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Authors:	Mark Sarkisian, Director, Skidmore, Owings & Merrill Dasui Wang, Chief Engineer, ECADI Sam Lee, Guangzhou Scientific Computing Consultants Co. Ltd. Neville John Mathias, Associate Director, Skidmore, Owings & Merrill
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Mark Sarkisian

Dasui Wang

Sam Lee

Neville Mathias

Authors

Mark Sarkisian, Director
Neville Mathias, Associate Director
Skidmore, Owings & Merrill LLP (SOM)
One Front Street, San Francisco, CA 94111, USA
e: somsanfrancisco@som.com

Dasui Wang, Chief Engineer
East China Architectural Design & Research Institute
No.151 Hankou Rd. , Shanghai 200002, China
www.ecadi.com

Sam Lee
Guangzhou Scientific Computing Consultants Co.
507/140 Donfeng Xi Rd. , Guangzhou 510170, China
e: szslee@gz-scc.com

Mark Sarkisian
Mark P. Sarkisian is the Director of Seismic & Structural Engineering at SOM San Francisco. He is also a member of the CTBUH Advisory Group. Mark holds four US Patents for high-performance seismic structural mechanisms designed to protect buildings in areas of high seismicity and has additional patents pending for seismic and environmental-ly-responsible structural systems. He currently teaches an Integrated Studio class that includes students from Berkeley, CCA, Stanford University & Cal Poly.

Dasui Wang
Dasui Wang is the Chief Engineer of ECADI, a China Design Master and member of the expert panel of the China Construction Ministry for review of code exceeding buildings. He is also Associate Director of the High-rise Building Committee of the China Architecture Institute and Professor of Tong Ji University. He has been responsible for the engineering of many important projects – Shanghai East Pearl TV Tower, Pudong International Airport, Shanghai WFC, China Center TV Tower, Jinta Tower, etc. He has taken part in developing the China Steel Structure Design Code and Tall Building Structure Design Standard.

Sam Lee
Dr. Sam Lee, prior to co-founding Guangzhou Scientific Computing Consultant Co. Ltd., was working as structural engineer for the Architectural Design and Research Institute of Guangdong Province, Guangzhou, China. He is a pioneer in the area of nonlinear dynamic earthquake analysis of tall buildings, and is responsible for the analysis of many landmark buildings in China – Shanghai WFC, China Center TV Tower, Jinta Tower, Tianjin 117 etc. He is also a member of the CTBUH Journal Editorial Board.

Neville Mathias
Neville Mathias is an Associate Director at SOM San Francisco. Neville is deeply committed to an integrated approach to structural engineering and has worked extensively on large-scale building projects across the US and around the world. Many of his projects have required non-prescriptive design approaches. His recent projects include Al Hamra-Fidrous Tower in Kuwait and Jinta Tower in Tianjin, China, both recently topped out.

"The use of steel plate shear walls found a fortuitous parallel in the history and capabilities of the construction industry in Tianjin: a major port city, and leading center for steel production and ship building in China long accustomed to working with steel plates. This led to the premise of a structure based entirely on the use of thin steel plates."

Faced with a challenge to design a very slender, uniquely-shaped supertall office building in Tianjin, China, the design team selected steel plate shear walls (SPSW) as the most efficient and appropriate lateral load resisting system for the 75-story Jinta Tower after investigating various structural solutions.

Structural Considerations

The 336.9-meter (1,105-foot) tall Jinta Tower, situated at the historical heart of Tianjin, captures the city's powerful confluence of history, culture, geography, and art. The project is intended to create a nuanced public place that embodies the city's international prominence as a physical and economic gateway to China (see Figure 1).

The program includes 205,000 square meters (2.69 million square feet) of first-class office, banking, and restaurant spaces. The 75-story tower has an elliptical plan footprint of approximately 81 x 42 meters (266 x 138 feet) at the base which results in an overall aspect ratio close to 1:8 (see Figure 2). Because of the slender form, a key design challenge was to develop an efficient lateral system capable of resisting significant wind and seismic lateral loads while simultaneously keeping wind-induced lateral drifts and oscillations under acceptable limits.

Figure 1. Tianjin Jinta Tower

Several structural system options were considered in the concept and early schematic design phases, including an all-concrete dual system with perimeter moment resisting frames and core shear walls, composite systems with perimeter steel moment resisting frames, steel floor framing and composite metal deck slabs, and composite concrete and steel plate shear walls, and all-steel systems with perimeter moment resisting frames and braced or SPSW cores. The steel and composite systems utilized circular concrete filled tube (CFT) columns to minimize their dimensions.

The all-concrete system was eliminated early primarily because the large required sizes of the shear walls and columns had a significant impact on rentable area and consequently the project's financial viability. All-steel dual systems with perimeter moment frames and braced cores were found to require as much as 20–25% more steel to satisfy structural performance requirements than systems utilizing SPSWs in the core leading to their elimination from further consideration.

This left dual systems that utilized either all steel or composite SPSWs. Composite shear wall solutions were eliminated after a detailed investigation showed that considering the specific features of the project such as the CFT columns, there was insufficient precedent and research/testing data available to convincingly demonstrate the feasibility of these systems to the authorities without very significant research, testing, and impact on the project cost and schedule. This fact, taken together with the minimal dimensional needs for SPSWs (slender, all steel), the availability of substantial code provisions and design guides, research and testing data that highlighted the superior ductility of SPSWs, and excellent predicted structural performance led to a decision to use SPSWs over braces in the tower core.

This decision could not have found a more fortuitous confluence with the project's location in Tianjin. Tianjin has a long history as the ancient entry port for travelers to the historic capital city of Beijing and a major center for ship building. Surrounding Tianjin is the province of Hebei, which boasts the third largest reserves of iron ore in China. The presence of appropriate material reserves and technology related to the production and fabrication of steel, and steel plates in particular, in and around Tianjin sealed the decision to use SPSWs as the primarily lateral load resisting system of the tower.

Because of the relative newness of the structural system as well as a height that significantly exceeded code limits, the project was subjected to review by panels of seismic and wind experts in accordance with the regulations in China at the end of the design development phase. The experts reviewed the seismic and wind performance of the proposed structure and imposed additional requirements to address the unique nature of the project and ensure its safety.

...competition

"I knew that in order to put me ahead of the competition, I had to do iconic buildings."

Danny Salvatore, Fernbrook President, developer of the Absolute Towers in Mississauga. From "Like Marilyn herself, the Absolute Tower is Smart, Sexy, Built to Impress," theglobeandmail.com, November 26, 2010.

Structural System Description

The main lateral force resisting system for the tower comprises a perimeter ductile moment-resisting frame, and an interior SPSW core linked together with outrigger and belt trusses (see Figure 3). The perimeter ductile moment-resisting frame consists of CFT columns and structural steel wide flange beams. Typical column spacing at the perimeter is approximately 6.5 meters (21.3 feet). The interior shear wall core consists of CFT columns and structural steel wide flange beam ductile moment-resisting frames in-filled with structural steel plates to create SPSWs. Four sets of outrigger trusses and

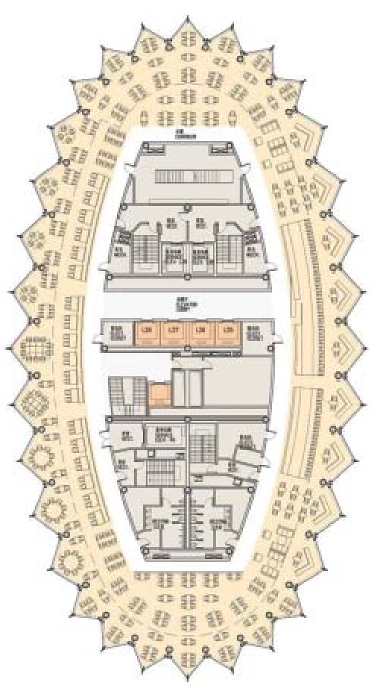

Figure 2. Typical floor plan

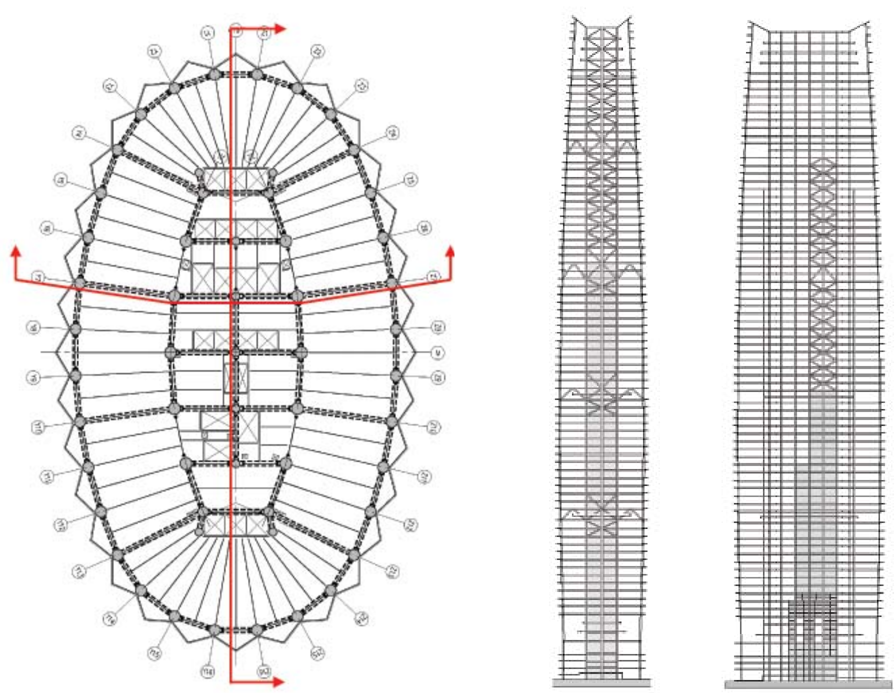

Figure 3. Structural plan at outrigger levels and overall building sections

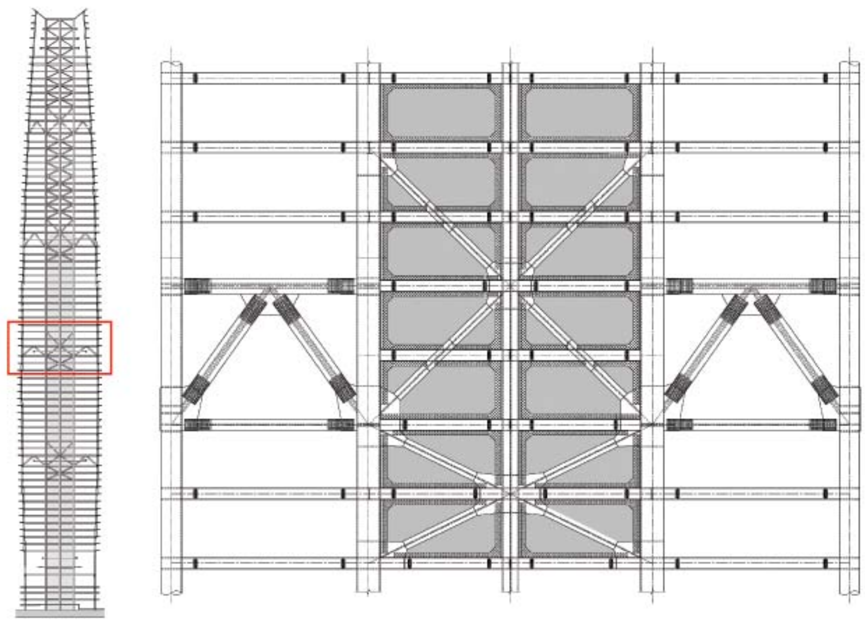

Figure 4. Outrigger truss

belt trusses, located at mechanical floors, have been used to link the core and perimeter frame and stiffen the structure in the transverse (narrow) direction (see Figure 4). As only eight perimeter columns are connected directly to the outrigger trusses, the belt trusses are utilized to link the remaining columns to work with the outrigger trusses and increase their effect. In the process of behavior and material optimization, steel plates were found to be necessary only over two thirds of the building height in the core, transitioning to braces above and eventually to moment frames.

The gravity system for the tower consists of conventional rolled structural steel wide-flange composite framing and typically 120 millimeters (4.7 inches) thick composite metal deck slabs. The CFT columns vary in size over the tower height from a maximum of 1,700 millimeters (66.9 inches) at the building base to 700 millimeters (27.6 inches) in diameter near the top. The perimeter columns bend to follow the shape of the tower exterior. Concrete in the CFT columns is grade C60 (60 MPa cube strength at 28 days) over the entire height of the tower.

The lateral and gravity systems of the superstructure are typically continued down into the substructure. The tower foundation system consists of a four-meter (13.1-foot) conventionally reinforced concrete mat foundation supported by augured cast-in-place caissons. The caissons are typically 800 millimeters (31.5 inches) in diameter and extend down approximately 60 meters (197 feet). All foundation concrete is C40. Strengthened reinforced concrete diaphragm slabs at and below the ground level transfer lateral shear forces to the perimeter reinforced concrete foundation walls and are also provided at outrigger floors.

Loading and Performance

General

The structure was designed to meet the loading and performance requirements in the Chinese code. Code design procedures for buildings in China typically require the use of 50-year return wind and seismic loads (63.5% probability of exceedence in 50 years) in design loading combinations. The seismic event corresponding to this recurrence period is termed as a frequent earthquake.

On account of the project's size and importance, however, the code required that the tower be designed to meet strength requirements under the 100-year wind loads (basic wind pressure 0.6 kN/m^2 in Tianjin). Code wind acceleration perception requirements were based on a 10-year return event (basic wind pressure 0.3 kN/m^2 in Tianjin) with damping set at 1.5% considering the beneficial damping effects offered by the concrete in the CFT columns. The horizontal floor accelerations were limited by code to 0.28 m/s^2 at the highest occupied floor. Wind tunnel testing was required. In accordance with local practice, wind speeds used in the tests were at least as high as those stipulated in the codes in the predominant wind direction, but directional effects were permitted to be considered.

Tianjin is located in Chinese seismic intensity Zone 7 and the 475-year return (moderate earthquake) peak ground acceleration corresponding to this seismic event, per local codes, is 0.15 g (147 cm/s^2). Inter-story drifts are limited to 1:300 in the frequent seismic event, with damping set at 3.5%.

The codes also required that the tower be analyzed dynamically using two measured and one simulated site specific time histories (length of record at least eight times the fundamental period), and required that two different analysis programs be used.

Additional Requirements Imposed

Wind and seismic experts who reviewed the project made several recommendations intended to ensure the safety and sound performance of the tower structure, recognizing that the code was not written for these types of structures. They recommended, among other things, that the design team:

- Satisfy code drift requirements using the code static 100-year wind loads in addition to the 50-year wind tunnel loads. Strength requirements were to be satisfied using the more critical of the code and wind tunnel 100-year loads.
- Perform scaled testing of the typical proposed SPSW assembly.
- Perform non-linear time-history analysis to evaluate the behavior of the structure in the code rare earthquake (2,475-year return). Damping was to be 5%.

- Design columns and outriggers to essentially not yield in a moderate earthquake (475-year return).
- Design the columns in the lower 16 floors (below the level of the lowest outriggers) to essentially not yield in the rare earthquake.

Figure 5 shows a summary of key aspects of the tower's structural behavior: the computed peak accelerations at the highest occupied floor using the code 10-year winds were found to be 0.20 m/s^2 as compared to 0.214 m/s^2 from the wind tunnel studies. These were less than the 0.28 m/s^2 limiting criteria in the code.

Rotational velocities were also checked in the wind tunnel studies and found to be 1.9 mrad/s based on the code 10-year winds. This value is less than the 3-mrad/s criteria recommended by Isyumov (1993). The Chinese code does not currently have any acceptance criteria relating to rotational velocities.

SPSW Design Philosophy and Procedures

The philosophy and procedures for the design of the SPSWs were based on the integration of US and Chinese code requirements. A slender SPSW design approach was adopted which means that the lateral strength and stiffness of the shear walls results from tension-field action in the steel plates. The relevant requirements of the following US codes and references were utilized:

1. AISC-341: Seismic Provisions for Structural Steel Buildings
2. FEMA 450: Recommended Provisions for New Buildings and Other Structures"
3. Steel Design Guide 20: Steel Plate Shear Walls

Chinese codes currently only have a few requirements that pertain to the design of SPSWs. A key feature of the requirements, and one that is not mirrored in the US codes and references, is that the SPSWs not buckle in the code's defined frequent earthquake and winds. The requirements of the JGJ 99-98, Technical Specification for Steel Structure of Tall Buildings, Appendix 4 in the Chinese code were utilized as well.

Comparison of Elastic Response of ETABS Elastic Model and SAP2000 Nonlinear Model

		ETABS	SAP2000	Difference
Period	1st mode transition	7.60	7.56	0.5%
	2nd mode transition	7.08	7.35	-3.8%
	3rd mode in torsion	5.90	6.33	-7.3%
	4th mode transition	2.53	2.67	-5.5%
	5th mode transition	2.19	2.31	-5.5%
	6th mode in torsion	2.14	2.21	-3.3%

	Seismic X	Seismic Y	50-Year Wind X	50-Year Wind Y
Maximum Inter-story Drift Ratio	1 / 372 (L36)	1 / 358 (L52)	1 / 625 (L36)	1 / 401 (L69)
Base Shear (kN)	38,900	37,400	25,830	39,740

Per CECS 159: 2004 Section 4.2.2, the inter-story drift ratio is:

Wind Load: 1/400

Frequent Seismic Load: 1/300

Notes:
1. Building period discount coefficient of 0.9 is used in dynamic spectrum analysis.
2. CECS: Chinese Engineering Construction Standard

Figure 5. Structural periods, base shears and drifts

General Design Philosophy

- The building frames are designed to carry gravity loads while neglecting the contribution of the SPSW plates. This ensures that the building fames have sufficient capacity to support the gravity loads during seismic events when the plates in shear experience buckling along the compression diagonal due to the development of the tension-field action.
- SPSW plates are sized to respond elastically without tension-field action or buckling under frequent earthquake loads and design wind loads as required by Chinese code JGJ 99-98.
- Tension-field action (see Figure 6) is expected to be the primary lateral load-resisting mechanism in the SPSW plates in the event of moderate or rare earthquakes. Compression contribution along the cross diagonal is conservatively neglected.
- The beams (horizontal boundary elements – HBEs) and columns (vertical boundary elements – VBEs) bounding the SPSW plates are designed for the forces determined from elastic analyses to meet the requirements of the Chinese code. The strength design forces include the component forces from the steel plates.
- Plastic hinging (but no failure/significant strength loss) is permitted at the ends of HBEs at moderate earthquake-demand levels as well as at rare earthquake-demand levels.

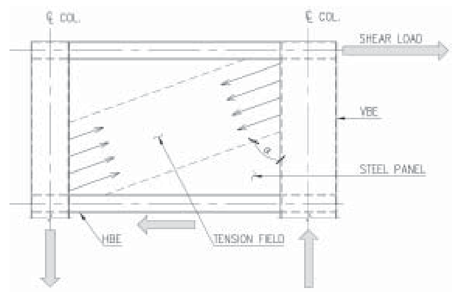

Figure 6. Tension-field action

- As per the requirement of the seismic experts, some minor yielding but no plastic hinging is permitted in the VBE's at moderate earthquake levels, and, in the lower 16 stories, some minor yielding but no plastic hinging is permitted in the VBEs at rare earthquake levels.

SPSW Modeling for Elastic Analysis

In accordance with the design philosophy detailed in the previous section, SPSW panels were designed to not buckle under frequent seismic and wind design loads. Therefore, SPSW panels were modelled using full shell elements and isotropic materials.

SPSW modeling for Preliminary Elasto-plastic Analysis.

Steel panels of the SPSWs are expected to buckle along compressive diagonals when subjected to moderate and rare earthquake loads depending on their slenderness ratios. After buckling, tension-field action of the tension diagonals becomes the primary mechanism to resist shear forces in steel plates.

The SPSW plates were modeled for non-linear analysis during design development using a series of truss members parallel to the tension field with the inclination angle as suggested in Figure 6. As both linear elastic and non-linear time history analyses were performed using SAP NL, the strips were provided in two directions with compression resisting capabilities depending on their effective unbraced lengths (see Figure 7). The stiffness of the strips was adjusted to ensure that the dynamic properties of the non-linear model in the elastic range matched those of the elastic model. The number of strips per panel in each direction was taken to be no less than ten. Each strip was assigned a non-linear uniaxial plastic hinge at its midlength. Construction staging was simulated in the models and compliance with the requirements of the code and the additional requirements of the experts was verified before the design development phase was complete.

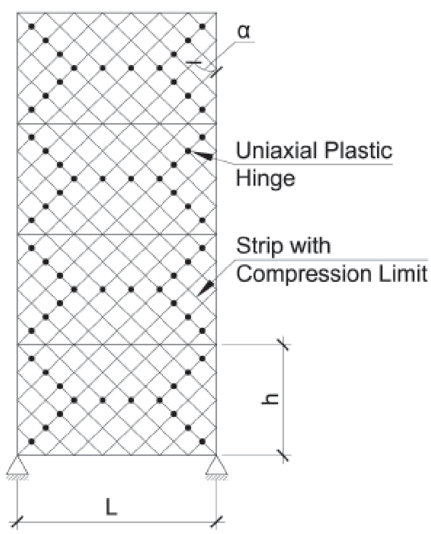

Figure 7. SPSW panel strip model

Construction Considerations

The contractors for the Jinta Tower initially recommended the use of slip-critical bolted connections between the steel panels and the boundary elements. In order to ensure that the SPSW panels did not buckle under the effects of frequent seismic and design wind loads, it was deemed necessary to minimize axial gravity loads transmitted to the steel panels. This was to be accomplished by providing vertically slotted or oversized holes for the bolted connections along the top and side edges of the steel panels. The steel panels were to be installed as the tower construction proceeded, but the bolts only tightened after the tower reached its full height and most of the dead loads had been imposed. The stability of the structure in the temporary condition was provided by the core and perimeter ductile moment-resisting frames.

Further consideration of the schedule implications of this approach, as well as concerns about erection tolerances led to a decision during the construction document phase to eliminate the delayed construction approach and to complete the SPSW connections as the construction proceeded. The panels would need to be designed to meet the Chinese code requirement of "no buckling under frequent lateral loads" by appropriately stiffening them to support the additional gravity stresses they would experience as a result of this approach without significantly impacting their "slender" ductile behavior, stiffness, or steel tonnage.

This was accomplished by revising the panel edge connections to be fillet welded connections and introducing vertical channel stiffeners on both sides of the steel panels designed to enable the panels to sustain the gravity axial loads imposed without buckling when combined with frequent wind and seismic load effects (see Figure 8). In order to prevent the vertical stiffeners from acting like columns, gaps were left between the ends of the stiffeners and the HBEs.

SPSW Stiffener Design and Performance Verification

The size and spacing of the vertical stiffener elements, as well as the gap dimension were determined using non-linear pushover analyses of individually modeled panels with stiff edge conditions. Models were made using the ABAQUS non-linear program for each unique panel proportion and plate thickness condition, and tested under a combination of gravity load and lateral pushover load to determine the optimal stiffener spacing and gap size that would permit the panel to comfortably undergo lateral drift associated with the frequent earthquake or wind without buckling (see Figure 9).

A detailed non-linear analysis model (more than 10 million degrees of freedom and capable of simulating the buckling responses of individual panels) of the entire structure was then made using ABAQUS (Lee et al., 2010), simulating construction staging and subjected to the full spectrum of gravity, wind and seismic loading conditions, including moderate and rare seismic events. Satisfaction of the performance requirements of the

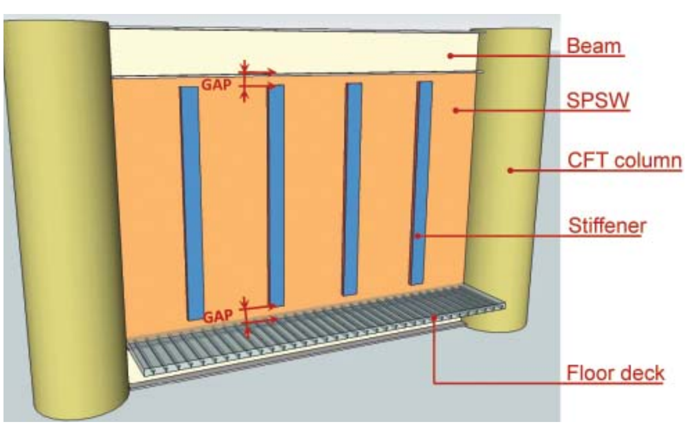

Figure 8. Stiffened SPSWs

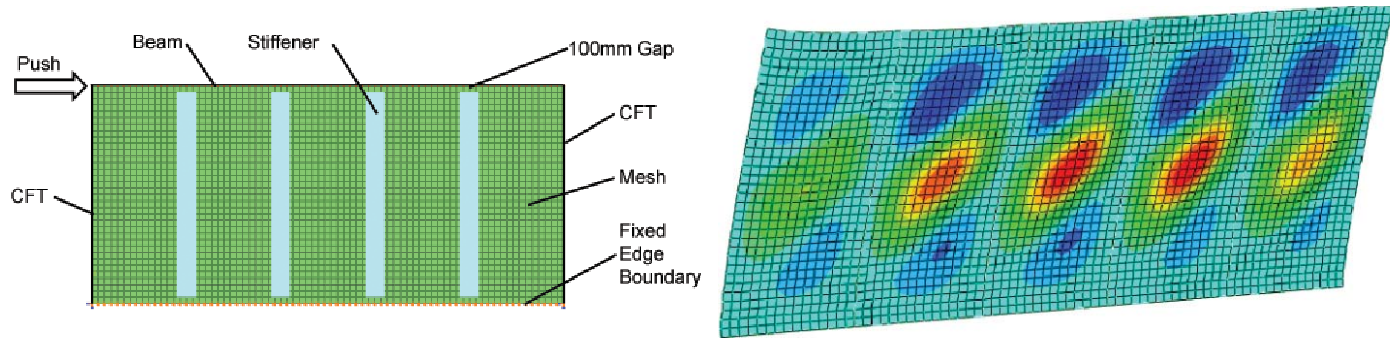

Figure 9. SPSW pushover model and deformed shape

code such as no SPSW plate buckling under load combinations including frequent lateral loads and no collapse or inter-story drifts to exceed 1:50 in rare seismic events, as well as the additional requirements set by the experts was verified.

Testing

Pursuant to the requirements of the seismic experts, a typical SPSW assembly two bays wide and four stories high at 1:5 scale was tested at Tsinghua University in Beijing. Low-cycle testing using the protocols of Chinese code (JGJ101-96) was performed (see Figure 10). The system demonstrated excellent ductility reaching inter-story drift ratios of 1:22 – the limit of the testing apparatus – comfortably meeting the drift requirement of the code, with evidence of ample reserve ductility.

Figure 10. SPSW assembly test set-up

Comparative performance of welded versus bolted steel panel to boundary element connections, and a typical outrigger connection to the core SPSW were also tested at Tsinghua University as required by the experts. All tests yielded results consistent with predictions using detailed non-linear analysis models and met respective requirements demonstrating the validity of the proposed details.

Conclusions

Steel plate shear walls acting in tandem with ductile moment resisting frames were determined to be the optimal and integrated architecture-engineering-construction solution to structuring the slender 336.9-meter (1,105-foot) tall Tianjin Jinta Tower. The structural solution was subjected to rigorous review by panels of seismic and wind experts in China, and this review resulted in the adoption of enhanced analysis and design procedures, performance goals; and testing. The project has been structurally topped-out and is nearing full completion.

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It is not possible to name all the team members, but particular acknowledgement is due to the following key team members: E. Long and Dr. S. Nie (SOM); D.Y. Lu, L. Huang, L. Xu (ECADI) and Y. Liao (RBS+GSCC).

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References

AISC COMMITTEE ON SPECIFICATIONS. 2005. ***AISC-34: Seismic Provisions for Structural Steel Buildings.*** Chicago: American Institute of Steel Construction.

FEMA (Federal Emergency Management Agency). 2003. ***FEMA 450: NEHRP Recommended Provisions for New Buildings and Other Structures.*** Washington D.C.: FEMA.

JGJ 99-98. 1998. ***Technical Specification for Steel Structure of Tall Buildings.*** Beijing: China Architecture & Building Press.

SABELLI, R. & BRUNEAU, M. 2008. ***Steel Design Guide 20: Steel Plate Shear Walls.*** Chicago: American Institute of Steel Construction.

LEE, S., WANG, D., LIAO, Y. & MATHIAS, N. 2010. "Performance Based Design of a 75-Story Buckling Restrained Slender Steel Plate Shear Wall Tower." Proceedings of ASCE / SEI 2010 Structures Congress in Orlando, Florida.

ISYUMOV, N. 1993. "Criteria for Acceptable Wind-induced Motion of Tall Buildings." Proceedings of the CTBUH International Conference on Tall Buildings, Rio de Janeiro, Brazil, May 17–19,1993.