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Performance-Based Evaluation for the 450m Nanjing Greenland Financial Center Main Tower

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As the Director in charge of Structural Engineering for SOM New York, Charles Besjak leads the structural engineering team to develop a diverse array of projects. As a licensed structural and professional engineer and licensed architect, he brings over 20 years of experience in the design and construction industry. He has supervised structural engineering for some of SOM's tallest building projects, including the 555-meter tall Lotte Super Tower in Seoul, Korea and zero energy 71 story Pearl River Tower in Guangzhou as well as Airports such as Terminal T3, Changi International Airport, Singapore and the 5 million sq. ft. Integrated Terminal at Chhatrapati Shivaji International Airport in Mumbai, India.

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"SOM has completed numerous projects in China which were super-tall and beyond the limits of the Chinese code, beginning with the Jin Mao Tower in Shanghai in the mid-1990's. Additional design and analysis measures are always required on these projects to prove their behavior and gain approval from seismic review panels and building authorities."

In order to obtain seismic review approval for the Nanjing State-Owned Assets & Greenland Financial Center's Main Tower, one of the tallest structures in the world to date, enhanced design measures and performance-based evaluations were utilized. The critical parts of the lateral system were designed for earthquake forces between two and six times that typically required by Chinese code. In addition a full 3-Dimensional Non-Linear Elasto-Plastic analysis for a 2500-year earthquake was completed to determine the structure's response and serviceability. A multi-stage axial shortening, creep and shrinkage analysis was also performed to evaluate the long-term load sharing between the central core and the perimeter of the Tower via the outrigger truss system.

Overview

The Nanjing State-Owned Assets & Greenland Financial Center Project (A1 Site) is a mixed-use development consisting of a 450-meter tall (1476'), 70-story office and hotel Main Tower; a 100-meter tall (328'), 22-story Accessory Office Tower; and a 7-story podium building linking the two towers and containing retail space, cinemas and hotel conference center. The total area above grade is approximately 197,000 square meters (2.1 million square feet). The 450-meter tower contains approximately 65,000 square meters of office space on levels 11 through 34 and 60,000 square meters of hotel, club, and restaurant space on levels 36 through 65. The project has 4 below-grade levels under the entire site with a partial mezzanine between the first basement floor and the ground floor. Total below-grade area is approximately 64,000 square meters. These floors contain retail, mechanical systems, hotel support, loading docks, car parking, and bike parking (see Figure 1.)
Across the street from the A1 Site is the Nanjing Greenland International Commercial Center Project (A2 Site), which is a thirteen-story multi-use building containing office, retail, dining and parking facilities. Surface parking is contained at basement Level B2. Retail, dining and atrium spaces occur from Level B1 to Level 3. Following are nine floors of office space with a partial mechanical floor and atrium at the top. Typical floor-to-floor heights are 6.3m at the retail floors and 4.2m at the office floors. The overall height of the building is 66.2m (217') above grade with a total area of 46,000 square meters (495,000 square feet).

Structural topping-out of the Main Tower was completed in September 2008. Cladding installation has been completed and interior fit-out is currently underway. When finished

Figure 1. Nanjing Financial Center Main Tower; Left: Architectural Rendering; Right: Construction Photograph

the Main Tower will be the 5th tallest building in the world according to the CTBUH criteria

The overall project was a competition that was awarded to the Chicago office of Skidmore, Owings and Merrill (SOM) in 2004. The schematic design and design development phases along with the seismic review process for the A1 Site were completed by SOM by the middle part of 2005 and then turned over to the Local Design Institute (LDI), East China Architectural Design and Research Institute (ECADI), for completion of the construction documents and construction administration phases. Schematic design for the A2 Site was completed by SOM in January 2005, and then turned over to ECADI to complete the remainder of the design phases. ECADI is the engineer of record for both the A1 and A2 sites.

Given the height of the Main Tower and the requirements for super-tall buildings which are well beyond the limits of the Chinese code, an extensive performance-based evaluation approach was employed. Particular emphasis and effort was put into the seismic design, analysis and review process including an elasto-plastic analysis on one of the tallest buildings in the world to date. The steps taken for the seismic design and approval of the Main Tower will be the primary focus of this paper.

Structural System for the Main Tower

The Main Tower consists of a composite system utilizing both structural steel and reinforced concrete elements to resist both gravity and lateral loads. Typical floor-to-floor heights are 6m to 7m in the podium zone, 4.2m in the office zone and 3.8m in the hotel zone. Mechanical floors are generally double-height spaces at 8.4m tall.

The lateral-load resisting structural system provides resistance to both seismic and wind loading. Refer to Figure 2 for a graphic of the overall lateral system. The primary lateral system is comprised of an interior reinforced concrete "super-core" shear wall system and exterior composite columns. Shear wall thicknesses range from 300mm to 1500mm over the height of the building with reinforced concrete link beams joining adjacent

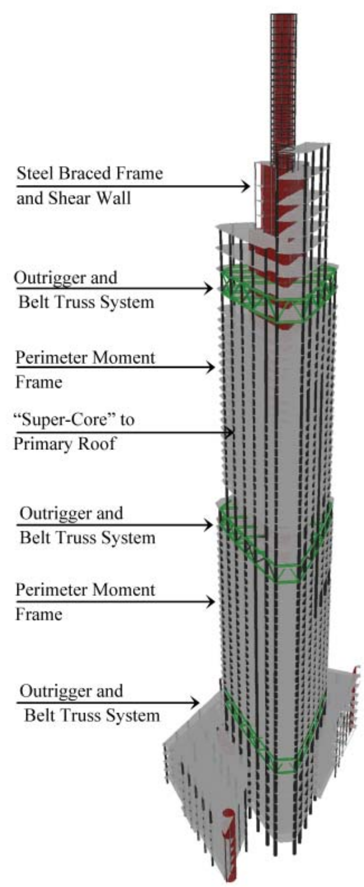


Figure 2. Main Tower Lateral System

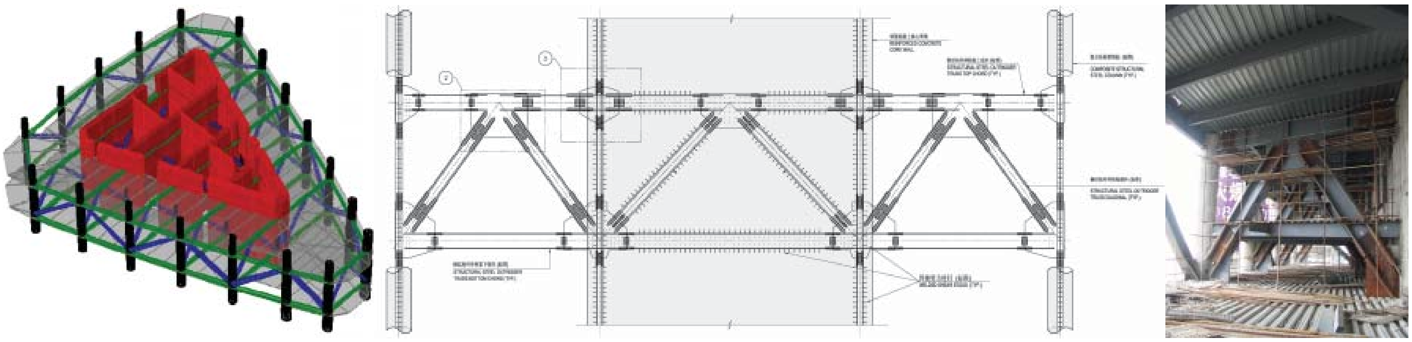


Figure 3. Left: Outrigger and Belt Truss Configuration; Middle: Typical Outrigger Truss Elevation; Right: Photograph of Outrigger Truss Construction

sections of shear wall around door openings and major mechanical penetrations. The closed form of the "super-core's" perimeter provides a large amount of the overall torsional stiffness of the building. The core wall thicknesses were optimized in order to better balance the triangular-shaped core for both bending stiffness and torsional rigidity. This resulted in thicker walls near the "tip" of the core for the trapezoid-shaped closed form and slightly thinner walls for the rest of the core. Figure 2 shows a photo of the core construction. The exterior composite columns are linked to the "super-core" by structural steel outrigger trusses at the 8.4 meter tall mechanical floors at Levels 10, 35, and 60.

...spiderman

"On a scale of one to 10, it was not even a one."

Alain Robert, the French skyscraper climber nicknamed 'Spiderman' was fined after scaling Aurora Place, a Sydney skyscraper without permission.

"It's one thing to be proud of your achievement, but it's another thing to be disrespectful of the laws as a guest in this country"

Sydney Chief Magistrate Graeme Henson, who fined Alain Robert for AU$ 750.
From 'Spiderman fined for scaling skyscraper' Agence France-Presse. June 3rd, 2009.

Outrigger trusses typically align with the web walls in the core and extend from the perimeter column through the core to the other perimeter column on the opposite side of the building. Figure 3 shows a typical outrigger and belt truss configuration at a major mechanical floor. Figure 3 is typical elevation of one of the outrigger trusses showing the proposed detailing. Note that the outrigger truss was carried through the core walls as an added layer of redundancy at the request of the seismic review panel. Embedded steel columns near the edges of the core walls were extended for a minimum of three floors above and below the outrigger trusses to aid in transferring the force couples developed under lateral loading. Figure 3 shows a photo of one of the outrigger trusses being erected. The exterior composite columns at these levels are linked together by a structural steel belt truss system at the perimeter to provide a more uniform load distribution in the columns. A portion of the belt truss system can be seen in the photo of Figure 3. Composite column sizes range from 900mm diameter to 1750mm diameter over the height of the building. From Level 63 to 67 a portion of the reinforced concrete core continues up in combination with a braced steel frame to form the lateral system. Above Level 67 to the Roof at 381m, the lateral system consists of small reinforced concrete core and a perimeter moment frame structure. A structural steel spire continues to 450m. The secondary lateral system for the Main Tower consists of a moment-resisting frame at the perimeter of the building. The perimeter moment frame system provides additional torsional stiffness, structural integrity, and redundancy for the overall building.

The gravity load-resisting structural system consists of structural steel floor framing supporting a 155mm thick composite metal deck floor slab. Typical floor framing is spaced at 3 meters on and welded, headed shear studs are used to provide composite behavior between the slab and supporting beams. Floor framing inside the "super-core" consists of reinforced concrete beams supporting a reinforced concrete one-way slab. The central reinforced concrete "super core" and the exterior composite columns then transmit the floor framing loads to the foundations. Refer to Figure 4 for typical floor framing plans in the office and hotel portions of the building, respectively.

The below grade levels were constructed of reinforced concrete using a temporary, internally-braced slurry wall retention system. A permanent reinforced concrete foundation wall was then constructed inside of the slurry wall system. The foundation system for the Main Tower consists of a 3500mm thick, cast-in-place reinforced concrete mat under the entire footprint of the building supported by cast-in-place reinforced concrete belled caissons in the underlying rock.

Lateral Loading Requirement and Evaluation

Both wind and seismic loading were evaluated in the analysis and design of the Main Tower.

A 100-year return period wind was required for this project due to the height of the building. Wind tunnel testing was performed by RWDI

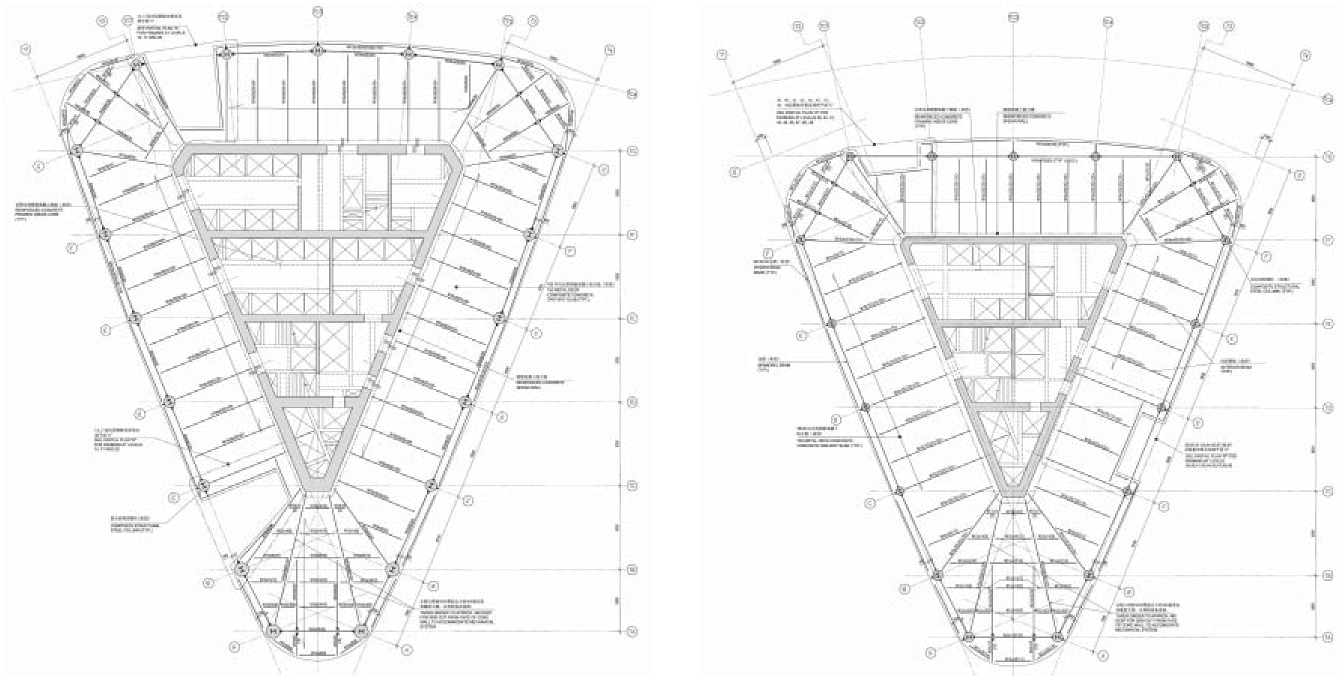


Figure 4. Left: Typical Office Floor; Right: Typical Hotel Floor

Laboratories in Ontario, Canada to determine more accurately the actual wind pressures applied to the building as well as the translational and torsional accelerations experienced at different levels. In general, the loads determined by the wind tunnel were substantially lower than those required by the Chinese code and were used for both serviceability checks. Per Chinese Code requirements, the interstory drift ratio under the 100-year wind load could not exceed 1/500. Strength design was done using forces calculated from the code.

Seismic requirements in the Chinese Code are somewhat different than those encountered in many other building codes around the world. There are three separate levels of earthquake that are considered depending on the type, height and complexity of the structure:

- Frequent Earthquake - 63% chance of being exceeded in 50 years (50-year return period)
- Moderate Earthquake - 10% chance of being exceeded in 50 years (~ 500-year return period)
- Major Earthquake - 2% chance of being exceeded in 50 years (~ 2500-year return period)

For small to medium buildings without irregularities, only the Frequent earthquake is generally used for all strength and serviceability checks.

Nanjing is defined as Seismic Intensity VII which is roughly equivalent to a Zone 2A per the UBC Code. A site-specific seismic evaluation study was done on the site, and it was found that a fault line runs through it. This led to an increase in the parameters provided for use in creating Response Spectrum and Time History curves. As an example, the peak value on the site-specific response spectrum curve for the Frequent earthquake was 50% higher than that required by the basic code values for Nanjing. From a serviceability standpoint, interstory drift ratios under the Frequent earthquake were also not to exceed 1/500 per code.

Comparing the wind tunnel loads with the site-specific response spectrum for the Frequent earthquake, it was found that wind load controlled in the weak direction of the Tower (narrow direction of the core) while seismic controlled for the strong direction.

Seismic Design and Review Process

The Main Tower at 450m in height is substantially over the code limit of 190m for a concrete core-steel frame structure. In addition there were vertical and horizontal irregularities created by transfer elements at the major mechanical floors, diaphragm cutouts at various floors over the height of the building and torsional movements near the base of the Main Tower where it supported the majority of the lateral loads on the Podium structure. As a result of the height and the irregularities, the Main Tower was defined as an Over-Limit and Complex structure per the Chinese Code. This resulted in additional measures required for analysis and design and for the seismic review process. A performance-based evaluation approach would be required to satisfy the seismic experts and building authorities that the Tower would be safe and behave appropriately.

One of the primary structural requirements for the Tower was the implementation of "Super Grade I" design and detailing for major components of the lateral system. This involved amplification factors on the seismic loads for the core walls and the perimeter moment frame system as well as large increases in the size and reinforcing details for boundary elements within the core wall system.

SOM has completed numerous projects in China which were super-tall and beyond the limits of the Chinese code, beginning with the Jin Mao Tower in Shanghai in the mid-1990's. Additional design and analysis measures are always required on these projects to prove their behavior and gain approval from seismic review panels and building authorities.

The seismic review process for the Main Tower first began in the April 2005 in the early part of design development. Due to the size and nature of the Tower, a national panel of experts from universities and design institutes from various parts of China was assembled. SOM presented the structural system and behavior with the assistance of ECADI, who was required by Chinese Code to develop their own separate, concurrent structural model for comparison with SOM's ETABS model. Knowing that the structure was beyond the code limits and that additional measures would surely be required, SOM suggested in this initial meeting that all structural members of the lateral system should be designed to remain elastic under the site-specific response spectrum for a Moderate earthquake rather than the code-prescribed Frequent earthquake. The seismic experts agreed this was an appropriate approach but suggested the use of the Code-prescribed response spectrum for the Moderate earthquake in lieu of the site-specific values. Discussions during this meeting led to several additional measures:

- An elasto-plastic time history analysis for the Major earthquake would be performed to verify overall structural behavior and determine any weak points in the structure.
- The core walls would be designed for the shear forces resulting from the Major earthquake.
- The outrigger trusses and belt trusses would be designed to remain elastic under the Major earthquake.

SOM developed the following table to summarize the performance-based evaluation approach that would be utilized including the purpose of and requirements for the Frequent, Moderate, and Major earthquakes as well as the Elasto-Plastic analysis. This served as a useful tool for guiding the process as well as summarizing the approach for review by the seismic experts at subsequent meetings.

Reviewing Figure 5 Parts A and B, it is seen that all members of the lateral system were designed for the larger of:

- The Frequent earthquake using the site-specific Frequent response spectrum, factored load combinations, reduced material design values, and all "Super Grade I" amplification factors;
- The Moderate earthquake using the code-specified Moderate response spectrum, factored load combinations, reduced material design values, but no "Super Grade I" amplification factors.

In addition to the seismic forces, all members were checked against the 100-year Code-prescribed wind loads for strength. The overall structure was then checked for serviceability interstory drift ratios for both 100-year wind tunnel loads and the site-specific response spectrum for the Frequent earthquake.

In Part C, the additional measures taken for the shear walls and outrigger/belt truss systems are documented. Because of the importance of the outrigger and belt trusses in transferring load between the interior and exterior systems and in controlling the drifts of the building under seismic loads, the forces in the trusses were designed for the Major earthquake using the code-specified Major response spectrum with service-level load combinations, unreduced material design values and no "Super Grade I" amplification factors. Similarly, since the majority of the shear forces on the structure are taken by the core walls and an alternate load path to carry these shear forces does not exist, the shear forces in the walls were designed for the Major earthquake using the code-specified Major response spectrum with service-level load combinations, unreduced material design values and no "Super Grade I" amplification factors.

Lastly in Part D, an elasto-plastic analysis was performed to further confirm the structure's behavior assuming that hinges could form in some members of the lateral system and that the forces in the outrigger and belt trusses and the shear in the core walls did not exceed the elastic design values accounted for in the response spectrum analysis for the Major earthquake. The interstory drift ratios were also checked to verify that acceptable movements were occurring. Three separate time-history curves were used that had been scaled up by six times, provided by the local geotechnical engineer for Frequent earthquakes to simulate the Major earthquake event. Two of the time history curves were scaled versions of actual earthquake records

...stronger than ever

"We are now in the middle of the 12th serious downturn since New York became a major financial center in the early 19th century. The lesson of every single one of these previous 11 busts is that the city always comes back stronger than ever. History is perfect on that one."

Dan Doctoroff, president of Bloomberg LP in NY. He was the former deputy mayor for economic development under Mayor Bloomberg. From the NY Times Magazine of March 15, 2009. The article is 'After the Bubble,' pages 47-49, written by Jonathon Mahler

	Part A	Part B	Part C	Part D
	Minor Earthquake	**Medium Earthquake**	**Major Earthquake**	**Elasto-Plastic Analysis**
Probability of Occurance as defined by Code	63% in 50 years	10 % in 50 years	2-3% in 50 years	
Purpose	Conform to Code requirements	Additional Measures and Analysis to overcome Over-Code-Limit conditions and answer Experts' comments		
Targets	Strength Design and Deformation Check	Strength Design for Entire Building	Strength Check at special areas	Deformation check; Define Weak Portion; Torsional effects
Seismic Input Data	Response Spectrum Curve and Time History data	Code Specified Response Spectrum Curve	Code Specified Response Spectrum Curve	Time History using 6x frequent earth-quake time history LDS1-LDS5
Interstory Drift Ratio and Building Deformations	Shall be restricted to elastic inter-story drift of L/500 (JGJ2002-4.6.3) < L/500	----	----	Will be checked to plastic deformation of (JGJ2002-4.6.5)
Member Strength Design	Members to be designed according to code as stated below	Elastic design according to seismic experts' comment as stated below	1) Shear check in inner areas of Shear Wall 2) Outrigger & Belt Truss strength Check 3) Transfer at Level 36 check 36	
Load Factors	Factored Load Combinations	Factored Load Combinations	Service Load Combinations	Service Load Combinations
Amplification Factors	Stress amplification values as per code requirement for Super grade 1	Stress amplification value of 1.0	Stress amplification value of 1.0	Stress amplification value of 1.0
Material Strength	Material strengths based on Design Values	Material strengths based on Design Values	Material strengths based on Standard Values	Material strengths based on Standard Values
Additional Lateral Force	Strength design per Code Wind; Deformation per Wind Tunnel	----	----	----

Figure 5. Summary of Analysis and Design Approach for Seismic and Wind Loading

while the third was a simulated earthquake record. The methodology and results of the elasto-plastic analyses will be described in greater detail below.

The next seismic review meeting was held in early July 2005, a few weeks before the end of the design development phase to present the progress of the design approaches noted above and incorporate the expert's requirements from the first meeting. For the most part everything was satisfactory to them with a few additional requests related to clarifying certain design procedures used and some additional information on particular detailing elements.

SOM's design continued until the end of the design development phase at the end of July 2005 at which time a formal seismic review calculation report was assembled and presented at the third seismic review meeting. This report was several hundred pages and documented the overall design of the structure as well as resolutions to the expert's recommendations and requirements from the previous meetings. Concurrently, SOM performed a Staged Construction and Creep-Shrinkage Analysis to determine long-term load transfer between core wall and the perimeter column via the outrigger truss system. At the conclusion of this meeting, seismic design approval was granted for the project. A handful of comments were made related to additional design considerations to be incorporated by ECADI during the construction document phase. Given the size and complexity of the project, the seismic review process went very smoothly with a limited number of review meetings. The performance-based evaluation approach taken by SOM including the enhanced design measures, creep and shrinkage analysis, and elasto-plastic analysis resulted in a very efficient and successful structure.

Nonlinear Elasto-Plastic Transient Dynamic Analysis Using Time History Curves

A three dimensional Transient Dynamic Analysis with material nonlinearity was performed to determine the rare earthquake (2% in 50 year probability) demand on the building's structural system. The Nonlinear Time History Analysis was carried out in order to evaluate the maximum drifts and verify that they were less than the allowable code maximum elasto-plastic drift ratio limit as per Chinese code. Work done by outriggers and belt-truss members were analyzed and compared to member capacity designed by elastic analysis so as to confirm that they remain elastic during the Major earthquake event.

Nonlinear Static Pushover Analysis versus Nonlinear Elasto-Plastic Time History Analysis

In the case of nonlinear static pushover analysis, usually the response spectrum curve representing the occurrence of a Major earthquake is applied to the elastic model and the generated story shears are used for loading purposes. A static load equal to the above mentioned story shears are applied in increments to the model to generate hinge formations and corresponding stress redistribution in the lateral system. After the entire load has been applied, the building interstory drift is plotted and compared to the allowable limit as per code. Another method involves incremental loading of the structure until target deflection is exceeded, resulting in forces generated in the members appropriate to a major earthquake and observed hinge formation and corresponding stress redistribution in the lateral system. This method is an approximation of the seismic response since it is a static load and not actual forces generated by accelerations from a time history curve.

On the other Hand a more exact method for seismic response is nonlinear elasto-plastic analysis, where accelerations from at least three time-history curves are applied to the model to generate hinge formations and corresponding stress redistribution in the lateral system. The structure is analyzed for each of the 3 time-histories in very small time step increments (50steps/second) for a total duration of 3-4 times the primary building period. With up to 10 iterations at every step in order to achieve equilibrium, this is a very intense analysis and requires significant computational time. At the conclusion of the required duration of the time history, building interstory drift for each time step is recorded and the maximum at any given time is plotted and compared to the allowable limit as per code.

For the performance based evaluation of Nanjing Greenland Financial Center Main Tower the more accurate 'Nonlinear Elasto-Plastic Time History Analysis' was employed.

Three Dimensional Nonlinear Modeling

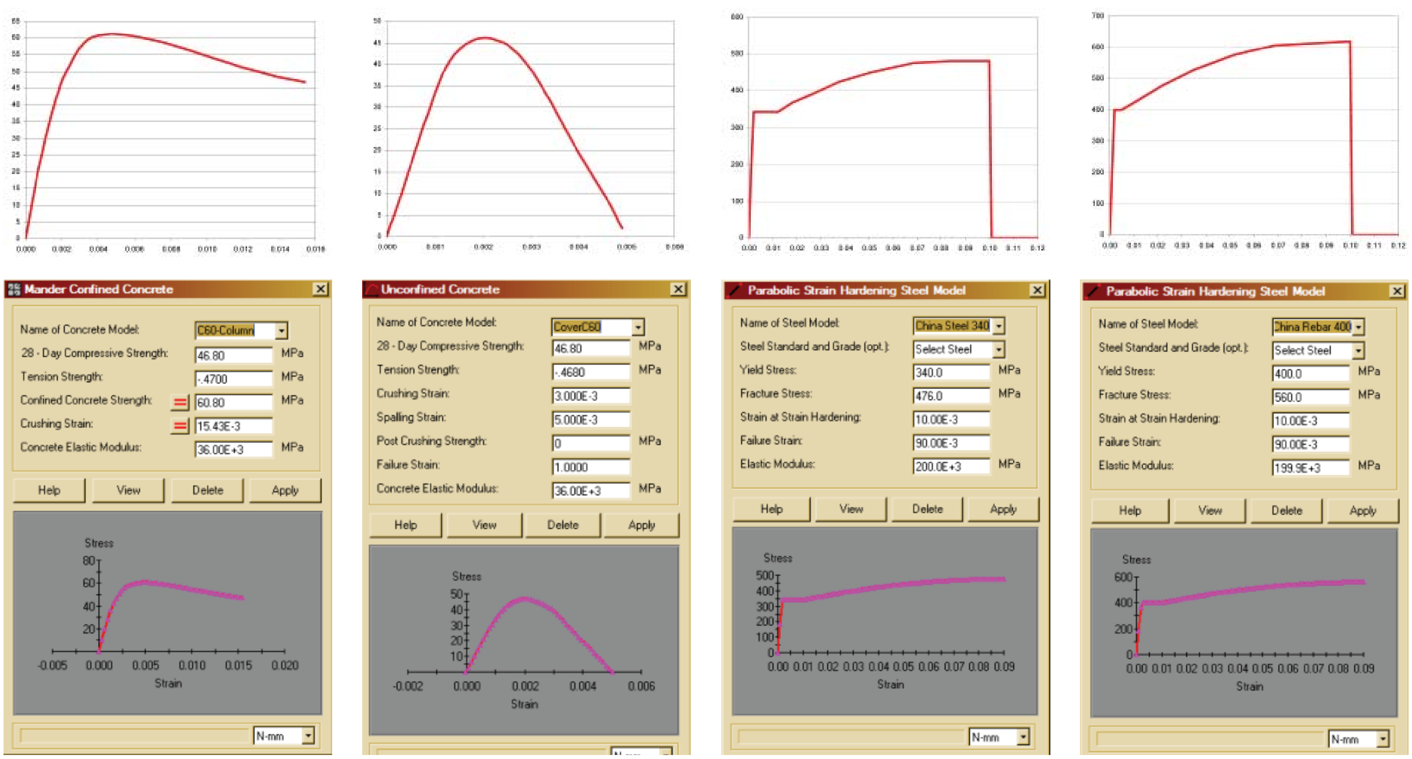
Only the elements that were part of the lateral system of the structure were modeled with nonlinear properties. These include reinforced concrete shear wall supercore, perimeter moment resisting frames comprised of steel beams and composite columns; and built-up structural steel outriggers and belt-truss connecting the supercore to the perimeter moment frame. The nonlinear model was built in SAP2000 V8 Non-Linear product of CSI (Computer and Structures, Inc.).

Mass and Rigid Diaphragms

Nodes at every level were linked with rigid diaphragms. A rigid diaphragm slaved the lateral displacement and the in-plane rotation of the nodes connected to it. The seismic mass was calculated from the self weight of the structure and applied superimposed loads.

Gravity Loads

For an elasto-plastic time history analysis the effect of the dead load on the modeled elements was important. The dead load was used to "pre-load" the structure before applying the earthquake time history, resulting in initial stressing of the members. Loads in the model were applied as area loads on shell elements (slabs) and line loads on horizontal linear elements (beams).



Top left to bottom right: Figure 6a: C60 Concrete Model Stress-Strain Curve - Confined Concrete; 6b: C60 Concrete Model Stress-Strain Curve - Unconfined Concrete; 6c: XTRACT Input for Concrete - Confined Concrete; 6d: XTRACT Input for Concrete - Unconfined Concrete; 6e: Steel Model Stress-Strain Curve - Q420 Grade Structural Steel; 6f: Steel Model Stress-Strain Curve - HRB400 Grade Reinforcement; 6g: XTRACT Input for Structural Steel and Reinforcement - Q420 Grade Structural Steel; 6h: XTRACT Input for Structural Steel and Reinforcement - HRB400 Grade Reinforcement

Software, Model, Material Properties, Elements Description and Hinges

Software

The software used for modeling was SAP2000 V8 Nonlinear, a finite element software product of Computer and Structures Inc. In order to run a non-linear analysis the software requires the elastic elements to be defined with nonlinear hinges; and since nonlinear hinges can only be applied to frame elements, all shear wall elements were modeled as vertical frame elements and connected together using rigid links. At each time step of the elasto-plastic analysis, the software solves equations for the entire structure, locating the formation of nonlinear hinges and redistributing the force level accordingly before proceeding to the next time step.

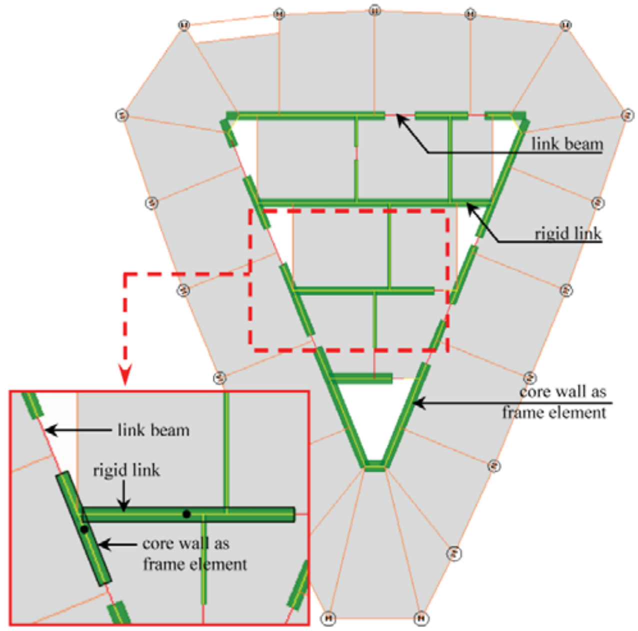


Figure 7. Structural 'Frame' elements in SAP Model

Simplified Frame Model

For the purpose of elasto-plastic analysis, a simplified frame model of comparable structural properties was built and compared to the ETABS elastic model in which the shear walls were modeled as shell elements. The two models were found to be comparable to each other in terms of their net reactions at base, building modes, modal mass participation ratios, etc.

Material Properties of Concrete

To accurately capture the nonlinear behavior of the elements, realistic material models were used for the different concrete strengths required by design. The concrete stress-strain relationship is related to the reinforcement and the confinement of the section. To represent the different concrete material possibilities, six different concrete models were set up: they were with properties for confined and unconfined C50, C60 and C70 respectively. The stress-strain curves are based on Mander's model for concrete behavior with confining stresses computed from the detail properties. As an example C60 material property and corresponding inputs into XTRACT are shown in Figure 6a.

Typical Material Properties assigned in the analysis program – XTRACT are based on the following assumptions. As an example material properties of C60 are listed below:

- Confined C60 Concrete
  - 28 days compressive cylinder strength, fc = 60x0.78=46.8MPa
  - Tension Strength ~ 1% fc = 0.468MPa
  - Confined Concrete Strength ~1.3 fc = 60.8MPa
  - Elastic Modulus, Chinese Code, C60 = 36000MPa
- Unconfined C60 Concrete
  - Crushing strain = 0.003
  - Spalling strain = 0.005
  - Failure strain = 1.00

Material Properties of Steel

A strength hardening steel model was used as a basis for the structural steel and the reinforcing bars behavior. The steel material model assumes symmetrical behavior for both compression and tension. As an example grade Q420 structural steel and grade HRB400 reinforcement material property and corresponding inputs into XTRACT are shown in Figure 6b.

Elements Description

Shear Wall Elements

As mentioned before, since nonlinear hinges can only be applied to frame elements, shear walls were modeled as vertical frame elements with same geometry, material properties and thus same stiffness as the shell elements. Frame elements were linked to each other with infinitely stiff rigid links to insure similar 3D behavior as walls (see Figure 7). Link beam were also connected to the infinitely rigid links at the face of the frame elements modeled as walls. Hinges were inserted to model the nonlinear behavior of both shear walls and link beams.

Column and Beam Elements

Columns and beams were modeled using line elements. These elements were assigned the actual geometry of the section and the elastic material properties. Nonlinear hinges were defined and inserted to model the nonlinear behavior.

Slabs Elements

The slab elements were modeled as shell elements and have only elastic properties; they have no nonlinear properties attributed to them.

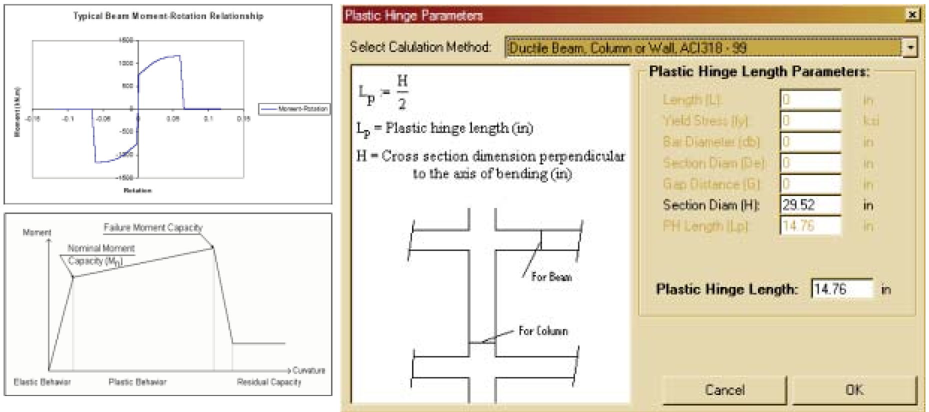


Figure 8. Top Left: Typical Beam Moment-Rotation Relationship; Bottom Left: Idealized Moment Curvature of a Reinforced Concrete Element; Right: Plastic Hinge Length

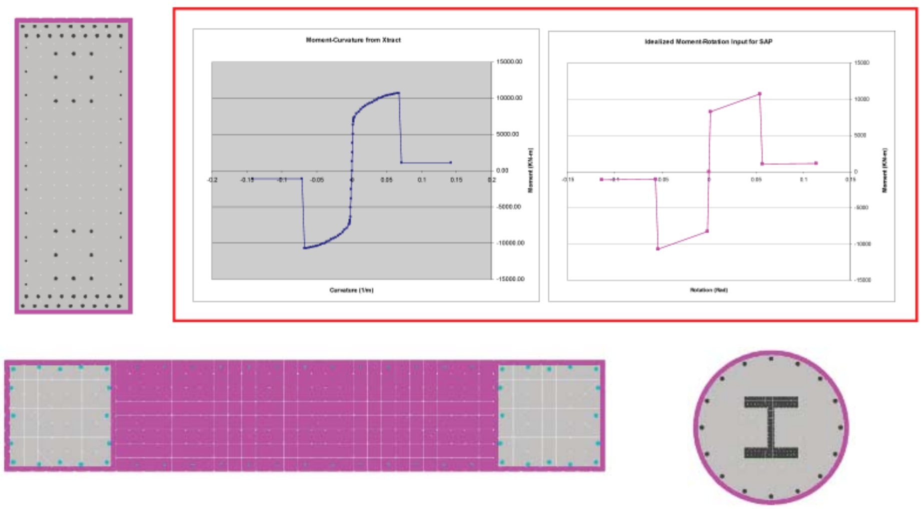


Figure 9. Top Left: Typical link beam finite element cross section in XTRACT; Top Right: Link beam moment curvature from XTRACT and corresponding moment-rotation input into SAP2000; Bottom Left: Typical core-wall finite element cross section in XTRACT; Bottom Right: Typical composite column finite element cross section in XTRACT

Hinge Definition

Nonlinearity was implemented in the model by inserting nonlinear hinges in elements. Nonlinear hinges are moment rotation relationship (Figure 8) of the sections computed for each section based on the member size, reinforcement detailing and axial load.

Hinge Properties

A plastic hinge in a member is taken to occur when the concrete compression strain reaches 0.003, or when the steel reinforcement reaches the yield strain at approximately .00207 (depending on steel grade), whichever occurs earlier; and the moment capacity of the section reaches M_n nominal moment capacity. Beyond this stage the member is said to be in the plastic behavior range till it reaches failure moment capacity. However, the member still has reserve capacity beyond this stage as can be seen from the idealized moment curvature diagram in Figure 8. In the deflection plots, the lights representing the hinges will turn ON as soon as the member reaches its nominal capacity M_n.

Plastic Hinge Length

Plastic hinging was assumed to occur according to ACI 318-99 ductile beam column or wall behavior (Figure 8). This procedure assumes that the plastic hinge has a length of half the member depth or member length (smaller of the two) in the direction of bending moment.

General Procedure

The computer program XTRACT (product of Imbsen and Associates Inc.) was used to determine the moment curvature relationship for all the different section types. This program requires the input of basic material non-linear properties, section geometry, reinforcement, as well as confined and unconfined concrete layout (see Figure 9). The program produces the moment-curvature relationships for a given axial load, assuming plane sections remain plane. The curvature is then multiplied by the plastic hinge length, which results in a moment-rotation relationship. Figure 9 shows the Moment –Curvature output from XTRACT and the corresponding Moment-Rotation relationship input into SAP2000.

Hinge Explanation

SAP2000 represents the formation of hinges with colored dots. When a color dot appears in a member, it means that a hinge has formed in that location. The color of the dot is representative of the state of the hinge. When the moment in the hinge reaches the nominal yield moment capacity of the hinge, a Pink dot will appear. The dot will remain pink through the ductile behavior part of the curve. When the hinge moment reaches the ultimate capacity of the member, the dot will turn yellow. From that point on, the moment will drop to a residual value, and the dot will turn orange. When the hinge reaches its curvature limit, the member fails and the hinge turns red. Figure 11 shows a generic moment curvature curve for a hinge and how color is reported for each state.

Analysis Methodology

Analysis Steps

Each elasto-plastic analysis consists of two separate nonlinear analyses that are appended to give one set of results. The first nonlinear run constitutes a load pattern representing the self weight and the superimposed dead load on the structure. This step is very important since it preloads the model non-linearly with gravity loads. The second nonlinear analysis is the integrated time history analysis. It consists of applying recorded (or simulated) earthquake accelerations on the structure.

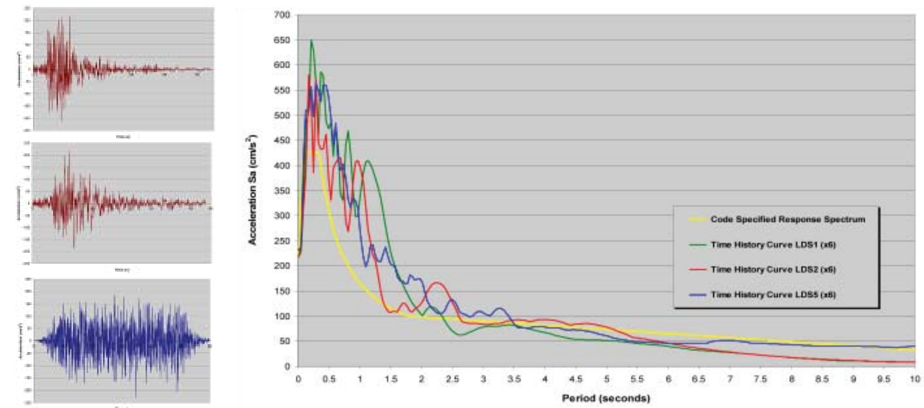


Figure 10. Top Left: Real Time History Curve - LDS1; Middle Left: Real Time History Curve - LDS2; Bottom Left: Simulated Time History Curve - LDS5; Right: Response Spectrum and Time History Curve Comparison

Analysis Directions

Due to the geometry of the structure and its differential stiffness along X and Y axis, acceleration from the different time histories had to be applied in both directions. For the Nanjing Greenland A1-Tower, the time history analyses were run individually for X and Y directions. Results from both the X and Y directions were plotted and compared to code limits.

Time Histories

A total of five time history curves for Frequent earthquakes were magnified by a factor of 6 to establish a rare earthquake level to be used in ETABS Elastic analysis. Base shears from the same were tabulated and compared to the code-specified response spectrum for a rare earthquake. This comparison formed a basis for the choice of the time history curves for Elasto-plastic analysis. LDS1, LDS2 and LDS5 were found to meet the criteria from the Chinese Code; which states that base shear from individual time history curve has to exceed 65% of the response spectrum base shear and the average base shear of three time history curves have to exceed 80% of the response spectrum base shear. Thus 2 recorded (LDS1and LDS2) and 1 simulated (LDS5) time histories were applied to the model for the Elasto-plastic Time History Analysis. The peak ground accelerations of modified time history curves LDS1, LDS2 and LDS are around 220cm/sec^2, thus conforming to Chinese code-required peak acceleration for Nanjing.

Additionally, all 3 time histories were converted to response spectra and compared to the code response spectrum for Nanjing. 7.5% damping ratio was considered to account for additional energy dissipation due to the inelastic structural behavior. The nonlinear spectral analysis program BI-SPEC constructs linear spectra for an earthquake. The mass, the damping ratio, and the period range were specified. A response spectrum curve was generated by computing the deformation response of a SDF system due to ground acceleration by numerical methods, and repeating the steps for each time step graphically to produce the response spectrum corresponding to a given set time history record (see Figure 10).

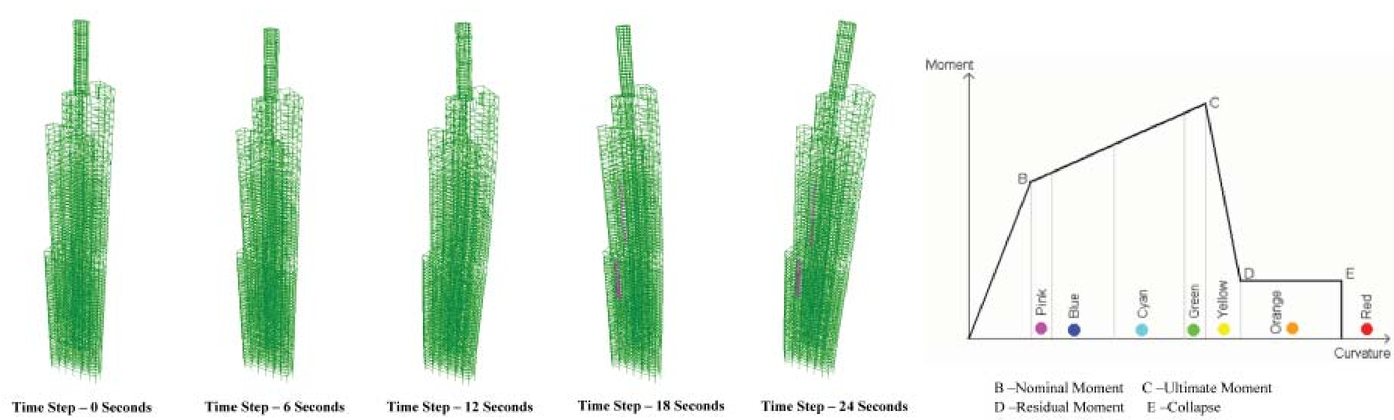


Figure 11. Left 1-5: Building Response for Time History LDS2 - Y Direction; Right: SAP 2000 Hinge Color Coding © CSI

Nonlinear Time History Analysis Evaluation

The evaluations of nonlinear transient dynamic analysis results using time history provided us the ability to understand the exact locations of hinge formations (seen as pink dots in Figure 11), the status of the hinge (the Status of all hinges was between Nominal and Ultimate moment capacity) and the corresponding time step at which it occurred. The analysis of the animations provided us the exact behavior of the building through the entire time duration and at any particular time step of the given time history curve.

Maximum building displacements for the 3 separate time histories in both X and Y direction were evaluated and it was seen that the maximum roof displacement under modified time history curve LDS2 in Y Direction equal to 894mm, which yields an overall building drift of 0.943 / 381 = 1 / 359, smaller than the 1/100 code-allowable elasto plastic drift limit.

For all the 3 Non-linear time history analyses in both X and Y directions, interstory drift at each time step increment of 0.02 seconds was computed and a maximum value summarized, an example of which is shown in Figure 12. These tables show that maximum interstory drift and the corresponding Time Step during the Time-History Analysis of the structure, under each of the earthquakes and in each direction.

The Chinese code states that the maximum allowed elasto plastic interstory drift ratio limit is 1/100 for Concrete Shear Wall – Moment Frame Buildings and 1/50 for Structural Steel Structures. It was seen that drifts were within the code-prescribed limits. The graphs on Figure 13 show that maximum Interstory Drift Ratio for all the 3 Earthquakes (in both X and Y directions) meets the 1/100 for the Building and 1/50 criteria for the Structural Steel Spire.

Story	Time Step	Drift (mm)	Interstory Drift Ratio (L∫)	< 1/100 for Building, < 1/50 for Steel Spire
Spire3	8.16	139	60	Yes
Spire2	8.16	270	62	Yes
Spire1	8.08	207	81	Yes
Roof	16.70	43	196	Yes
66M	16.52	29	288	Yes
66	16.50	19	290	Yes
65	16.50	19	293	Yes
64	16.48	19	299	Yes
63	16.48	26	326	Yes
62	16.48	25	342	Yes
61	16.46	21	397	Yes
60	16.46	24	355	Yes
59	16.46	14	348	Yes
58	16.44	11	347	Yes
57	16.44	11	345	Yes
56	16.42	11	344	Yes
55	16.42	11	343	Yes
54	16.42	11	342	Yes
53	16.40	11	341	Yes
52	16.40	11	341	Yes
51	16.40	11	642	Yes
50	16.38	11	345	Yes
49	16.38	11	346	Yes
48	16.38	11	348	Yes
47	16.38	11	351	Yes
46	16.38	11	354	Yes
45	16.38	11	357	Yes
44	16.36	11	362	Yes
43	16.36	10	367	Yes
42	16.36	10	373	Yes
41	16.36	10	380	Yes
40	16.36	10	387	Yes
39	16.34	10	396	Yes
38	16.34	10	407	Yes
37M	16.34	10	422	Yes
37	16.34	19	450	Yes

Story	Time Step	Drift (mm)	Interstory Drift Ratio (L∫)	< 1/100 for Building, < 1/50 for Steel Spire
36	16.26	15	575	Yes
35	16.26	8	526	Yes
34	16.26	8	522	Yes
33	16.24	8	526	Yes
32	16.20	8	532	Yes
31	16.16	8	538	Yes
30	16.16	8	545	Yes
29	15.86	8	550	Yes
28	15.84	8	552	Yes
27	15.82	8	555	Yes
26	15.82	8	559	Yes
25	15.80	7	563	Yes
24	15.80	7	569	Yes
23	15.80	7	577	Yes
22	15.80	7	592	Yes
21	15.80	7	601	Yes
20	15.80	7	612	Yes
19	15.80	7	624	Yes
18	15.80	7	638	Yes
17	15.80	6	655	Yes
16	15.80	6	674	Yes
15	15.80	6	698	Yes
14	15.82	6	727	Yes
13	15.84	6	762	Yes
12	15.86	5	830	Yes
11	15.90	8	1058	Yes
10	15.90	5	925	Yes
9	15.90	9	888	Yes
8	15.90	7	937	Yes
7	15.90	7	1023	Yes
6	15.90	5	1158	Yes
5	15.90	5	1330	Yes
4	8.44	4	1579	Yes
3	8.44	3	1974	Yes
2	8.44	2	3226	Yes

Figure 12. Example of Maximum Interstory Drift and their Ratios for Time History LDS1 - X Direction

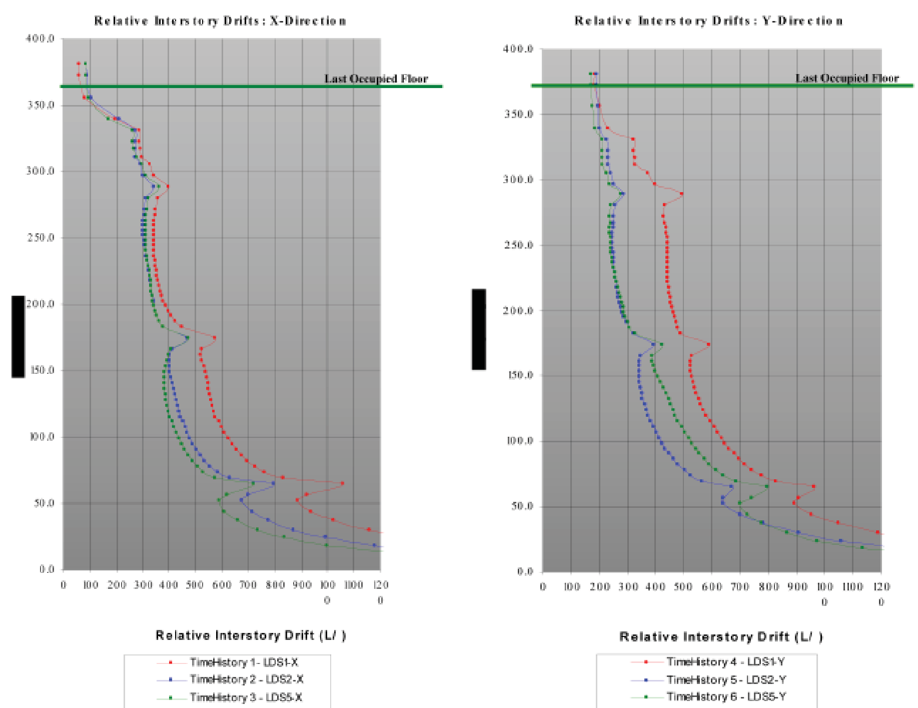


Figure 13. Example of Maximum Interstory Drift and their Ratios for Time History LDS1 - X Direction

Conclusion

The Nanjing State-Owned Assets & Greenland Financial Center Project's 450m tall Main Tower will soon be the 5th tallest building in the world. In order to obtain seismic approval from the national panel of experts, several analyses and design approaches over and above the Chinese Code were required including elastic design of all members for a 500-year earthquake and elastic design of some key elements for a 2500-year earthquake. Finally the elasto-plastic analyses were performed to verify the structure's behavior under the 2500-year earthquake. This cumulative seismic design effort has resulted in one of the tallest structures in the world to date and represents the state-of-the-art in performance-based evaluation. As other super-tall structures continue to be built in seismic regions in China and around the world, this project can serve as a basis for design approach and methodology to ensure safe and economical structures. ■