grade. The basic wind speed corresponds to a design wind pressure for the Tower of approximately 0.7 kPa at the bottom of the building and 3.5 kPa at the top of the building. Results from the wind tunnel studies, considering the existing site condition and the “developed Pudong” condition as well as extratropical and typhoon winds confirmed that the Chinese Code requirements for design were conservative.

Serviceability design, including the evaluation of building drift and acceleration, was based on the “rational” wind tunnel study results. Wind tunnel studies were performed for 1-year, 10-year, 30-year, 50-year and 100-year return periods. The studies considered the actual characteristics of the structure. The fundamental translational periods of the structure are 5.7 seconds in each principal direction and the fundamental torsional period is 2.5 seconds. The overall building drift, with comparable inter-story drifts, for the 50-year return wind with 2.5% structural damping is H/1142 for the existing site condition and H/857 for the “developed Pudong” condition. It was determined that the two (2) ultra-tall structures proposed to be located near the Jin Mao Tower would have a significant effect on the dynamic behavior resulting in significantly higher effective structural design pressures. Building drifts are well within the internationally accepted building drift of H/500. Considering 1.5% structural damping and a 10-year return period, the expected building acceleration ranged from 9 - 13 milli-g’s for the top floor of the occupied hotel zone. In addition, expected building acceleration ranged from 3 - 5 milli-g’s for a 1-year return period considering 1.5% structural damping. The internationally acceptable accelerations for a hotel structure are 15 - 20 milli-g’s for a 10-year return period and 7 - 10 milli-g’s for a 1-year return period. Because of the favorable serviceability behavior of the building, the passive characteristics alone could be used to control dynamic behavior with no additional mechanical damping required.

Wind tunnel study results determined that the Code requirements for lateral load design was equivalent to a 3000-year return wind. The overall building drift based on this conservative wind loading is H/575 which also meets internationally acceptable standards for drift.

1.5 *Seismicity*

The approach for evaluating seismic loadings for the Jin Mao Tower considers both Chinese Code-defined seismic criteria and actual site-specific geological, tectonic, seismological and soil characteristics. Actual on-site field sampling of the soil strata and engineering evaluations were performed by Woodward-Clyde Consultants, the Shanghai Institute of Geotechnical Investigation and Surveying, and the Shanghai Seismological Bureau.

All lateral load resisting systems, including all individual members, were designed to accommodate forces generated from the Chinese Code-defined response spectrum as well as site specific response spectrums. Extreme event site-specific time history acceleration records (10% probability of occurrence in a 100-year return period) were used in time history analyses to study the dynamic behavior of key structural elements including the composite mega-columns, the central core, and the outrigger trusses.

The site specific response spectrums used to describe the Tower’s dynamic behavior included analyses for a most probable earthquake with a 63% probability of occurrence in a 50-year return period and a most credible earthquake with a 10% probability of occurrence in a 100-year return period. In addition, the Tower was evaluated using a 3-dimensional dynamic time history analysis for a most credible earthquake with a 10% probability of occurrence in a 100-year return period.

In all cases, the Chinese-defined code wind requirements governed the overall building behavior and strength design; however, special considerations were given to the outrigger trusses and their connections. In all design cases, these structural steel trusses were designed to remain elastic.

1.6 *Unique structural engineering solutions*

The structural design for the Jin Mao Tower created an opportunity to develop unique structural engineering solutions. These solutions included the practical development of theoretical concepts, unusual detailing of large structural building components, and comprehensive monitoring of the in-place structure.

The overall structural system utilizes fundamental physics to resist lateral loads. The slender cantilevering reinforced concrete central core is braced by the outrigger trusses which act as levers to engage perimeter composite mega-columns, maximizing the overall structural depth. The overall structural redundancy is limited by engaging only four (4) composite mega-columns in each primary direction. Structural materials are strategically placed to balance the applied lateral loads with forces due to gravity. Very little structural material premiums were realized because of the structural system used. Lateral system premiums essentially related to material required for the outrigger trusses only without measurable structural material premiums required for central core wall and composite mega-column elements. The combination of structural elements provides a structural system with 75% cantilever efficiency.

Even after equalizing the stress level within the central core and composite mega-columns, the expected relative shortening between the interconnected central core and composite mega-columns was large. By calculation, considering long-term creep, shrinkage, and elastic shortening, the expected relative movement between these elements at Levels 24-26 was as much as 50 mm. The magnitude of relative movement would have induced extremely high stresses into the stiff outrigger truss members weighing as much as 3280 kg/m. Therefore, structural steel pins with diameters up to 250 mm were detailed into the outrigger truss system (see figure 1c). These pins were installed into circular holes in horizontal members and slots in diagonal members to allow the outrigger trusses to act as free moving mechanisms for a long period during construction. This allowed a majority of the relative movement to occur free of restrain, therefore, free of stress. After a long period of time, high strength bolts were installed into the outrigger truss connections for the final service condition of the lateral load resisting system. The expected relative movement after the final bolting was performed was a maximum of 15 mm at Levels 24-26. Considering the flexibility of the long composite mega-columns, the final forces attracted to the trusses did not appreciably increase the member and connection sizes.

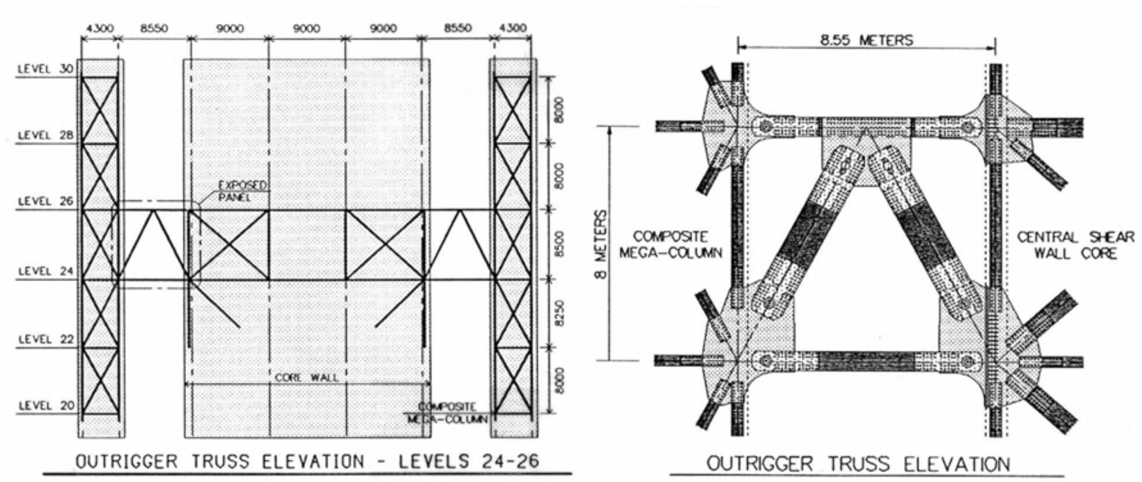


Figure 1c. Elevation and detail of outrigger truss system

A comprehensive structural survey and monitoring program was designed and implemented into the Jin Mao Tower. Extensometers were placed on the reinforced concrete central core and on the reinforced concrete of the composite mega-columns. In addition, strain gages were placed on the built-up structural steel mega-columns as well as on the wide-flanged structural steel columns location within the concrete encasement for the composite mega-columns. Sample results of measured strain versus calculated strain are shown in figure 1d. In addition to the gaging of the superstructure, the mat was periodically surveyed for long-term settlement. The mat foundation system under the Tower was initially surveyed just after pour completion in October 1995 and was periodically surveyed after placement. Based on a sub-structure / soil analysis, the expected maximum long-term Tower mat settlement is 75 mm. The final measured settlement of the Tower was very close to 75 mm. Tower mat settlement results are shown in figure 1e. Laser surveying techniques were used for both lateral and vertical building alignment. Floor levels of the structure were typically built to drawing design elevation, compensating for creep, shrinkage, and elastic shortening which occurred during construction. Lateral position of the Tower was constantly monitored from off-site benchmarks and was found to be well within acceptable tolerances.

1.7 *Conclusions*

Incorporating fundamental structural engineering concepts into the final design of the Jin Mao Tower lead to a solution which not only addressed the adverse site conditions but also provided an efficient final design. The final structural quantities included the following for the Tower superstructure from the top of the foundation to the top of the spire (gross framed area = 205,000 m2):

Structural Concrete	0.37 m3/m2
Reinforcing Steel	30.4 kg/m2
Structural Steel	73.2 kg/m2

Data from the as-built structure subjected to actual imposed loads was correlated with theoretical results. This comparison proved to be invaluable for the correlation of results in this ultra-tall occupied structure.

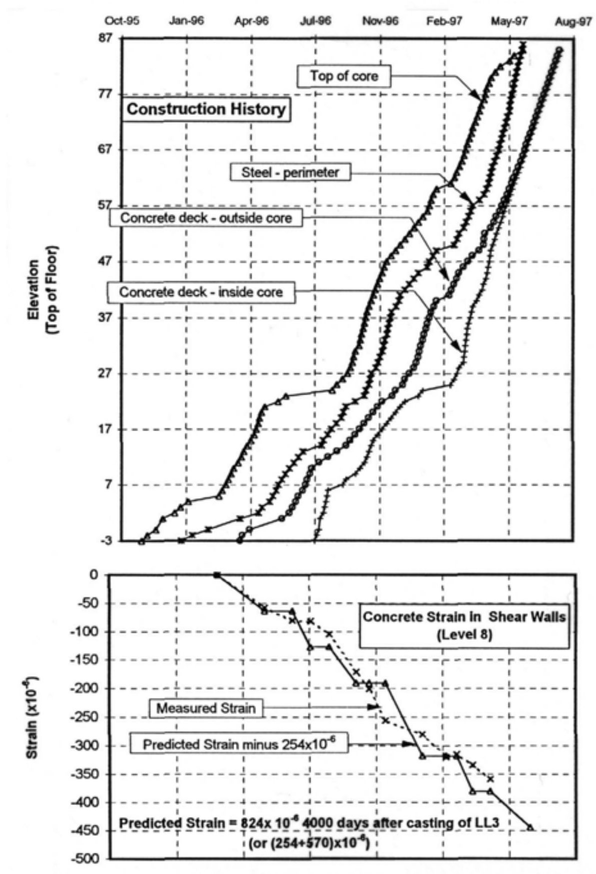


Figure 1d. Comparison of measured strain versus predicted strain in shear walls (level 8)

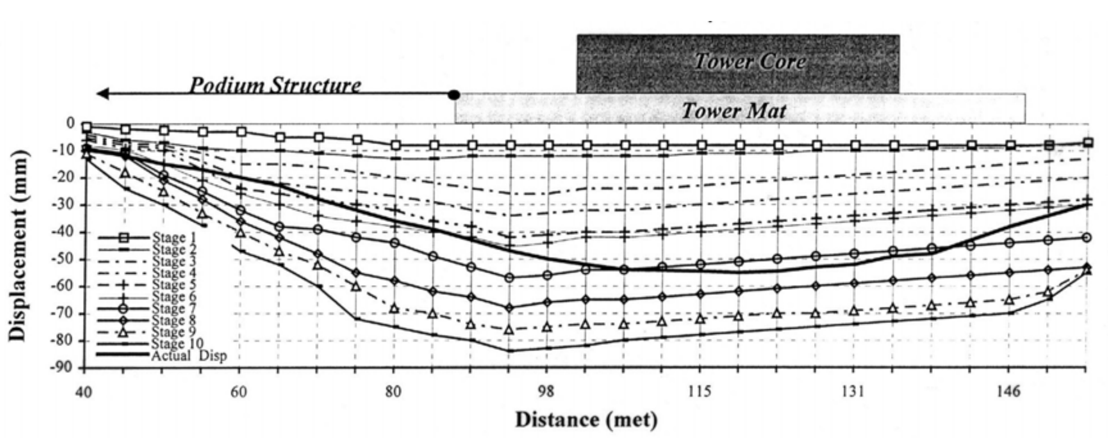


Figure 1e. Comparison of estimated versus actual tower mat settlement

2 INFINITY COLUMN
JINAO TOWER, NANJING, THE PEOPLE'S REPUBLIC OF CHINA

2.1 *Introduction*

The Hexi (west of the river) masterplan called for a pair of towers with simple, pure forms accommodating office and hotel uses and forming a gateway to a large neighboring park. The 60-story, 232-meter tall Jinao Tower serves as an easily identifiable iconic form. A double-wall façade provides solar shading and creates a climatic chamber of air, offering excellent insulation in the hot summer months. Vented openings in the outer exterior wall allowing wind pressure to draw built-up heat out of the cavity, lowering temperatures along the interior exterior wall. The structural system, an "infinite column" (named in homage to Brancusi's Endless column sculpture in Romania), consists of a tube-in-tube system of reinforced concrete with a perimeter braced steel frame. The central core consists of a hybrid reinforced concrete shear wall-frame, while the perimeter consists of tubular frame. Introducing a wrapping diagonal steel brace on each side of the structure (outside of the tubular plane and between the double wall) resulted in a 40% reduction in concrete and rebar in the lateral load resisting system and a 20% reduction in concrete and rebar for the overall building structure. The braces, only 500 mm in diameter with a typical wall thickness of 25 mm, efficiently direct lateral loads from the superstructure to the foundation. Since less material was required for each lift of the structure, the construction time is reduced from 6 days per story to 4.5 days per story.

The structure responds harmoniously to the building use. The central core wall was not designed as a traditional shear wall limiting the number of openings while attempting to have continuous walls throughout. The walls were specifically punched to create increased flexibility of use while adjusting stiffness properties. In the lower portion or office area of the tower this allowed easier access to the core service areas and in the upper portion or the hotel this allowed for open atrium spaces. The reduction in central core stiffness allowed for lateral forces to be efficiently shared with exterior steel braces. Up to 60% of the lateral forces due to wind and seismic are carried by the exterior steel braces alone.

Figure 2a. The infinite column

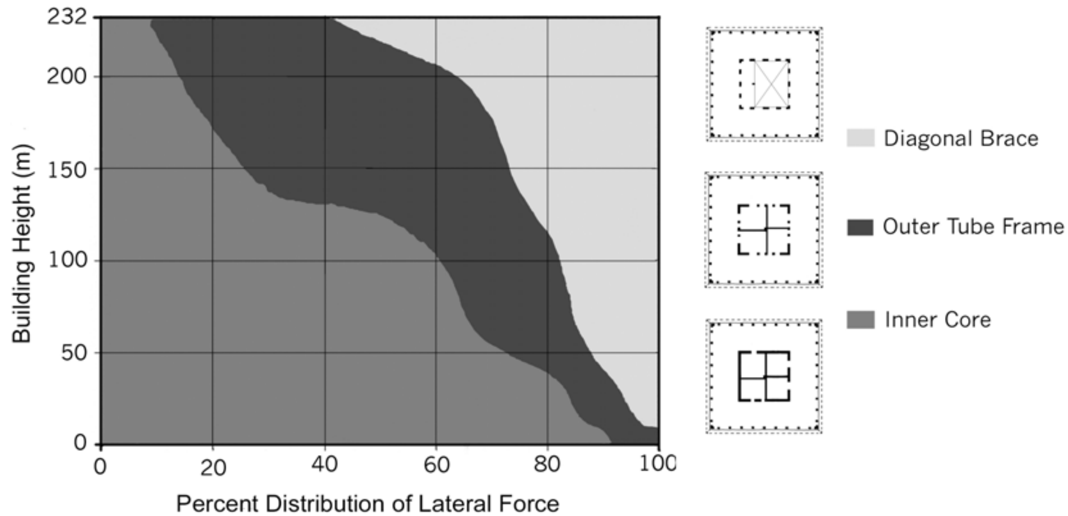


Figure 2b. Percent distribution of lateral force vs. building height

2.2 *Structural System*

The structural system conceived for the Jinao Tower was developed to maximize structural efficiency and minimize material quantities while integrating directly with the architecture and maximizing column-free lease spans. When construction is complete in 2008, the Tower will be occupied with both office and hotel spaces. The podium area around the Tower will be occupied by residential and retail spaces.

The primary structural system used to resist lateral wind and earthquake loads is located in the central core area of the building where elevators and back-of-house functions exist and at the perimeter of the building where maximum structural resistance to overturning can be achieved. The basis for the structural system is a punched shear wall – tubular / braced frame concept combining a central reinforced concrete core, a perimeter reinforced concrete moment-resisting frame, and an exterior diagonal structural steel braced frame. The diagonal bracing is essential to maximizing structural efficiency while the reinforced concrete core and perimeter frame provides a high level of ductility in a seismic event.

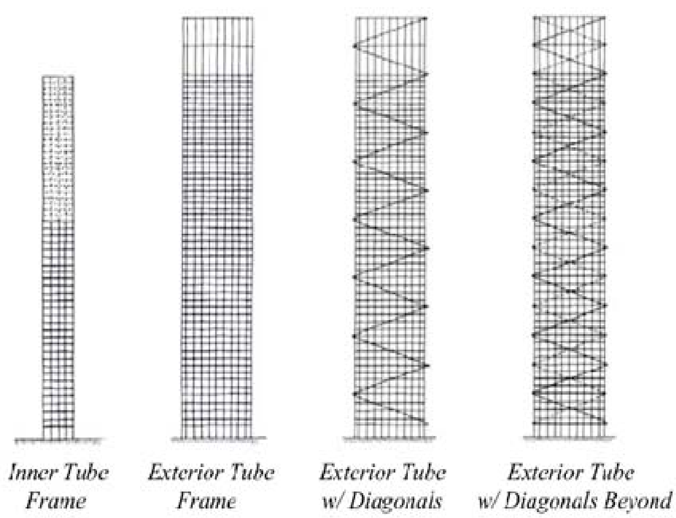


Figure 2c. Structural system elevations

2.3 *Superstructure*

The superstructure typically consists of conventional reinforced concrete for the Tower. Composite construction is used for the perimeter moment frame columns in the lower one third of the building to minimize size and impact to the office lease space. Structural steel pipe is used the exterior diagonal braced frame.

The lateral system for the Tower consists of punched shear wall – tubular / braced frame concept combining a central reinforced concrete core, a perimeter reinforced concrete moment-resisting frame, and an exterior diagonal structural steel braced frame. Long-span reinforced concrete framing clear spans between the central core wall and perimeter frames. The clear span allows for column-free interior spaces while placing all gravity loads on lateral load resisting elements. This gravity load eliminates any tensile loads within the central core or the perimeter frames caused by lateral loads.

The reinforced concrete core includes interior web walls at the lower portion of the Tower with those walls eliminated in upper portions of the Tower as structural demand decreases. Open core wall areas are used for atria. The reinforced concrete core wall thickness for perimeter flange components varies from 900 mm at the base of the building to 600 mm at the top. Web walls are typically 450 mm thick. Concrete strength varies from C60 at the base to C40 at the top.

Columns within the perimeter frames are typically spaced at 4.5 m. Reinforced concrete columns typically vary from 900 mm x 900 mm to 600 mm x 600 mm. The tubular frame beam members are typically 600 mm. The concrete strength for the perimeter frames varies from C60 to C40. Perimeter columns are designed compositely with structural steel for approximately one third of the building height and all corner columns that interconnect with the bracing system are composite for the entire height of the Tower.

The gravity system for the Tower typically consists of reinforced concrete beams and slabs. Framing beams are spaced on the same module at the exterior columns with the spacing of the framing 4.5 meters on-center. Conventional reinforced concrete slabs span between the beams. Concrete is normal weight with strength of C40. The floor framing depth is typically 750 mm – 850 mm deep with a slab thickness of 135 mm – 175 mm. Conventional reinforced concrete framing is also used in the core wall areas. The typical depth of the framing in the core area is 600 mm.

In additional to resisting lateral loads, the reinforced concrete core wall and perimeter frames are primary gravity load resisting elements.

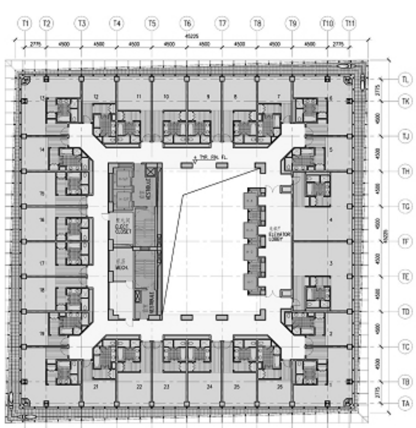


Figure 2d. Typical hotel level floor plan

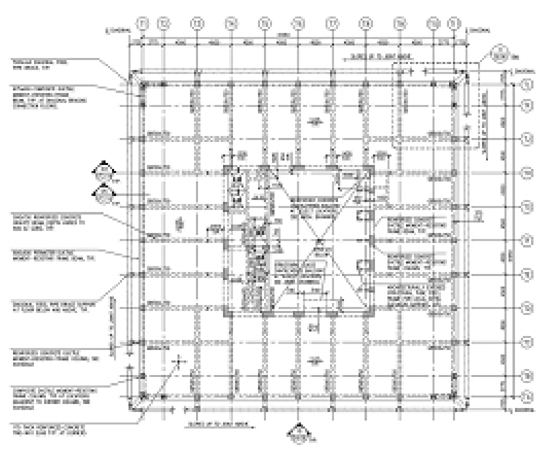


Figure 2e. Typical hotel level framing plan

2.4 *Foundations*

The foundation system for the Tower will consist of a conventionally reinforced concrete mat supported by caissons / piles. The mat thickness is 2.75 meters thick and uses a concrete strength of C50. Under the Tower footprint, hand-dug cast-in-place concrete caissons support the mat. The caisson diameters are typically 1200 mm. The foundation system for the podium / low-rise areas consists of reinforced concrete pile caps supported by piles. The piles were drilled and cast. A hydrostatic slab is used to span between pile caps. The mat and the piles caps are fully waterproofed. A conventional perimeter reinforced concrete foundation wall is used around the perimeter of the site with a typically thickness of 600 mm. The outside of the foundation wall is waterproofed. The design water table is approximately 1.0 – 1.5 m below grade. Secant piles were used as the temporary retention system. Cross-lot bracing was used to laterally support the secant piles during construction. Dewatering of the site was required during construction.

2.5 *Construction*

Construction for the Tower uses repetitive forming techniques for the core and perimeter tubular frame. The overall shapes of the core and perimeter frame are square throughout the Tower with the core gradually punched more frequently throughout out the upper regions of the Tower. The building is designed to be stable under construction conditions without the installation of the steel bracing members with the structure essentially completed prior to the final connection of the steel bracing. This technique allows for a large portion of creep, shrinkage, and elastic shortening to occur within the reinforced concrete frame prior to final connection of the braces. All braces are designed for loads due to any additional building shortening after installation as well as exterior wall loads in addition to the lateral loads introduced from the base building. Large composite piers anchor the braces to the lower levels of the structure and the foundations.

3 SCREEN FRAMES THE GOLDFIELD INTERNATIONAL TOWERS BEIJING, THE PEOPLE'S REPUBLIC OF CHINA

3.1 *Introduction*

The Goldfield International Garden project consists of three separate concrete structures built over a common basement. Tower A is an office tower 36 stories and 150 m tall, above grade, with a gross framed area of 65,382 m^2 (including basements within tower footprint). Tower B is an office tower 28 stories and 97 m tall, above grade, with a gross framed area of 46,163 m^2 (including basements within tower footprint). Situated between the two towers is a retail podium 4 stories and 16m tall, above grade, with a gross framed area of 24,704 m^2 (including basements within superstructure footprint). In addition to areas described above, the majority of the site is excavated to 3 stories below grade. Basement areas include restaurants and truck loading areas at level B1 and parking at levels B2 & B3. This accounts for an additional 13,751 m2 of basement framed area not included in the numbers give above. The gross framed area for the project is 150,000 m^2 (including all basement areas).

The towers incorporate a unique structural engineering concept, the introduction of unsymmetrical lateral load resisting screen frames into a regular mega-frame system. These frames are used on two (2) faces while conventional frames are used on the opposite two (2) faces.

Figure 3a. Architectural rendering

3.2 *Structural system*

The superstructure consists of conventionally reinforced concrete slabs, beams, girders, columns, and shear walls. The superstructure is designed considering earthquake and wind load requirements for Beijing in addition to gravity loads. The structure is a Super High-Rise structure exceeding the typical height limitations for the lateral system type.

The reinforced concrete screen frames introduced into the structure were conceived from interpreting the architectural, developing multi-bar, multi-story mega-frames and infilling these frames with geometrically eccentric screen frame panels. The screen frames are located outside of the exterior wall system and are expressed. The screens also act as sun shading devices to control heat gain within the tower during the summer months. The screen frames were optimized to have similar stiffness to conventional frames located on opposite facades. The screen frames, as well as the conventional frames, incorporate the latest advances in ductile detailing to resist seismic loading. In addition, the stiffening panels are designed to essentially resist lateral loads only (with some building live load). Therefore, critical joints are left open during construction and then grouted before placement of the exterior wall system. This allows for most of the creep, shrinkage, and elastic shortening to occur in mega-frame elements only prior to engaging the stiffening frames.

3.3 *Superstructure*

3.3.1 *Tower A*

The lateral system for the building, 150 m tall from ground floor to roof level, will be a ‘frame core-wall’ dual system, consisting of reinforced concrete shear walls and a reinforced concrete moment frame. The shear walls will be located in the service area of the structure, around passenger and service elevators as well as stairways and mechanical rooms. The moment frame is located around the perimeter of the building and consists of square and rectangular columns and beams. The moment frame consists of two main beam and column configurations. The moment frames on the south and east sides of the building are conventional frames consisting of frame columns spaced at 6m on center and frame beams connecting the columns at every floor level. These conventional frames are located inside the glass enclosure of the building.

The moment frames on the north and west sides of the building form an exposed concrete ‘stiffened frame’ outside the architectural glass enclosure of the building. The ‘stiffened frame’ consists of primary frame columns spaced at 9m on center, connected together with primary frame beams connecting the columns together at every 3 stories. Each 9m wide x 12.3m tall bay is further stiffened with the addition of secondary frame columns and beams that infill the primary bay in the form of an architectural pattern. Delayed pour joints are provided where stiffening element connected to the primary frame to minimize gravity loads transmitted into the stiffening elements. Floor slabs are held back from the screen frames to create slots, with the only engagement of the floor structure and screen frames occurring at the connecting floor girders that frame into the columns. The “necks” of the girders at the slots are specially designed for seismic and gravity loads.

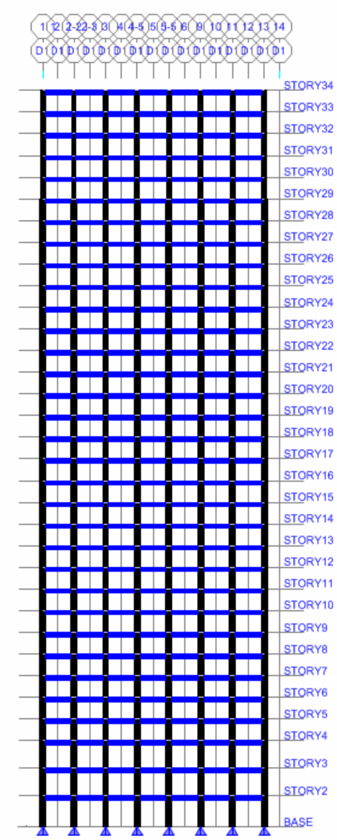


Figure 3b. Conventional moment frame (east façade)

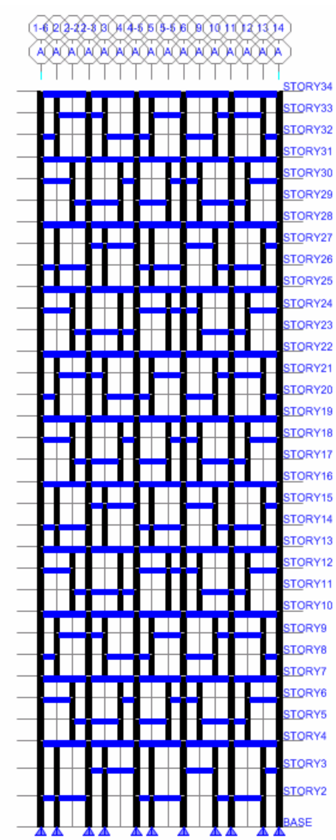


Figure 3c. Screen moment frame (west façade)

The gravity system for the tower will consist of conventional reinforced concrete slabs and beams. Shear walls and moment frame columns used in the lateral system will also be used to resist gravity loads.

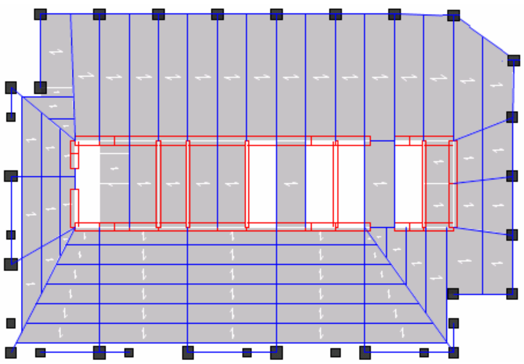


Figure 3d. Floor Framing Plan at Level 2

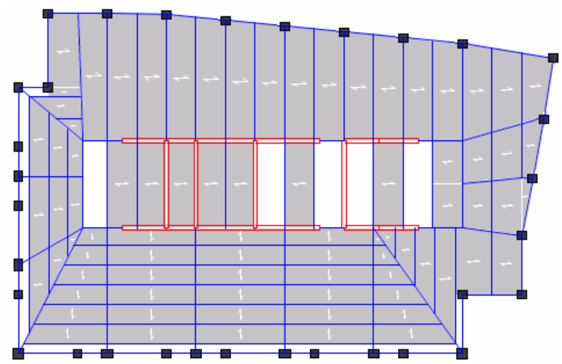


Figure 3e. Floor framing plan at Level 34

3.3.2 *Tower B*

The superstructure consists of conventionally reinforced concrete slabs, beams, girders, columns, and shear walls. The superstructure is designed considering earthquake and wind load requirements for Beijing in addition to gravity loads.

The lateral system for the building, 97 m tall from ground floor to roof level, will be a 'frame core-wall' dual system, consisting of reinforced concrete shear walls and a reinforced concrete moment frame. The shear walls will be located in the service area of the structure, around passenger and service elevators as well as stairways and mechanical rooms. The moment frame is located around the perimeter of the building and consists of square and rectangular columns and beams. The moment frame consists of two main beam and column configurations. The moment frames on the south side of the building are conventional frames consisting of frame columns spaced at 6m on center and frame beams connecting the columns at every floor level. These conventional frames are located inside the glass enclosure of the building. The moment frames on the north, east and west sides of the building form an exposed concrete 'stiffened frame' outside the architectural glass enclosure of the building. The 'stiffened frame' consists of primary frame columns spaced at 8.49 m on center, connected together with primary frame beams connecting the columns together at every 4 stories. Each 8.49m wide x 13.84m tall bay is further stiffened with the addition of secondary frame columns and beams that infill the primary bay in the form of an architectural pattern.

The gravity system for the tower will consist of conventional reinforced concrete slabs and beams. Shear walls and moment frame columns used in the lateral system will also be used to resist gravity loads.

3.4 *Foundations*

The foundation system consists of a conventionally reinforced concrete mat. CFG piles are installed under the mat foundation to limit differential settlement between the tower and the podium structure. Waterproofing will be required for the foundation wall system.

3.5 *Special analyses – moment frames comparison*

The “Screen” moment frame consists of moment frame columns spaced 9m on center and of moment frame beams located at specific levels (level 2, 3, 6, 7, 9, 12, 14, 18…). In addition to the primary elements, secondary beams and columns are located between the primary elements in a pattern to provide additional stiffness. The original moment frame on the North and West façade is to be replaced by the screen moment frame.

The East and South façade consist of moment frame columns spaced 6m on center and of moment frame beams at ever floor.

In order to insure that the behavior of the building is acceptable under seismic loading (translation in the weak direction in the first mode shape, translation in the strong direction in the second mode shape, torsion in the third mode shape…) the stiffness of opposite façades should be matched leading to similar displacement under the same loading.

Each façade have been analyzed separately, and subjected to the same static triangular loading. The difference in stiffness is proportional to the difference in displacement under the triangular loading.

Tuning the sizes of the members resulted in a perfect match of the stiffness of opposing frames as shown in figure 3i.

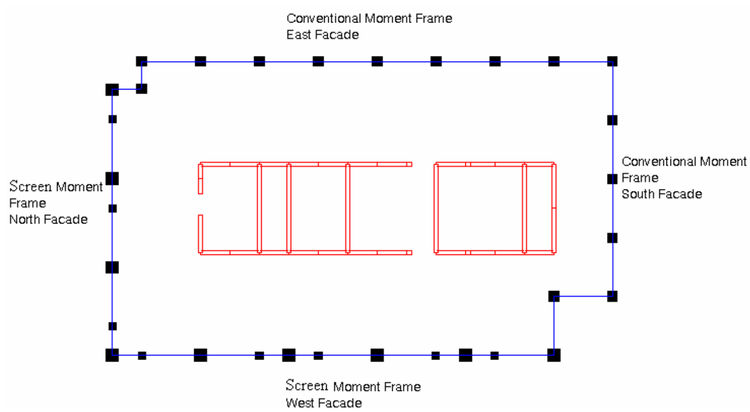


Fig. 3f: Plan View of the Building

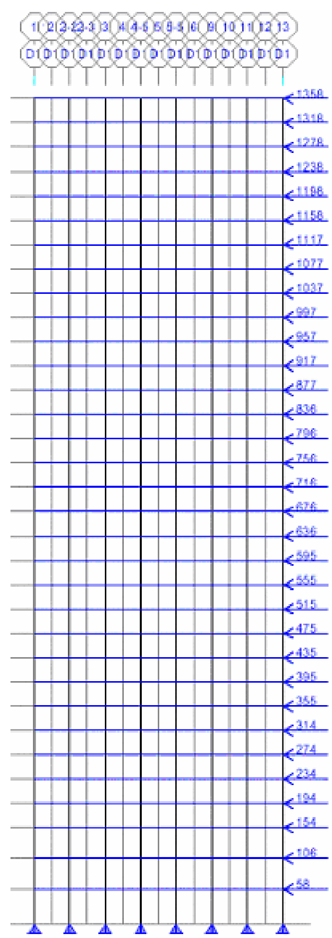


Fig. 3g: Triangular Loading on conventional moment frame

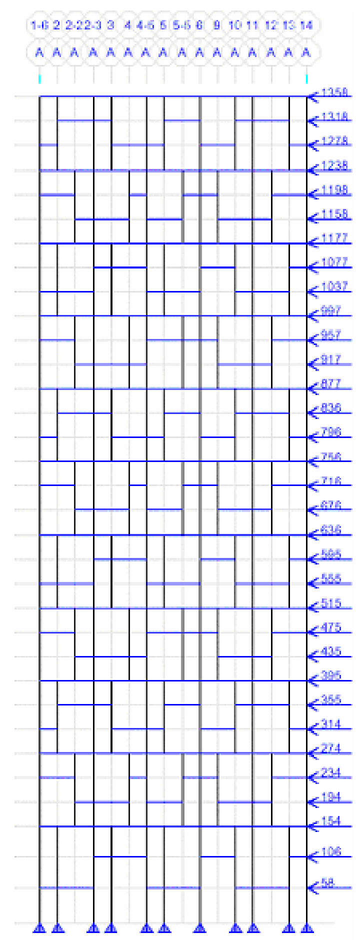


Fig. 3h: Triangular Loading on screen moment frame

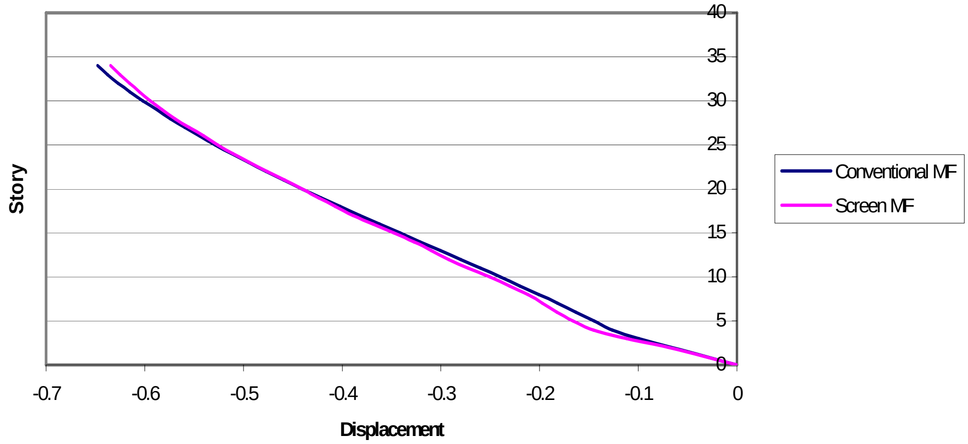


Fig. 3i. Story displacement comparison

3.6 *Non-linear push over analysis*

In order to reduce complexity and assure numerical stability, the structural behavior was studied using a two-dimensional equivalent stiffness model. This was accomplished by creating sets of two-dimensional frames made of line elements that matched the properties of the lateral resisting moment frames of the structural system. The core was modeled as a single line element with section properties matching the entire structural core on account of the relatively solid webs parallel to the critical direction. To account for the additional stiffness due to 3D effects in the 3D model, an additional moment frame was added on each side of the core. The modeled elements where then linked together at each floor level with rigid diaphragms.

The additional 3D effects stiffness is cause by the interior moment frames and the tubular action of the perimeter moment frame. The cross sections of the additional moment frame columns and beams were tuned to match the stiffness of the 3D and the 2D models in the elastic response range.

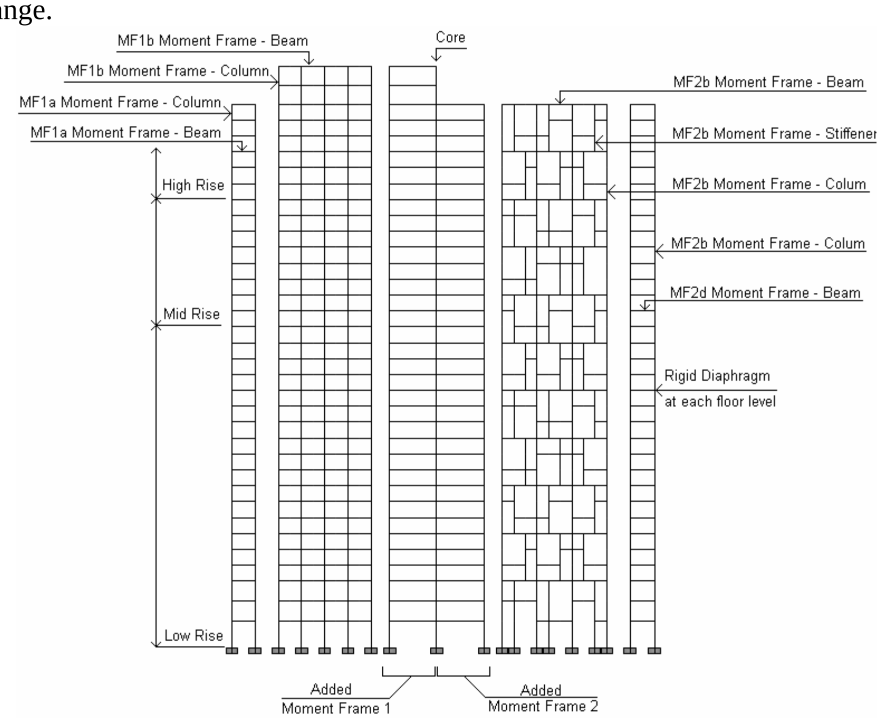


Fig. 3j. Non-Linear Pushover Two Dimensional Model (Critical Direction)

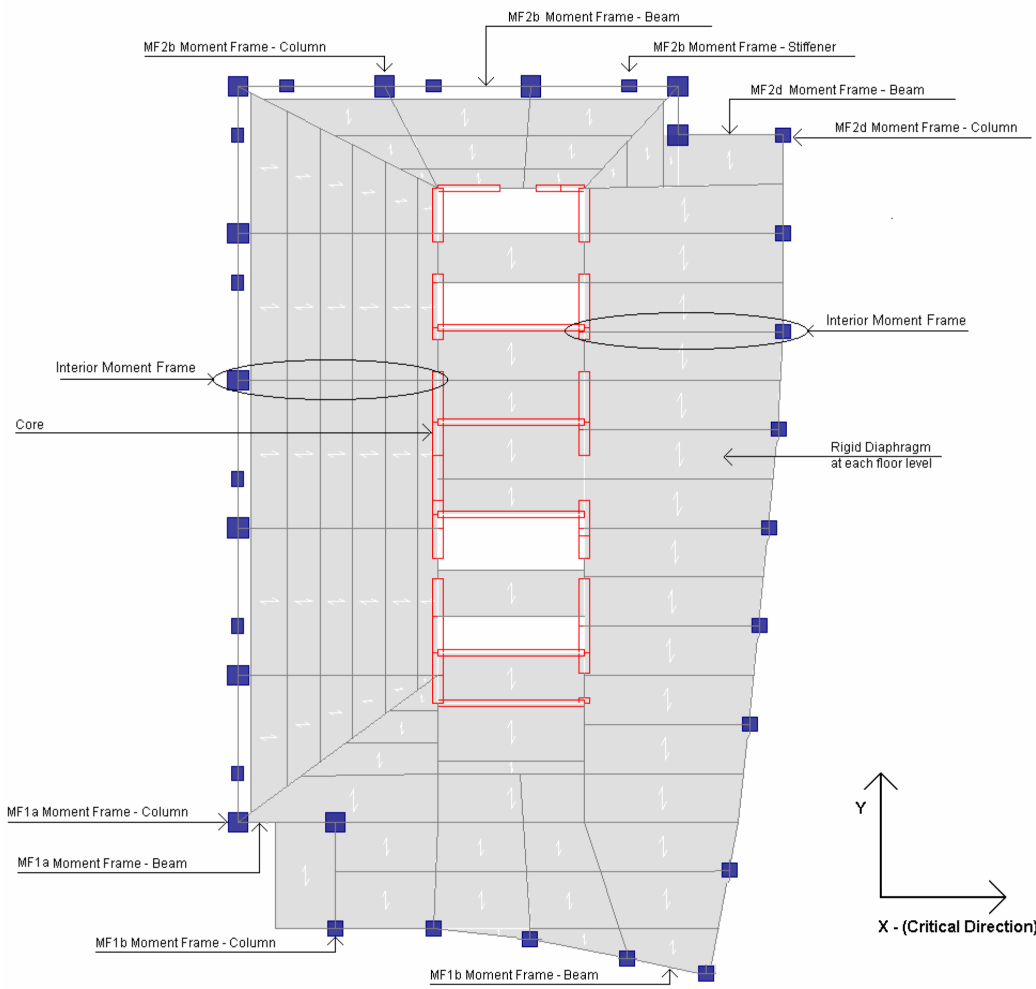


Fig. 3k. Typical Floor Framing Plan

Only the most critical "softest" direction was studied. The findings were then conservatively extrapolated to the stiffer, less critical, direction.

	X-Direction	Y-Direction
Max elastic Drift	0.001181	0.000096

Selection of Critical Pushover Loading Pattern

A unitized loading pattern based on the actual dynamic story force distribution was applied as point loads at each floor. The loading was incrementally increased and the roof displacement recorded till failure of the structure.

Generation of moment/rotation relationships

The computer program EXTRACT was used to determine the moment-curvature relationship for each different section type. This program requires the input of basic material behavior and section geometry. Extract generates a finite element model of the section and for a given axial load generates the moment-curvature relationships and given the hinge plastic hinge length (determined based on ACI 318-99) the moment-rotation relationship.

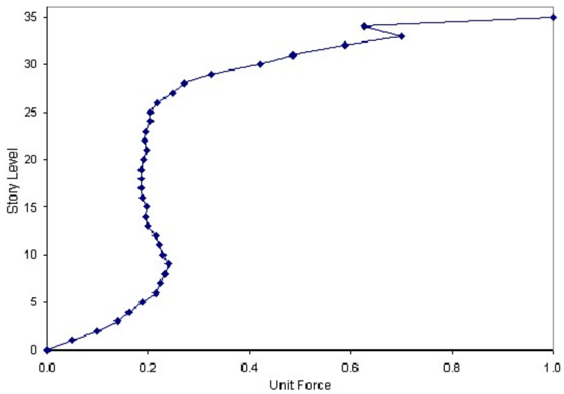


Fig. 3m. Pushover loading pattern

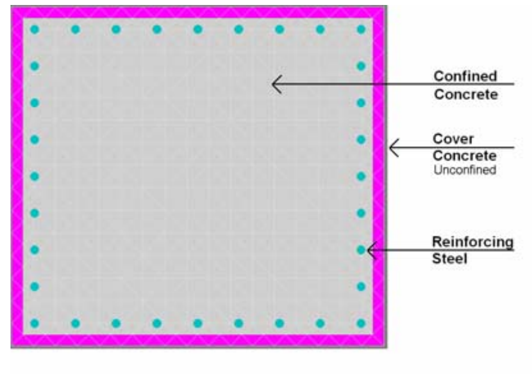


Fig. 3n. Element Cross Section in Extract

Material Modeling

To accurately model the non-linear behavior of plastic hinge formation in each of the plastic hinge zones the nonlinear properties of the materials were used. Concrete behavior was based on the Mander model of behavior with confining stresses computed from detailed properties. The steel material behavior was developed using a strain-hardening model, which assumes symmetric behavior for both compression and tension.

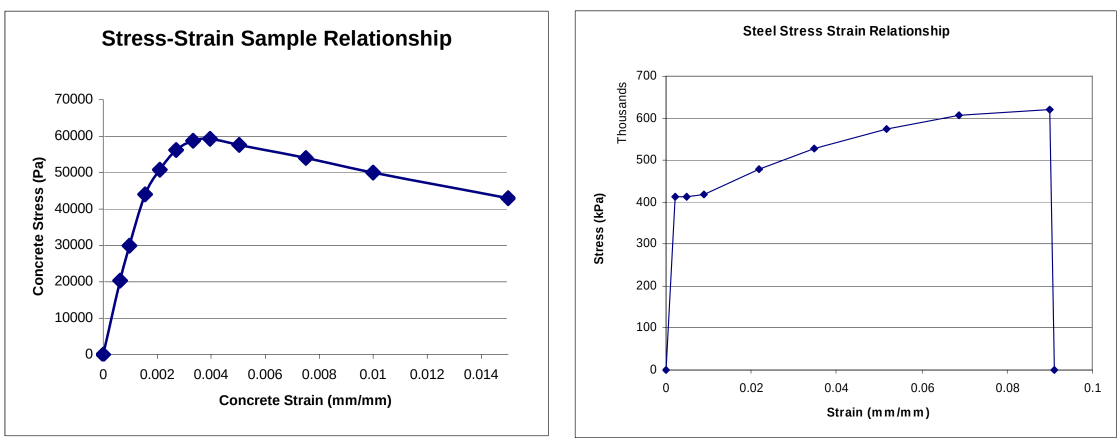


Fig. 3p. Concrete and Steel Nonlinear Stress-Strain Relationship

Displacement Demand Generation, ATC 40 – Nonlinear Static Procedure

In order to achieve an estimate of potential earthquake demand displacement ATC40 – Nonlinear Dynamic Procedure was utilized. This method consists in generating demand curves from ground motions and code design response acceleration spectra. The acceleration spectra are generated from the provided time history data, two recorded and one simulated rare earthquake ground motion history. The ground motion acceleration spectra and code response spectrum were generated and converted to acceleration vs. spectral displacement curves. Next, the acceleration spectra were scaled by the seismic mass of the building and plotted versus the spectral displacements to yield force displacement demand curves. The pushover curve data was then superimposed onto the demand curves. In the generation of demand curves 8% damping was utilized to account for additional energy dissipation due to inelastic structural response in the rare earthquake events.

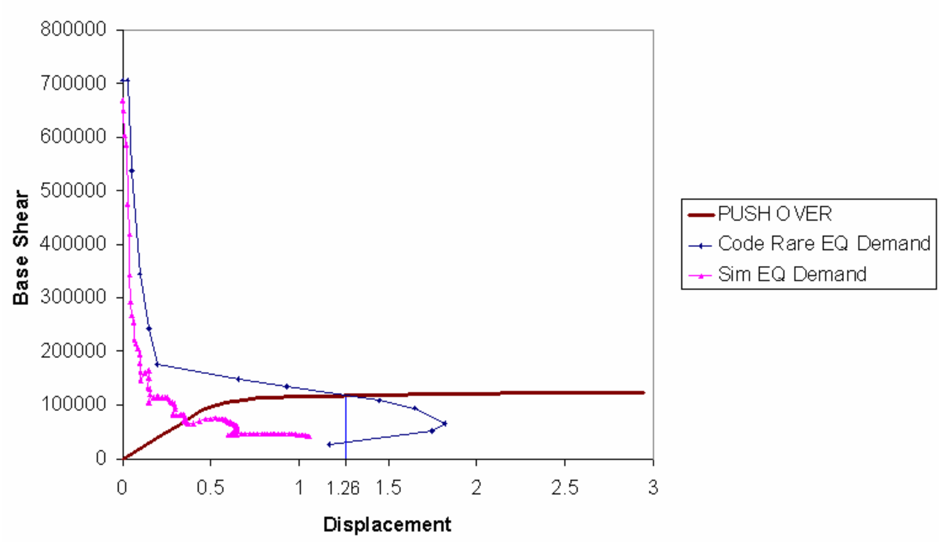


Fig. 3q. Pushover curve

The performance points (point of intersection of the pushover curve and the demand curves) satisfy the code maximum drift requirement of 1/100 per GB 50011-2001 section 5.5.5. The recorded earthquakes showed little energy for the period of the building and are omitted from the graph.

Structural Assessment

To verify that the building will not exhibit unacceptable torsional behavior as plastic hinges are formed in the pushover model, pins at member ends corresponding to plastic hinge location are added in the 3D ETABS model, and the code torsion check (JGJ3-2002 section 4.3.5) was performed. The torsion check was performed at four different stages of the plastic behavior:

1 - At the frequent earthquake base shear (step 2: the structure was still elastic)
2 - At the moderate earthquake base shear (step 4: the structure is slightly plastic)
3 - Halfway between the moderate earthquake and the code rare earthquake performance point demand (step 8)
4 - At the performance point of code rare earthquake demand (step 14)

The building did not exhibit any dramatic increase in torsional behavior due to the hinges formation.

3.6 *Construction*

The towers were designed to allow the central cores an all perimeter frames to be built at the same time. Screen frames were designed to incorporate pour joints that decoupled any gravity load transfers to lateral load resisting screens during construction. Instead of using the as-designed grout joints, the contractor choose to withhold pours on secondary screen frame columns and secondary bracing beams. All primary gravity loads in the screen frames are designed to be resisted by columns and beams located on a 9 meter wide by 3 story-tall (4 stories in Tower B) mega-frame.

4 THE ROCKER
THE NEW BEIJING POLY PLAZA, BEIJING, THE PEOPLE'S REPUBLIC OF CHINA

4.1 *Introduction*

The New Beijing Poly Plaza project is an unusual mixed-use development which includes 24 stories of office space, an eight story hanging museum 'lantern' structure and a 90 meter-tall atrium enclosed by what is expected to be the world's largest cable-net supported glass wall. The cable-net wall is 90 meters high by 60 meters wide - dimensions making a simple cable-net supported wall uneconomical. The design is achieved by folding the cable-net around diagonal V-shaped, parallel-strand bridge cables, subdividing the wall into three facets and reducing the effective cable spans. The parallel-strand cables also support the 'lantern' as it hangs in the atrium space without any columns extending to grade. Gravity loads from the 'lantern' are used to induce high levels of pre-tension in the parallel-strand cables. An innovative 'rocker mechanism' is used to isolate the cable hanger system from forces induced by lateral drift. The 'rocker mechanism' is architecturally 'celebrated' - an exposed articulated joint mechanism made of rigid pin-connected castings which perform as a pulley equivalent.

Figure 4a. Northeast Rendering

4.2 *Structural system*

The base building is a composite concrete and steel structure, roughly triangular in shape, and 24 stories tall above grade. The lateral system is a dual system consisting of reinforced concrete shear wall cores at the three corners of the building, (figure 4b), and steel moment resisting frames in the north-south and east-west wings.

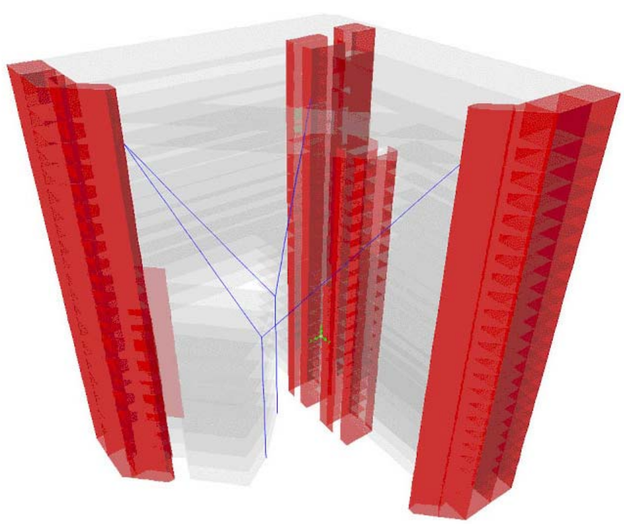


Figure 4b. Reinforced concrete cores

The floor framing system above grade consists of structural steel trusses acting compositely with metal deck slabs and lightweight concrete fill. The building also has a rectangular four-story basement, the lowest slab being located at approximately 20 meters below surrounding grade. Gravity framing in the basement consists of conventional concrete beam and slabs framing. The structure is underlain by a mat foundation anchored against hydrostatic uplift using tie-down anchors, where required.

Two areas of the base building structure required special treatment. The first area was the entire south wing of the building. To open the atrium up to direct sunlight from the south, steel columns in the southern 'wing' do not continue below the tenth floor creating a 'bridge' between the cores and columns at the east and west ends. The 'bridge' structure is supported by vierendeel trusses over its entire height from level 10 to level 24 (figure 4c). The bridge structure is considered part of the lateral system, acting, along with the columns at each end as a 'mega' frame. The 'bridge' and floor slab diaphragms tie the three cores of the structure together to form a monolithic structure, modeled and analyzed as such (figure 4d). Lightweight concrete fill was typically used on metal deck floor slabs, but critical connecting diaphragms used a thickened normal weight slab.

The second area requiring special engineering treatment was the museum occupancy termed the 'lantern', which protrudes from the southeast core towards the building atrium. The 'lantern' consists of an eight-story tall (starting at level 2) cross-braced steel frame that cantilevers 24m from the building core. There are no column elements underneath the lantern. The tip of the cantilevered frame is effectively propped by its connection to the primary diagonal cables which simultaneously stiffen the cable-net wall (figure 4e).

Figure 4c. Vierendeel bridge at south façade

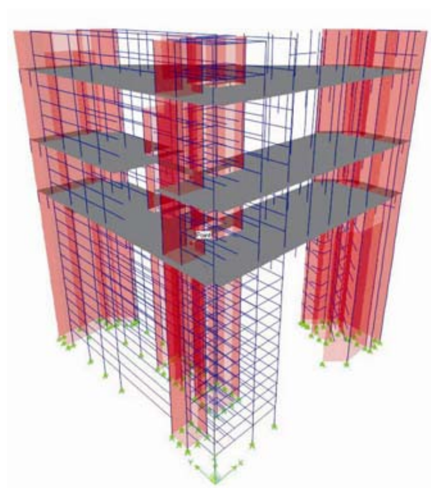


Figure 4d. Structural steel frames

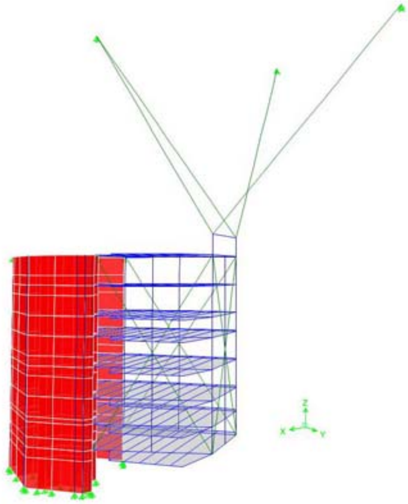


Figure 4e. Lantern structural system

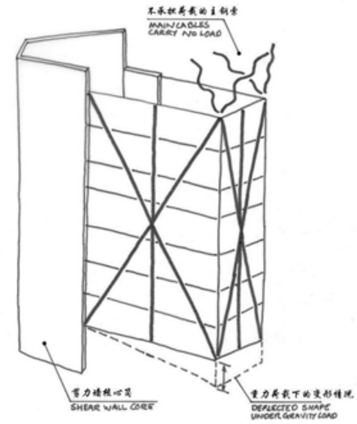


Figure 4f. Lantern redundancy concept

The gravity load bearing elements of the lantern are the southeast building core, and the primary diagonal cables which transfer gravity loads back to the cores at the top of the building. To provide a redundant gravity load path, the braced frame of the 'lantern' has been designed to achieve a life-safety performance level when cantilevered from the shear-wall core without the load supporting benefit of the primary diagonal cables (figure 4f).

Lateral forces in the 'lantern' are resisted by the shear wall core at the south-east side acting as a torsion box. The shear wall core is torsionally restrained by the ground floor slab at level 1 and by its connection to the main building through the level 12 and higher level diaphragms. The 'lantern' floor diaphragms transfer the lateral force to the core on a level by level basis.

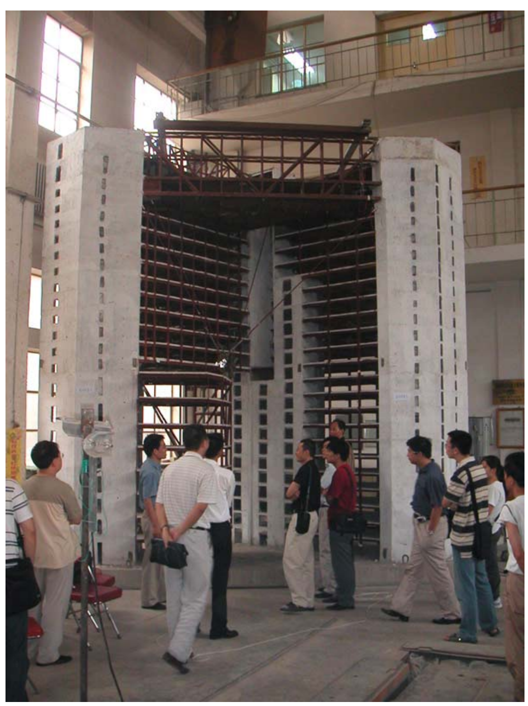


Figure 4g. Shake table test model

4.3 *Special structural systems – cable-net wall*

The New Beijing Poly Plaza project includes a 90 meter-tall atrium enclosed by a cable-net glass wall, 90 meters high by 60 meters wide. The scale of this wall greatly exceeds that which has been built before, introducing specific challenges that are not critical in smaller walls. SOM's preliminary analysis showed that the cable-net spans were too large to be economically achieved using a simple two-way cable-net design. SOM determined however that the cable-net could be achieved by subdividing the large cable-net area into three smaller zones by folding the cable-net into a faceted surface, and introducing a relatively stiff element along the fold lines. The faceted cable-net solution allows the individual sections of the cable-net to span to a virtual boundary condition at the fold line, effectively shortening the spans. Rather than introduce a major beam or truss element to stiffen the fold line, a large diameter cable under significant pre-tension is used. The cable-net wall system was designed to meet a span to deflection ratio limit of 45, when subjected to the service level wind load condition (50-year wind event). The cables were designed to meet the requirements of ASCE 19-96: Structural Applications of Steel Cables for Buildings. The design strength load factors of ASCE 19 were increased from 2.0 and 2.2 (depending on load condition) to 2.5 to meet additional requirements set by the committee of Chinese Structural Engineering Experts reviewing the design of the project. In addition to the application of increased load factors, the cable design forces were based on the internal forces resulting from a higher level wind condition (100-year wind event).

The 50-year and 100-year wind loading conditions were determined through careful wind engineering studies performed by Beijing University. The wind studies included a traditional rigid model of the building massing placed within a proximity model, and an aero-elastic wind tunnel study. The aero-elastic study was performed on a flexible model of the northeast cable-net wall, constructed using wires and a flexible membrane and tuned to simulate the anticipated dynamic response of the cable-net system. This study allowed the effect of feedback between the dynamic behavior of the cable-net and the wind forcing function to be considered. This additional study was used to verify and modify where appropriate the results of the rigid model study.

Analysis and testing shows that the New Beijing Poly cable-net wall behaves very much as conceived. The results from the static non-linear analysis (geometric non-linearity) clearly show that the strategy of subdividing the wall into facets with shorter individual spans was successful (figure 4h). This strategy allows the overall displacements to meet the L/45 deflection limit between hard boundary conditions while maintaining the economic viability of the project.

The final design solution was achieved with the largest of the four primary cables 275mm in diameter and consisting of a parallel strand bundle of 199 individual 15.2mm diameter 1x7 strands. The largest cable is pre-tensioned to 17,000kN, and experiences a maximum in service loading of 18,300kN during a 100 year wind event. Using the faceted design solution, the typical horizontal and vertical cables are limited in diameter to 34mm and 26mm, pre-tensioned to 210kN and 100kN respectively. Horizontal and vertical cables are spaced at 1333mm and 1375mm on center respectively.

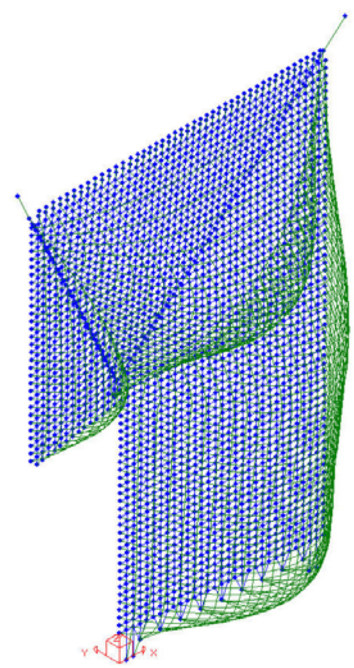


Figure 4h. Northeast cable-net deflection under static wind load condition

4.4 *The rocker mechanism*

The four primary diagonal cables which support the self-weight of the lantern connect diagonally from the roof of the ‘museum lantern’ at level 11, to the top of the atrium at level 23. As the base building structure will drift under anticipated seismic loads, the cables will act as braces and attempt to resist the base building drift unless the force levels in the cables are limited in some manner. Designing the primary diagonal cables to resist these brace forces while maintaining an appropriate factor of safety would have significantly increased the primary diagonal cable sizes that as employed in the final design solution. This would also have resulted in the initial level of pre-tension in the primary diagonal cables being a lower portion of the cable breaking strength, to accommodate the additional brace demands. Pre-tensioned cable systems typically rely on a high initial level of pre-tension to maintain the desired architectural form in the permanent load condition. When cable systems are installed with only a nominal level of initial pre-tension, the tendency of that system to exhibit significant deflections due to the self-weight of the cables is greatly increased. Therefore, it was determined that the design solution required that the primary diagonal cables (the only cables that may act as braces) be decoupled from the lateral system of the base building structure.

The connection between the primary diagonal cables and the roof of the lantern is complicated by the need to decouple the primary cables from the lateral system of the base building structure, and to simultaneously provide a flexible wall system which allows the relative lateral movements between the roof of the lantern and the roof of the building to be incrementally accommodated over the height of the cable-net. Several connection concepts were evaluated before the final design solution was determined. One option connected the main cables to the lantern roof through a sliding connection (figure 4i). This solution was difficult to achieve due to the resulting eccentric load path of the very large primary cable forces through the eccentric connection when the connection was displaced. It also resulted in the upper half of the cable-net moving with the roof of the building, and one course of glass at the roof of the ‘lantern’ being required to accommodate the full drift between the roof of the building and the roof of the ‘lantern’. This resulted in this course of glass likely to fail given any significant lateral displacement of the building, causing a safety hazard in the atrium and street below. A second concept connected the bottom of the ‘V’ cables to the top of the lantern through a 4m tall, pin-ended’ link element (figure 4j). This solved the load eccentricity issue, but still resulted in the relative lateral drift of the upper half of the cable-net being concentrated in a small portion of the wall. This solution also induced tension in the main cables as the building drifts due to the downward movement of the lowest point of the cables caused by the rotation of the link around its base. The concentration of a significant portion of the lateral drift of the building in a 4m high zone still resulted in the high likelihood that glass panels would be lost during the design level lateral drift event, representing an unacceptable risk to the occupants of the building and adjacent outdoor spaces.

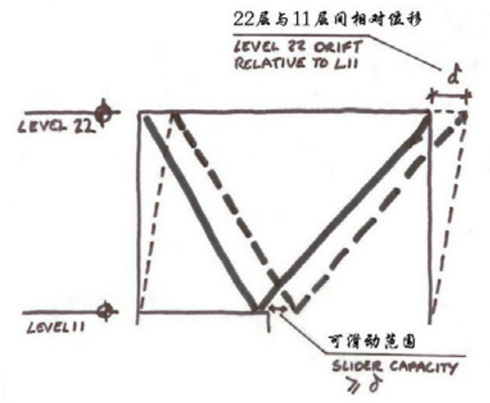


Figure 4i. Slider connection concept

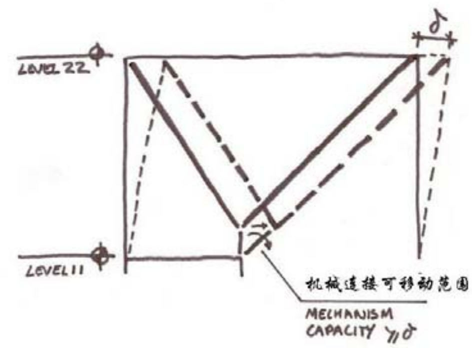


Figure 4j. Link connection concept

The final solution is shown diagrammatically in figure 4k. The decoupling mechanism consists of the equivalent of a pulley at the lower point of the 'V' cables. As the overall building drifts, one half of the 'V' tries to lengthen and the other half tries to shorten. By connecting them together using a pulley or equivalent mechanism, the strains are able to offset each other, without inducing additional load in the cables. A cast steel 'rocker mechanism' was designed to perform the equivalent function of the pulley. By crossing the cables and connecting to the rocker casting arms, the need to provide curved pulley surfaces and curved sections of the main cable were eliminated (figure 4m). The 'rocker mechanism' solution allows the load path at the connection to be concentric, and also allows the relative lateral drift of the upper half of the building to be distributed through the upper portion of the cable-net wall. Small relative movements between adjacent nodes on the main diagonal cables and the cable-net cables are accommodated using pin-ended tie-rod connections.

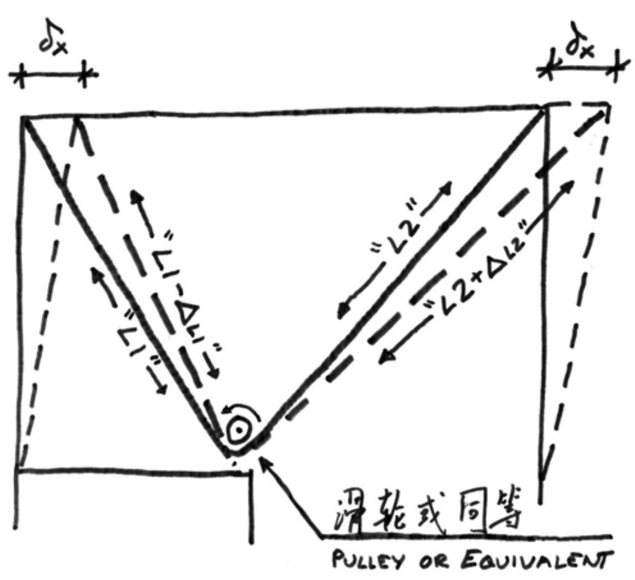


Figure 4k. Pulley equivalent concept

To evaluate the effectiveness of the design solution prior to completing in-depth analysis of the system, a physical model of the 'rocker mechanism' was built along with a model of the 'link' concept for reference comparison. The models were installed in a pin-connected frame, with soft springs installed in series with the diagonal cables. By racking the frame backwards and forwards, the relative effectiveness of the two concepts could be visually evaluated. The physical model test demonstrated significant extension in the springs using the 'link' model and negligible extension in the springs using the 'rocker mechanism' model, highlighting the ability of this connection to decouple the main cables from the base building lateral system.

The final design of the 'rocker mechanism' included five large castings per connection. The main cable clevis castings are approximately 4m in length. The clevis castings are designed to pass through each other to maintain concentric load paths through the connection (figure 4n).

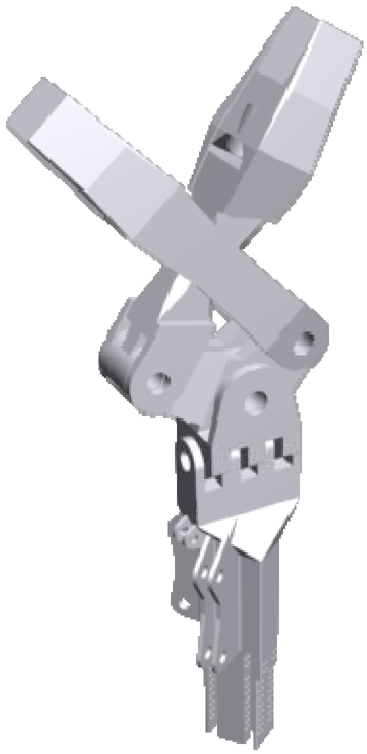


Figure 4m. The 'rocker mechanism'

Figure 4n. Rocker clevis castings on site

5 SUN SHADES
POLY INTERNATIONAL PLAZA
GUANGZHOU, THE PEOPLE'S REPUBLIC OF CHINA

5.1 *Introduction*

The Poly International Plaza complex occupies a 57,565m^2 site in the Pazhou district of Guangzhou. The site is located immediately to the south of the Pearl River, with the river front drive of Bin Jiang Road forming the northern boundary. Ke Yun Road bounds the site to the east, and is carried north by bridge over the Pearl River. Directly to the west is the site of a proposed five star hotel, beyond which is the Guangzhou International Convention and Exhibition Center. An access road forms the southern boundary of the site.

Figure 5a. Plaza complex (above)
Figure 5b. Complex in plan (below)

The Poly International Plaza complex comprises two similar 150m high, 35-story office towers, and two 3-story podium structures. The buildings are arranged around a large rectangular landscaped court, with the office towers occupying the north-east and south-west corners, and the podium structures occupying the east and west perimeters. The towers contain approximately 108,920m^2 of office space, and the podium structures approximately 18,586m^2 of additional program, to include a health club, business center, retail and restaurants. There are two basement levels that extend over the footprint of the buildings and the landscaped court. The basement levels contain mostly car parking and mechanical space. The total basement area is approximately 49,741m^2.

The two office towers are exceptionally slender, each having an aspect ratio of 8 to 1. The gravity and lateral support for the towers utilizes conventional structural systems that are creatively integrated to define a distinctive architectural form that not only addresses the issues related to the slender proportions of the buildings, but also helps to reduce the energy demand on the buildings' heating and ventilation systems.

5.2 *Structural System*

The architectural design concept for the towers was to provide a long, rectangular, floor plate with no internal columns. The rectangular shape was to align in an east-west direction, with the broad, north face of each tower offering views directly over the Pearl River. As a consequence of the tower alignment, it was recognized that the south face would be vulnerable to considerable heat gain from the sun, and there was a desire to provide some form of shading to combat this.

The structural system for the towers comprises braced frames on the east and west ends to provide lateral stability in the slender direction. These frames are supplemented by outrigger trusses at mid-height and the top of the tower. A conventional moment frame on the north face, with a double line of cross-braced frames and moment frames connecting piers on the south face provides lateral stability in the long direction. The double line of cross-braced frames also provides out-of-plane stiffness to supplement the lateral support in the slender direction. In combination with the piers, the cross-bracing has the additional advantage of providing essential sun-shading on the south face of each tower. The combination of these lateral systems provides the required stiffness to resist the typhoon wind loads and moderate seismicity of the region, achieve the strict interstory drift ratio demanded by the Chinese building codes, all whilst maintaining a slender building profile.

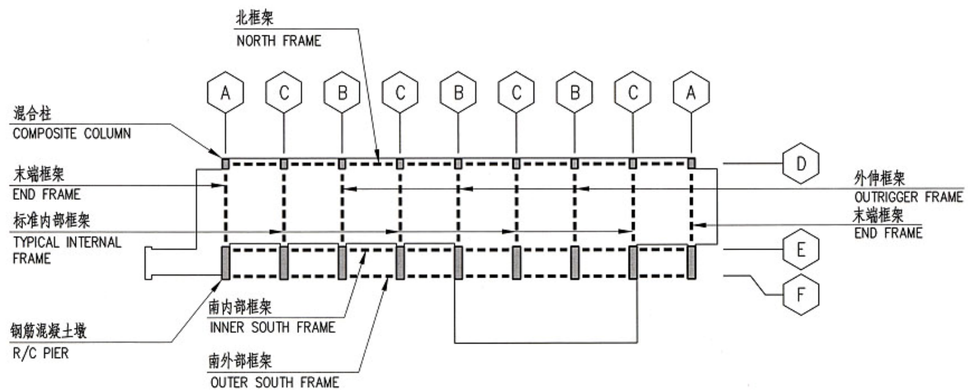
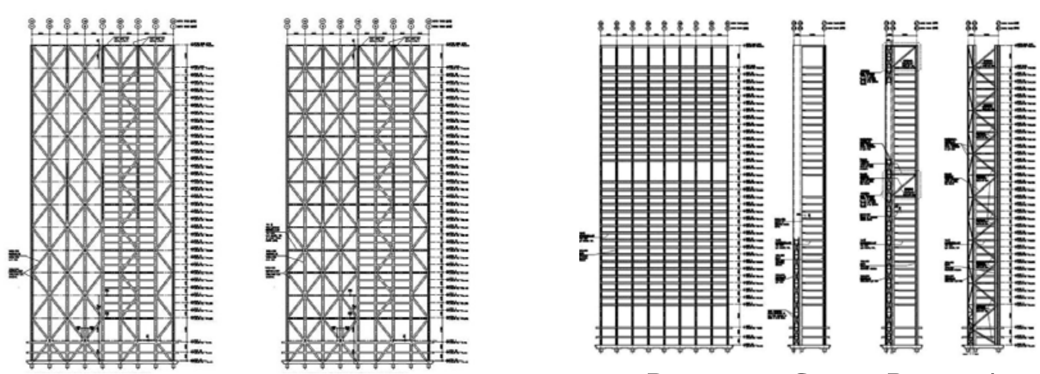


Figure 5c. Structural system plan

5.3 *Superstructure*

Both towers consist of a combination of reinforced concrete and structural steel. Primarily, reinforced concrete is used to construct the slabs, beams, columns and piers. For each tower, nine composite columns, incorporating structural steel shapes, are spaced at 9m on center along the north face. The columns are typically 1000mm wide in the east-west direction by 1500mm deep, although at either end the columns are 3000mm deep. Nine reinforced concrete piers, each 1100mm wide by 4950mm deep align with the composite columns on the north to form the south face. The clear span between the composite columns and piers is typically 12.0m. Braced frames comprising three-story high built-up steel box shapes link the composite columns to the piers at the east and west ends. Internal outrigger trusses, also comprising three-story high built-up steel box shapes link the composite columns to the piers between Levels 18 and 21 (at building mid-height) and between Roof Level and the Upper Roof Level. Three outrigger trusses are provided at each, located at 18m on center to link every second composite column and pier along the length of the building. The alignment of each outrigger truss diagonal matches those on the end braced frames.



D C B A

F E

Figure 5d. Building Frame Elevations (Refer to figure 5c.)

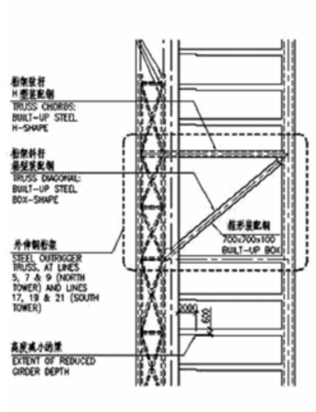


Figure 5e. Outrigger truss detail

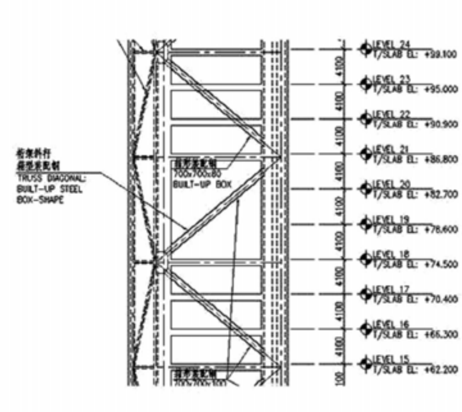
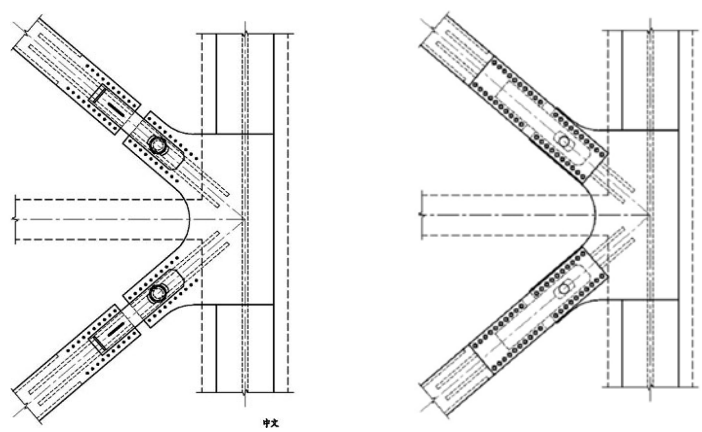


Figure 5f. End truss detail

Figure 5g. North tower



Initial stage

Final stage

Figure 5h. Typical frame joint details

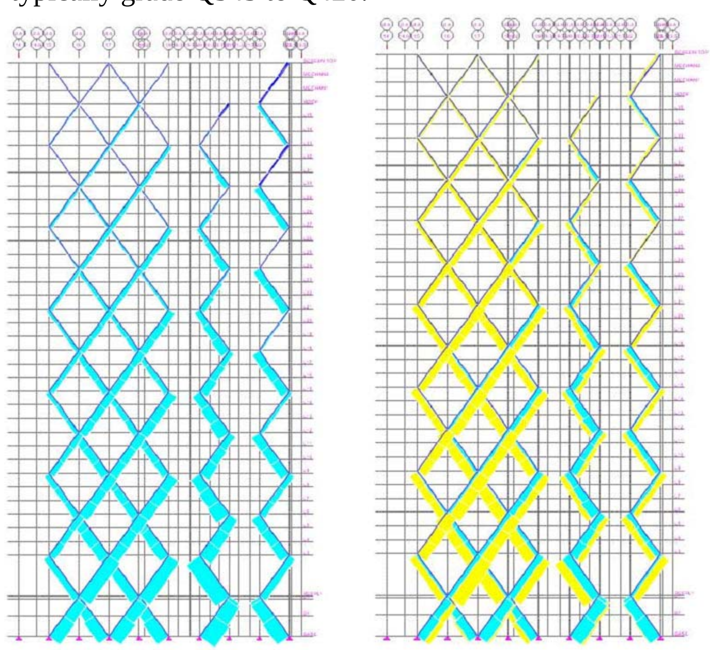
Along the north face of each tower, 600mm wide by 1000mm or 1150mm deep reinforced concrete beams span between the composite columns to provide a moment frame. On the south face, two lines of braced frames link the piers to provide a very stiff lateral support system for east-west stability. These braced frames comprise three-story high built-up steel box shapes that alternately zig-zag within each 9m bay, to give an overall cross-braced façade. On the north tower, the four westernmost bays plus the single easternmost bay are braced in this way. The remaining three bays are not braced to provide access between the main floor area and the building core. The configuration on the south tower is similar, but with the location of the core flipped such that the four easternmost bays and the single westernmost bay are braced. The outer line of braced frames is centered 450mm back from the south face of the piers. The inner line is centered 3600mm behind the outer line. At every third floor level, where the inner line of braced frame diagonals connect to the piers, 1000mm wide by 750mm deep reinforced concrete moment frame beams also connect the piers. These moment frame beams repeat at every floor level within the three bays that interface with the building core. Within the five braced bays, the moment frame beam also defines the southernmost slab edge of the main floor area. At the intermediate floor levels where moment frame beams do not occur, the slab edge is set back 1200mm behind the center-line of the inner braced frame. This allows for the metal and glass curtain wall to be located entirely behind the braced frames over a continuous three-story height, with the provision for sufficient space for window washing equipment.

The gravity system for the towers is comprised of one-way spanning reinforced concrete floor slabs, supported by north-south spanning reinforced concrete beams and girders at 3.0m on center. These long-span members are constructed with an upward camber to minimize the effects of dead load deflection. The composite columns on the north face and piers on the south face of each building act as the primary vertical load bearing elements.

The building core for each tower is 27m (three bays east-west) wide by 12m deep. The core area, containing elevators, a stair and other building service facilities, is positioned south of the main floor area. The core is structured using reinforced concrete columns, beams and slabs and has nominal lateral stability as provided through moment frame action. Lateral support is otherwise achieved through the connection of the core to the main building. In addition to the stair in the core, access stairs constructed from reinforced concrete are also located at the east and west ends of the main building. At the east end, the stair and vestibule cantilevers from the end bay, whilst at the west end an additional 9m bay with pier is provided within which the stair is located. Metal and glass curtain wall encloses both these stairs.

Between Levels 18 and 21, where the mid-height outrigger trusses occur, the typical building footprint is interrupted. At Level 19, a four-bay long floor plate, set-back from the north edge of the building is provided adjacent to the building core. The roof of this area is at Level 20. Otherwise, three-story high openings penetrate the exterior wall of the towers, providing an outdoor refuge assembly area at Level 18. Provision of the three-story high exterior wall opening also assists in reducing the wind loads at this level.

Concrete for the superstructure is typically grade C40 throughout, with grade C45, C50 and C60 being utilized for the composite columns and piers. Structural steel is typically grade Q345 to Q420.



Max. compression = 11,470 kN

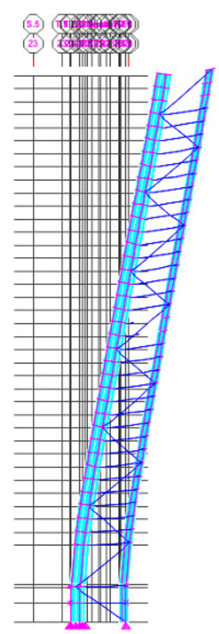
Gravity load combination

Max. compression = 12,051 kN

Max. tension = 3,792 kN

Lateral load combination

Figure 5i. Force levels in outer south frame



Building drift due to wind

= 176 mm

= building height / 855

Figure 5j. Deflected shape

5.4 *Substructure*

Two basement levels are located beneath the ground level of each tower. The suspended basement and ground floor levels are comprised of two-way spanning reinforced concrete floor slabs, supported by two-way spanning reinforced concrete beams at 3m on center, and reinforced concrete girders located on the building column and wall lines. The columns and walls for the building structures above are supplemented where necessary by additional basement columns and walls. The perimeter of the basement excavation is retained by a reinforced concrete foundation wall. Concrete for the substructure is typically grade C40 throughout, although grade C60 is used for certain vertical load bearing elements supporting the towers.

5.5 *Foundations*

The foundation system typically comprises individual hand-excavated caissons that support each column. Groups of caissons are provided to support the piers of the office towers and the shear walls of the podium buildings. Grade beams interconnect the caisson caps of the office towers. The basement wall is supported by a continuous grade beam that in turn is supported by caissons spaced at regular intervals. A hydrostatic slab is provided over the entire basement to resist the hydrostatic uplift forces. This slab is a two-way spanning system, supported by the grade beams (where these exist) and the caisson caps. The foundations are constructed using grade C40 concrete.

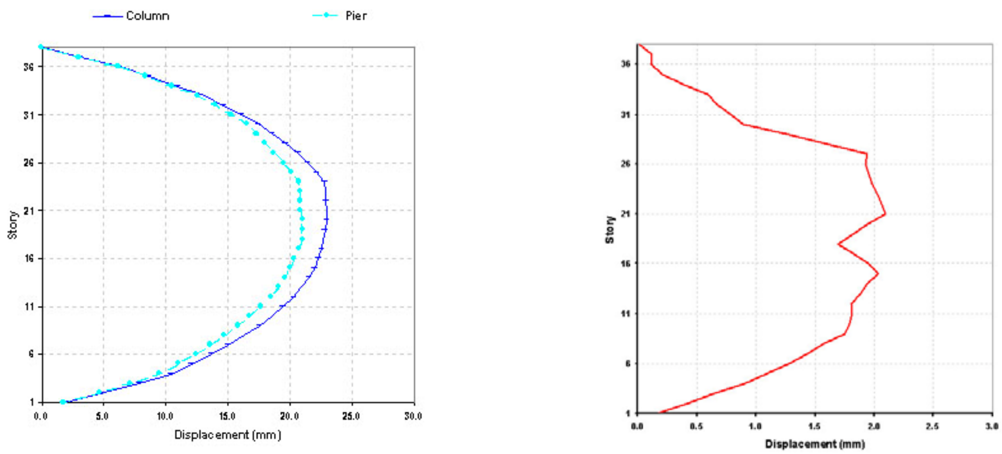
Architectural concept

Under construction

Figure 5k. View of the towers looking north

5.6 *Construction Issues*

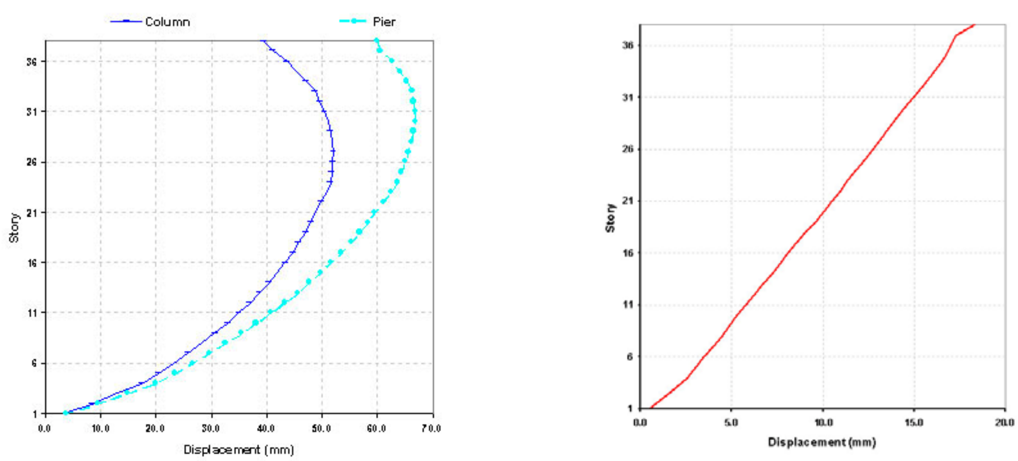
Concrete creep and shrinkage analyses were performed to establish the difference in elevation that may be expected between the composite columns on the north face of the tower and the piers on the south face. This was necessary to assess likely construction scenarios related to the installation of the end braced frame and outrigger truss elements. A possible construction schedule was established and analyses performed for periods of 260 days and 10,000 days after the start of construction. The differences in elevation between the north and south sides of the tower due to the effects of creep and shrinkage were calculated to be approx. 2mm at Day 260 (assumed to be the end of building construction) and approx. 18mm at Day 10,000. Details were developed for the end connections of the steel frame elements that allowed for a limited amount of movement between the north and south sides of the tower during construction, with the final torqued bolted connections being completed near the end of building construction.



Relative vertical movement between start of construction and Day 260

Difference between pier and composite column

Figure 5l. Creep and shrinkage effects at Day 260 (end of construction)



Relative vertical movement between start of construction and Day 10,000

Difference between pier and composite column

Figure 5m. Creep and shrinkage effects at Day 10,000

6 THE PERFECT TUBE
JINLING HOTEL TOWER, NANJING, THE PEOPLE'S REPUBLIC OF CHINA

6.1 *Introduction*

The Jinling Hotel Tower is a twisting, sculptural form that expresses the building's program and its structural system to create an iconic form on the Nanjing skyline. The 320-meter tall, 80-story building consists for four vertical quadrants, each floor gradually rotating relative to the floor below for a total of 90 degrees over the building height. The bottom floor plates are square to provide the efficient plans for Class A office space. Cruciform-shaped floor plans are used in the middle two quadrants housing luxury apartments. Since the apartments required more exposure to natural light, air and views these floor plates maximum surface area for each unit. At the top of the building, the floor plates become square once again to house a six-star hotel. In addition to changing shape in plan, the tower tapers allowing for ideal room sizes off of the core area. A 28-story atrium brings light into the hotel interior through a glazed roof and exterior creases in the torqued façade. The core area below this atrium is centrally located and typically houses all services including elevators, mechanical rooms, washrooms, and storage rooms. All spaces outside of the core are column-free allowing column free floor plans whether used for office or residential spaces.

The Tower is enclosed in a diagonal-mesh tubular frame and is combined with the central core. The system provides efficient resistance to seismic and wind loads. Tubular frames are one of the most efficient structural systems for tall buildings by resisting lateral loads and minimizing bending of individual primary frame members. The Jinling Tower relies on a "mesh-tube" system using a fine grid of diagonal structural members. When subjected to lateral loads, the individual members bend minimally, resulting in primarily axial deformation and leading to what is virtually 100 percent structural efficiency. The structural grid allows for highly efficient construction since entire story segments can be pre-fabricated, ship to the site, and quickly erected.

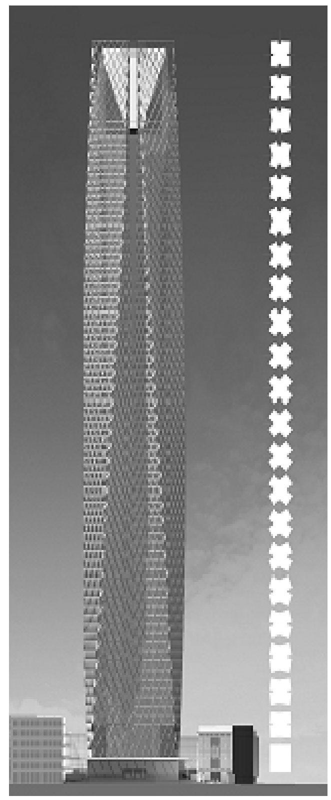


Figure 6a. The perfect tube

6.2 *Structural system*

The structural system conceived for the ultra-tall Jinling Hotel Tower was developed to maximize structural efficiency and minimize material quantities while integrating directly with the architecture and maximizing column-free lease spans.

The primary structural system used to resist lateral wind and earthquake loads is located in the central core area of the building where elevators and back-of-house functions exist and at the perimeter of the building where maximum structural resistance to overturning can be achieved. The basis for the structural system is a tube-in-tube concept combining a central reinforced concrete core and perimeter moment resisting tubular frame.

While developing the designs of two of the most efficient structural systems in the world for tall buildings, Dr. Fazlur Khan, Partner and Chief Structural Engineer of Skidmore, Owings & Merrill discovered that tubular structural systems located at the perimeter of the building would be ideal in resisting lateral loads. He concluded that the ideal cantilever tube would be one that consisted of a continuous solid perimeter wall. Recognizing that this was not practical in building design practice, he developed the tubular system that consisted of closely spaced columns and large spandrel beams. For the John Hancock Center (completed in 1970) in Chicago he conceived of a mega-diagonally braced tube and for Sears Tower (completed in 1974) in Chicago he bundled tubes to increase structural efficiency. Although he came close, he was never able to achieve pure cantilever behavior in the tubes.

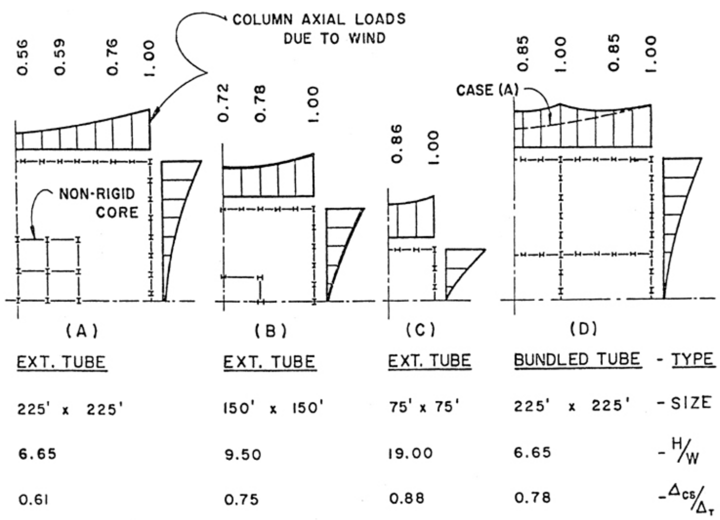


Figure 6b. Dr. Fazlur Khan – tubular frame efficiency

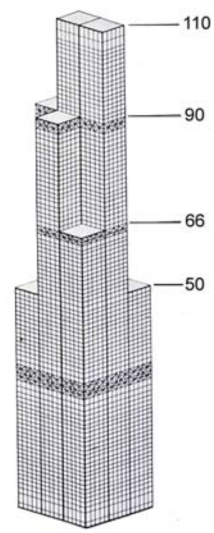


Figure 6c. Sears Tower bundled tube

Khan considered optimal structural efficiency in a tall tubular building as the lateral displacement of the building due to axial column shortening only. Bending and shear deformation of frame elements only reduces the structural efficiency leading to higher material quantities and therefore costs. Khan defined structural efficiency as the lateral displacement of the frame due to axial column shortening only divided by the total lateral displacement when considering all shear, bending, and axial shortening. He established the a threshold of 0.8 or 80% efficiency was required to achieve an efficient structural system for a tall building without adding significant structural materials to the structure to resist lateral forces. Sears Tower's tube was initially 61% efficient considering the base plan width, the column spacing, the floor-to-floor height, and the building height. By introducing interior frames and creating a bundled tube, he was able to achieve 78% efficiency, approximately equal to the target efficiency of 80%.

The perimeter tubular system for the Jinling Hotel Tower was initially conceived as a conventional tubular frame. Columns were spaced at 4 meters on-center and floor-to-floor heights were spaced at 4 meters and 3.2 meters for the lower and upper areas of the building respectively. The plan dimension at the base was 48 meters and the height 320 meters with an aspect ratio (height / width) of 6.7. The aspect ratio is the same as that for Sears Tower. Through hand calculations and a finite element computer analysis, it was determined that the efficiency of the Tower was 62%. The results were slightly better than Sears but far short of Khan's established criteria of 80%.

A diagonal mesh was then considered at the building perimeter. The diagonal spacing was the same as the columns on the conventional tube (4 meters on-center). Floor-to-floor heights, plan size, and building height were all considered the same as the conventional tube. The analysis results yielded a tubular system that was essentially 100% efficient. Fundamentally, no bending occurred anywhere in the frame. This behavior confirms the highest lateral efficiency with minimal structural material. Khan's goal of achieving 100% efficiency without using a continuous perimeter wall has been achieved with the "mesh-tube" frame concept.

Shear lag, the distribution of axial load along the leeward face of the tower due to tubular behavior, is significantly reduced with the mesh-tube concept. Columns within the center region of the leeward in a conventional tube experience a normalized axial load of 0.56 when considering unity at the corner while the same columns in a mesh tube experience a normalized load of 0.74. This represents an approximate 50% reduction in shear lag.

6.3 *Superstructure*

The superstructure typically consists of a combination of conventional reinforced concrete for the central shear wall core and structural steel for the floor framing and the exterior diagonal mesh.

The lateral system for the Tower consists of a tube-in-tube structural system combining a ductile reinforced concrete central core wall and a perimeter structural steel diagonal “mesh tube.” Diagonal bracing or mesh members at the perimeter form a highly efficient braced tube. The mesh tube is interconnected on typical floors with structural steel floor framing members and composite metal deck slab diaphragms and on select floors (primarily mechanical spaces) with outrigger / belt trusses. The most advanced reinforced concrete detailing is introduced into core wall link beams and walls with ductile structural steel detailing introduced into the perimeter mesh tube. This detailing ensures the strength and ductility of the Tower.

Long-span composite structural steel framing clear spans between the central core wall and perimeter mesh tube. The clear span will allow for column-free interior spans while placing all gravity loads on lateral load resisting elements. This gravity load reduces any tensile loads in diagonal mesh members caused by lateral loads.

The reinforced concrete core wall includes interior web walls at the lower portion of the Tower with those walls eliminated in upper portions of the Tower as structural demand decreases. Open core wall areas are used for atria. The reinforced concrete core wall thicknesses for perimeter flange components vary from 850 mm at the base of the building to 450 mm at the top. Web walls are typically be 450 mm thick. Concrete strength varies from C60 at the base to C40 at the top.

The structural steel members within the perimeter tubular mesh frame are typically spaced at 4 meters on-center at the base of the building varying to 3.33 meters on-center at the top of the building. The member sizes are typically W12 or W14 (300 mm to 350 mm) with built-up sections of similar outside dimensions at the base of the building. In the event of an earthquake, ductility is ensured by interconnecting the diagonal members with horizontal structural steel beams. These beams also provide support for exterior wall and floor framing elements. The members are typically 600 mm deep.

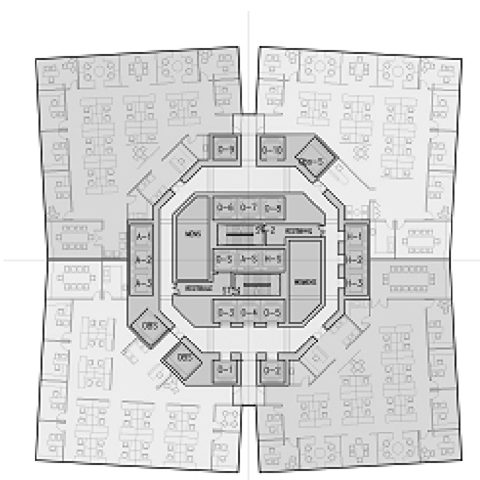


Figure 6d. Typical low-rise office floor plan

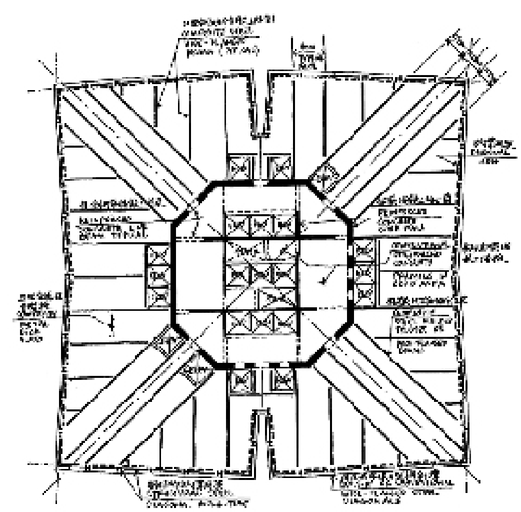


Figure 6e. Typical low-rise office framing plan

The gravity system for the Tower typically consists of composite structural steel floor framing elements and composite metal deck slabs. The floor framing elements consists of built-up steel trusses or wide-flanged steel beams. Steel framing members are typically spaced at 4.0 meters on-center with a composite metal deck slab with a total thickness of 135 (50 mm deep metal deck topped with 85 mm of light-weight concrete). As an alternate, normal weight concrete can be used with a total slab thickness will be 185 mm (75 mm deep metal deck topped with 110 mm of normal-weight concrete). C30 concrete is used in the metal deck slab system to top the composite metal deck. Shear studs are used to achieve composite action between the structural steel floor framing and the composite metal deck.

The floor framing depth of built-up steel trusses is 750 mm – 900 mm with wide-flanged beam framing typically 450 mm deep. Conventional reinforced concrete framing is used in the core wall areas. The depth of the framing in the core area is typically 600 mm.
In addition to resisting lateral loads, the reinforced concrete core wall and perimeter mesh tube act as primary gravity load resisting elements.

Conventional sprayed-on cementitious fireproofing is required for structural steel framing members and perimeter mesh members. However, sprayed-on fireproofing is not required for metal deck slabs since the thickness is engineered to achieve the required code-defined fire rating.

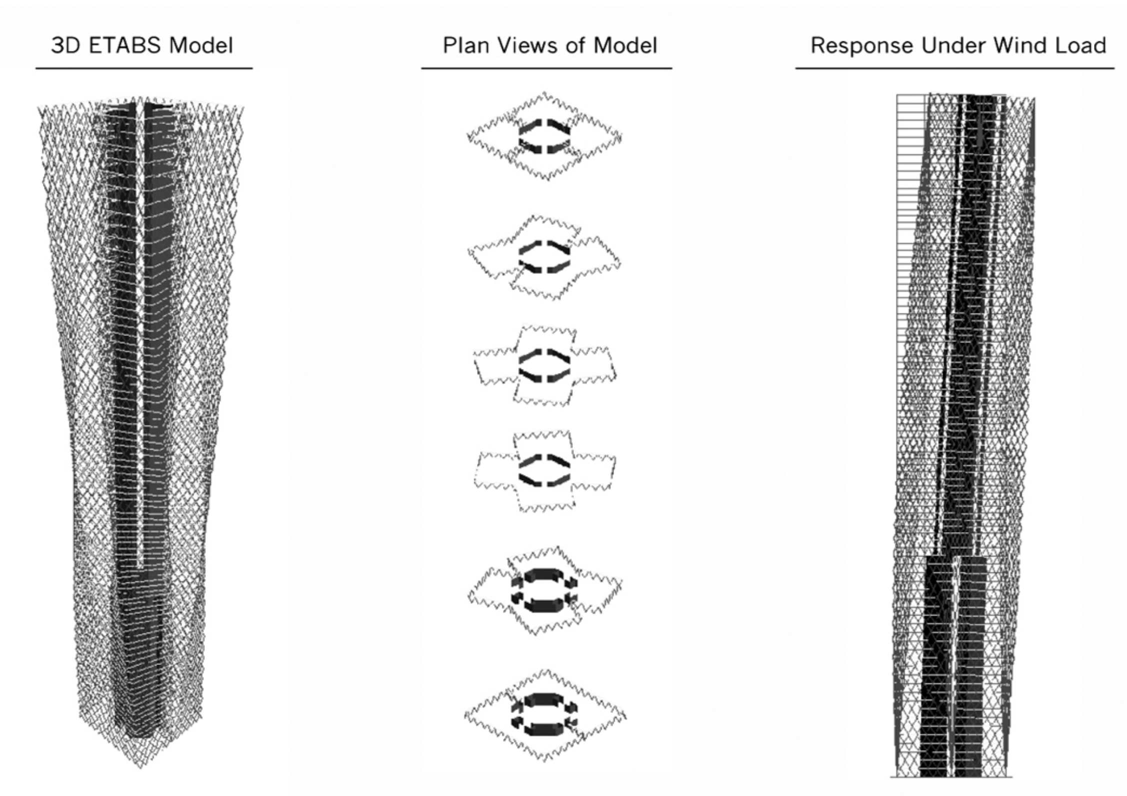


Figure 6f. Preliminary finite-element analysis model

6.4 *Foundations*

The foundation system for the Tower consists of a conventionally reinforced concrete mat supported by caissons. The mat thickness is varies from 3 to 4 meters with a concrete strength of C50. Under the Tower area, hand dug cast-in-place concrete caissons will support the mat. The caisson diameters are a minimum of 1200 mm and a maximum of 2000 mm. The foundation system for the podium / low-rise areas consists of reinforced concrete pile caps supported by piles. The piles consist either of driven precast, prestressed concrete or cast-in-place hand dug caissons. Precast piles are typically 400 mm x 400 mm square. A hydrostatic slab spans between pile caps. The mat and the piles caps will be fully waterproofed. A conventional perimeter reinforced concrete foundation wall is used around the site. The thickness of the wall varies from 300 mm to 500 mm. The outside of the foundation wall is waterproofed. The design water table is approximately 1.0 – 1.5 m below grade. Secant piles are used as the temporary retention system. Cross-lot bracing or tie-backs are used to laterally support the secant piles during construction. Dewatering of the site is required during construction.

6.5 *Construction*

Forming systems for the core area of the Tower and core framing are repetitive. The structural steel floor framing radiates from the core in the same manner on all floors with the only changes at perimeter conditions. Here a systematic floor-by-floor change is made to accommodate the twisting of the perimeter conditions and the reduction in spans. The core wall is conventionally formed with climbing or slip form systems. The exterior diagonal mesh is constructed from pre-fabricated steel “trusses.” These trusses extend the typical width of one face segment (varies from 24 m at the base to 20 m at the top). All trusses are field bolted at the mid-height of the diagonals. All geometric changes in the structural steel work for the mesh occurs at the joints allowing all diagonal members to be straight. This pre-fabrication and erection system allows for swift floor-to-floor construction. The reinforced concrete core wall system is constructed first, with the steel perimeter mesh frame and steel floor framing following and the metal deck slab and concrete following.

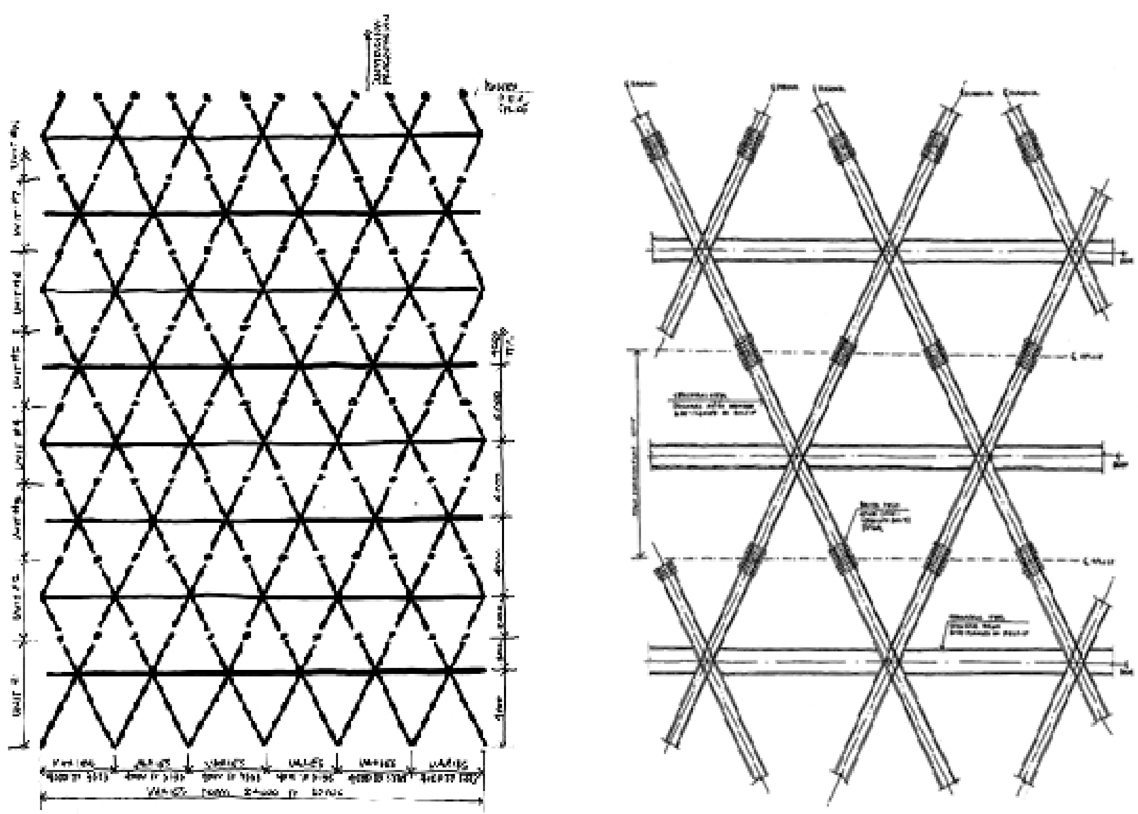


Figure 6g. Exterior frame construction sequence and details