Title: **Steel Plate Shear Walls, Efficient Structural Solution for Slender High-rise in China**

Authors: Mark Sarkisian, Structural Director, Skidmore, Owings & Merrill LLP
Neville John Mathias, Associate Director, Skidmore, Owings & Merrill LLP
Eric Long, Associate, Skidmore, Owings & Merrill LLP
Zhihui Huang, Senior Engineer, Skidmore, Owings & Merrill LLP

Subjects: Building Case Study
Structural Engineering

Keyword: Structure

Publication Date: 2008

Original Publication: Seismic Engineering international Conference 2008

Paper Type:

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 Unpublished conference paper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 Unpublished

Steel Plate Shear Walls: Efficient Structural Solution for Slender High-Rise in China

Neville Mathias, PE, SE [a]; Mark Sarkisian, PE, SE[b]; Eric Long, PE[c] and Zhihui Huang PhD, PE [d]

[a] *Associate Director, Skidmore, Owings & Merrill LLP, One Front Strert, San Francisco, CA 94111*
[b]*Structural Director, Skidmore, Owings & Merrill LLP, One Front Strert, San Francisco, CA 94111*
[c]*Associate, Skidmore, Owings & Merrill LLP, One Front Strert, San Francisco, CA 94111*
[d]*Senior Engineer, Skidmore, Owings & Merrill LLP, One Front Strert, San Francisco, CA 94111*

Abstract. The 329.6 meter tall 74-story Jinta Tower in Tianjin, China, is expected, when complete, to be the tallest building in the world with slender steel plate shear walls used as the primary lateral load resisting system. With an overall aspect ratio close to 1:8, the design challenge for this tall building was to develop an efficient lateral system capable of resisting significant wind and seismic lateral loads, while simultaneously keeping wind induced oscillations under acceptable perception limits. The paper will describe the genesis of the structural system selection considering various available options and significant aspects of the analysis, design, testing and proposed construction. US and Chinese codes and standards, current on-going research and development, and project specific testing were integrated to develop the analysis and design procedures used.

Keywords: Steel plate shear wall; SPSW; tension field action; boundary elements.

INTRODUCTION

The 329.6 meter tall 74-story Jinta Tower (Tower) in Tianjin, China, is expected, when complete, to be the tallest building in the world with slender steel plate shear walls (SPSW) used as the primary lateral load resisting system. The Tower has four stories of parking space below existing grade and 74 stories of office space above grade (Figure 1). It has an elliptical footprint approximately 42m by 81m at the base which changes with height to create an “entasis” effect. The total framed area of the tower is approximately 205,000 sq. m. The building is intended for office use.

Because of the tower’s slender form, it has an overall aspect ratio close to 1:8, 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 oscillations under acceptable perception limits. A wind tunnel analysis was performed for the tower.

Figure 1. Architectural Rendering of Tower

Several structural system options were considered in the concept and early schematic design phases including a concrete dual system with perimeter special moment resisting frames and core shear walls; composite systems with perimeter steel special moment resisting frames, steel floor framing and composite metal deck slabs, and composite concrete and steel plate shear walls; and steel systems with perimeter special moment resisting frames and braced or SPSW cores. The steel and composite systems utilized concrete filled circular tubes (CFT) columns to minimize their dimensions. Concrete systems were eliminated primarily on account of the large sizes of the members required that had a significant impact on rentable area. Two types of composite shearwalls were considered; a double steel plate "hull" system with concrete infill and a more conventional system with a single steel plate embedded in the middle of a concrete wall. Both alternatives were eliminated after detailed investigation showed that there was insufficient precedent and research / testing data available (considering the specific features of the project such as the CFT columns) to convincingly demonstrate the feasibility of these systems to the authorities without very significant research, testing, cost and, most significantly, time. Steel dual systems with braced cores were found to require as much as 20 - 25% more steel to satisfy structural performance requirements than SPSWs. This fact, taken together with the minimal dimensions of the steel plates, 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 resulted in a decision to use SPSWs over braces in the tower core.

Because of the relative newness of the structural system as well as its significant height that exceeded the code, at the end of the Design Development Phase the project was subjected to review by panels of seismic and wind experts in accordance with the regulations in China. 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.

STRUCTURAL SYSTEM DESCRIPTION

Following is a description of the structural system of the tower:

Lateral system

The main lateral force resisting system for the tower consists of a frame-shear wall system, comprised of a perimeter ductile moment-resisting frame, and an interior SPSW core linked together with outrigger and belt trusses (Figure 2).

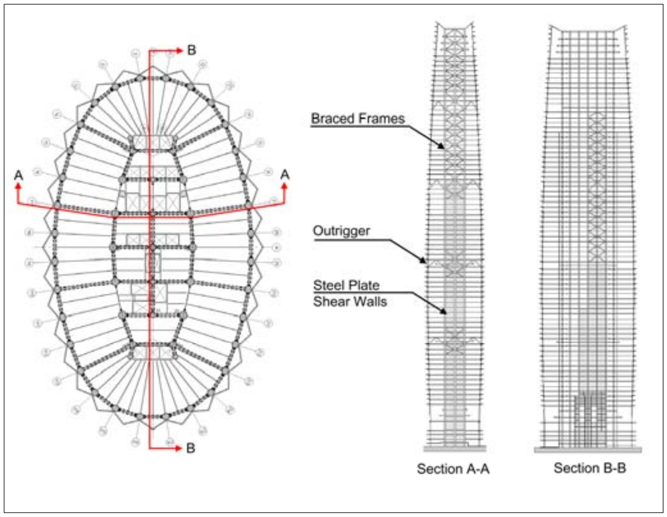

Figure 2. Typical Structural Plan, Sections

The perimeter ductile moment-resisting frame consists of concrete filled steel pipe composite (CFT) columns and structural steel wide flange beams. Typical column spacing at the perimeter is approximately 6.5m. Ductile moment-resisting beams are used at typical exterior column-to-column framing. The interior shear wall core consists of CFT columns and structural steel wide flange beams infilled with structural steel plates to create steel plate shear walls (SPSW). The SPSWs are located in the service area of the tower core, around passenger and service elevators as well as stairways and mechanical rooms. The outrigger trusses, which are placed in the short direction of the tower plan, consist of structural steel bracing connecting the interior shear wall core with the perimeter ductile moment-resisting frame (Figure 3).

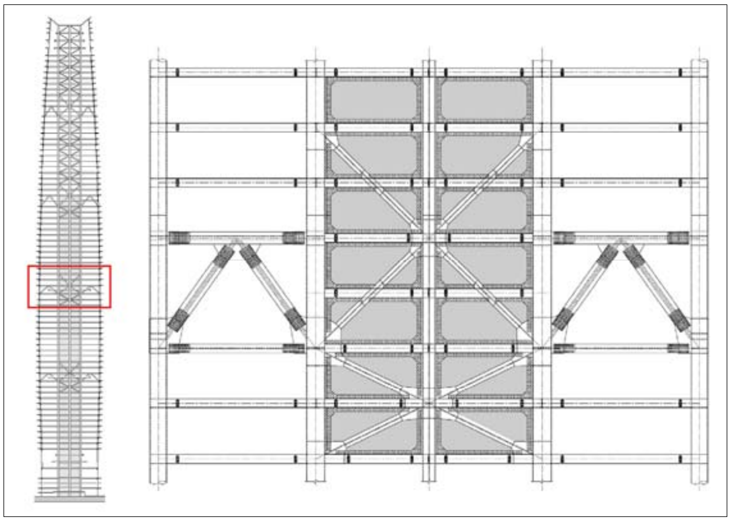

Figure 3. Outrigger Truss

The belt trusses consist of structural steel bracing in the perimeter ductile moment-resisting frame (Figure 4). Four sets of outrigger and belt trusses are provided and located at the mechanical levels of the tower. Strengthened floor diaphragm slabs are used at the outrigger levels. The belt trusses serve to better distribute the axial loads resulting from overturning action among the perimeter columns.

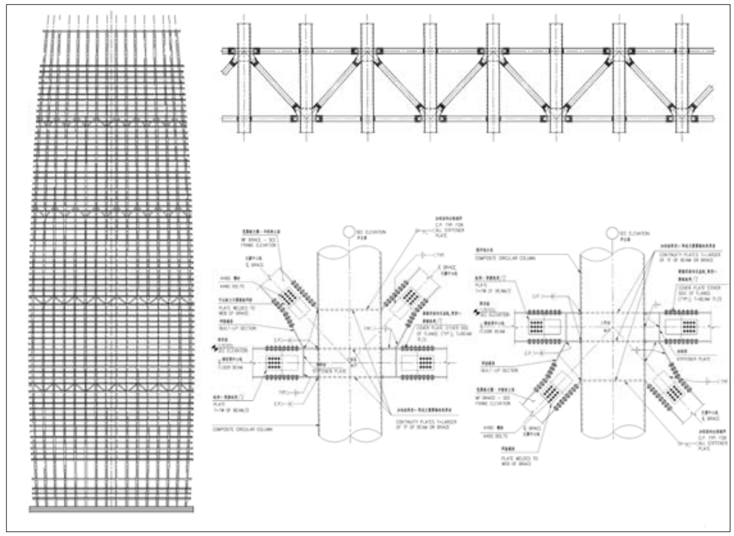

Figure 4: Belt Truss

Gravity system

The gravity system for the tower consists of conventional rolled structural steel wide-flange framing and composite metal deck slabs. Metal decking is typically 65mm deep with 55mm normal weight concrete topping for a total slab depth of 120mm. Composite structural steel wide-flange beams are typically 450mm deep and span from the interior shear wall core to the perimeter ductile moment-resisting frame. Beams are typically spaced at 3.25m on center. CFT columns at the interior shear wall core and perimeter ductile moment-resisting frame are also used to resist gravity loads. The core perimeter CFT columns vary from 1700mm to 700mm in diameter over the tower height and the core interior CFT columns vary from 1150mm to 600mm in diameter. The perimeter moment-frame CFT columns vary from 1700mm to 700mm in diameter over the tower height. Concrete in the CFT columns varies in grade from C80 (80 MPa 28 day cube strength) at he base to C60 at the top of he tower.

Substructure and Foundations

The lateral and gravity system of the superstructure are typically continued down into the substructure.

The tower foundation system consists of a 4000mm conventionally reinforced concrete mat foundation supported by augered cast-in-place caissons. The caissons are typically 800mm in diameter and extend down approximately 60m below the bottom of mat foundation to local soil layer 11. All foundation concrete is C45. The foundation system is overlain by a 400mm thick gravel layer and a 150mm thick reinforced concrete topping slab. Strengthened reinforced concrete diaphragm slabs at and below the ground level transfer lateral shear forces to the perimeter reinforced concrete foundation walls.

LOADING AND PERFORMANCE REQUIREMENTS

General

The design was required to meet the Chinese code loading and performance requirements including those for gravity, wind and seismic loads, as well as those for strength and stiffness. Code design procedures typically utilize 50 year return wind and seismic loads (63.5% probability of exceedance in 50 years). The seismic event corresponding to this recurrence period is termed the "frequent earthquake".

Based on the project's size and importance, the codes required that the tower be designed to meet drift requirements under the 50 year wind (basic wind pressure 0.5 kN/m2 in Tianjin) and strength requirements under the 100 year wind (basic wind pressure 0.6 kN/m2 in Tianjin). Damping was set at 3.5% considering the composite effect of the CFT columns. Inter-story drifts under the 50 year wind load were limited to 1/400 with damping set at 3.5%. Code wind acceleration perception requirements were based on a 10 year return event (basic wind pressure 0.3 kN/m2 in Tianjin) with damping set at 1.5% by code, once again considering the CFT columns. Acceleration was limited by code to 0.28 m/s2 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 seismic intensity zone 7 and the peak ground acceleration corresponding to this seismic event, per local codes, is 0.15g (147cm/sec^2). Inter-story drifts are limited to 1/300 in this 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). The codes also required two different analysis programs be used.

Additional Requirements

Wind and seismic experts who reviewed the project made several recommendations intended to ensure the safety and sound performance of the tower structure considering that it is not typical of the structures the codes were written to address. They recommended, among other things, that the design team:

- Satisfy code drift requirements using the code static 100 year wind loads (instead of 50 year static wind loads) in addition to 50 year wind tunnel loads. Strength requirements were to be satisfied using 100 year loads determined using code procedures and wind tunnel tests.
- Perform scaled testing of the typical proposed SPSW assembly.
- Perform non-linear time-history analysis to evaluate the behaviour of the structure in the code rare (2% in 50 year) earthquake. Two measured and one simulated record were to be used. Damping was to be 5%.
- Design columns and outriggers to typically not yield in a moderate (10% in 50 year) earthquake.

- Design the columns in the lower 16 floors (below the level of the lowest outriggers) to typically not yield in the rare (2% in 50 year) earthquake.

STRUCTURAL BEHAVIOUR SUMMARY

Following, in Figure 5 is a summary of key aspects of the tower's structural behaviour:

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

Wind acceleration performance

The computed peak accelerations at the highest occupied floor using 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 milli-rads/sec based on the code 10 year winds . This value is less than the 3 milli-rads/ sec criteria recommended by the CTUBH/Isyumov. 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 was based on the integration of the 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 following US codes and references were utilized:

- 2005 AISC – 341 “Seismic Provisions for Structural Steel Buildings”[1]
- FEMA 450 “ Recommended Provisions for New Buildings and Other Structures”[2]
- AISC “Steel Design Guide 20: Steel Plate Shear Walls”[3]

Chinese codes currently only have a few requirements that pertain to the design of SPSWs. A key feature of the requirements, and one that s not mirrored in the US codes and references, is that the SPSW’s not buckle in the code frequent earthquake (50 year return). The requirements of the following Chinese codes were utilized:

- JGJ 99-98 Technical Specification for Steel Structure of Tall Buildings, Appendix 4[4].

Based on the documents listed above as well as a review of he Canadian codes and other pertinent papers and references[5]-[8], the design philosophy and procedures outlined in section 5 were developed.

General Design Philosophy

- The building frames are designed to carry gravity loads while neglecting the contribution of the SPSW plates, which ensures that the building fames have sufficient capacity to support the gravity loads during seismic events, when the plates may experience buckling 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 is expected to be the primary lateral load resisting mechanism in the SPSW plates in the event of moderate or rare earthquakes.
- 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.
- 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.

Selection of SPSW plate thickness

The determination of the required thickness of SPSW plates is based on satisfaction of the requirements described below, to not be less than any of the following:

- Thickness required to satisfy frequent earthquake / design code wind drift limits.
- Thickness required to satisfy the following formulas from the Chinese code JGJ 99-98 that assure no buckling under frequent earthquake and design wind loads:

$$\tau \leq f_v \tag{1}$$

$$\tau \leq \tau_{cr} = \left[123 + \frac{93}{(l_1/l_2)^2}\right]\left(\frac{100t_w}{l_2}\right)^2 \tag{2}$$

Where:
τ : actual shear stress in steel plate
f_v : permitted steel plate design shear strength
l_1 : max. dimension for the SPSW plate
l_2 : min. dimension for the SPSW plate
t_w : plate thickness

- Thickness required to satisfy the following equations from FEMA 450 and AISC 341-05 that assure nominal strength (based on tension-field action - See Figure 6) is greater than the code factored load demands. $V_u \leq \phi V_n$

$$V_n = 0.42 F_y t_w L_{cf} \sin 2\alpha \qquad \phi = 0.9 \tag{3}$$

Based on the Chinese building code design philosophy, this can be adapted to:

$$V_n = 0.5 f t_w L_{cf} \sin 2\alpha \qquad \phi = 1 \tag{3a}$$

Where
t_w : thickness of the plate
L_{cf} : clear distance between VBE flanges
F_y : steel tension yield strength
f : permitted steel design tension strength
α : angle of SPSW plate tension-field action yielding as measured relative to the vertical, and is given as :

$$\tan^4 \alpha = \frac{1 + \frac{t_w L}{2A_c}}{1 + t_w h \left(\frac{1}{A_b} + \frac{h^3}{360 I_c L} \right)} \quad (4)$$

Where:
h : distance between HBE centerlines
A_b : cross-sectional area of a HBE
A_c : cross-sectional area of a VBE
I_c : moment of inertia of VBE taken perpendicular to the direction of the SPSW plate plane
L : distance between VBE centerlines

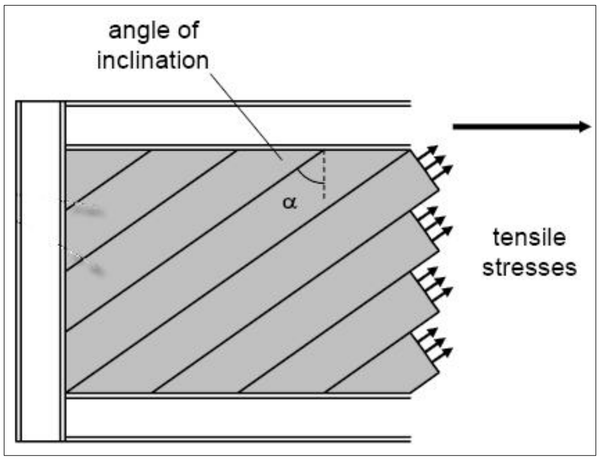

Figure 6. Tension Field Action Angle

Design Criteria

Based on the requirements of the referenced codes, standards and documents, the following criteria were established for the design of the SPSWs:

- SPSW aspect ratio: $0.8 \le L/h \le 2.5$. (AISC 341-05, FEMA 450)
- Maximum SPSW plate slenderness ratio: $25\sqrt{E/F_y} \approx 800(\text{Q235})$. (FEMA 450)
- Minimum SPSW plate slenderness ratio: 200
- SPSW plate connections: the required strength of SPSW plate connections to boundary HBEs and VBEs shall equal the expected yield strength, in tension, of the plate calculated at an angle α. (AISC 341-05, FEMA 450)

- VBEs: the required strength of VBEs shall include the forces corresponding to the steel plates. I_c should not be less than $0.00307 t_w h^4 / L$ to prevent significant "pull-in" of the columns. (AISC 341-05, FEMA 450)
- HBEs: the required strength of HBEs shall be the greater of the forces corresponding to the expected yield strength, in tension, of the plate calculated at an angle α or that determined from load combinations in the applicable building codes assuming the plate provides no support for gravity loads. Similar to VBEs, I_b should not be less than $0.00307 \Delta t_w L^4 / h$. Δt_w is the difference in plate thickness above and below a HBE. HBEs at roof and foundation level shall anchor the pulling action from the yielding plates. (AISC 341-05, FEMA 450)
- HBE-to-VBE connections: shall consider the shear resulting from the expected yield strength, in tension, of the plates yielding at an angle α. (AISC 341-05, FEMA 450)
- Lateral bracing: HBE shall be laterally braced at all intersections with VBE and at a spacing not to exceed $0.086 r_y E / F_y$. r_y is radius of gyration of the HBE about the y-axis (AISC 341-05, FEMA 450)
- Beam-column connections: beam-to-column connections are rigid moment connections (AISC 341-05 and FEMA 450).
- Openings in plates: openings in plates shall be bounded on all sides by HBEs and VBEs extending the full width and height of the panel. (AISC 341-05, FEMA 450).

SPSW MODELLING FOR ELASTIC ANALYSIS

In accordance with the design philosophy detailed in section 5, SPSW panels were designed to not buckle under frequent seismic and design wind loads. Therefore, SPSW panels were modelled using full shell elements and isotropic materials.

SPSW MODELLING FOR ELASTO-PLASTIC ANALYSIS

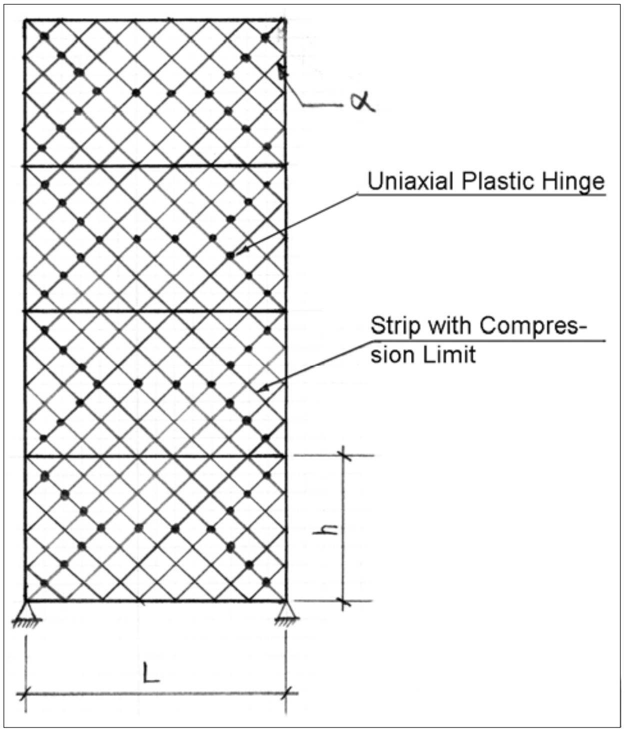

Figure 7. SPSW Panel Strip Model

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 are replaced with a series of truss members parallel to the tension field with the inclination angle α given by equation (4) above as suggested in AISC 341-05 (Figure 7). As non-linear time history analyses are to be performed, strips are provided in two directions with compression resisting capabilities depending on their effective un-braced lengths. The stiffness of the strips are adjusted to ensure that he dynamic properties of the non-linear model in the elastic range match those of the elastic model. The number of strips per panel in each direction shall be taken greater than or equal to 10. Each strip is assigned a non-linear uni-axial plastic hinge. (AISC 341-05, FEMA 450, CAN/CSA S16-01).

SPSW STEEL PLATE BUCKLING CHECK

As has been pointed out, a key requirement of the Chinese codes is that the SPSW panels not buckle under frequent seismic and design wind loads.

A buckling check of SPSW steel panels under frequent seismic and design wind loads in addition to that recommended in the Chinese codes and described in Section 5.2.2 is performed based on “Theory of Plates and Shells” by S. Timoshenko and S. Woinowsky-Krieger. SPSW panels can be regarded as fixed along their vertical edges on account of the very large stiffness of the VBEs when compared to the relatively flexible plates. Simply supported conditions along the vertical edges were, however, used conservatively in the checks. For plates simply supported along four edges while under combined bending and axial stresses at the ends, along with shear, an approximate evaluation of the critical combined loads is obtained by use of a three-part interaction formula (Gerard and Becker, 1957/1958) as follows:

$$\frac{\sigma_c}{\sigma_c^*} + \left(\frac{\sigma_{cb}}{\sigma_{cb}^*}\right)^2 + \left(\frac{\tau_c}{\tau_c^*}\right)^2 = 1 \tag{5}$$

$\sigma_c^*, \sigma_{cb}^*, \tau_c^*$ denote critical stress, respectively, under compression, bending or shear alone, where:

$$\sigma_c^*, \sigma_{cb}^* \text{ or } \tau_c^* = k \frac{\pi^2 E}{12(1-v^2)(b/t)^2} \tag{6}$$

Where:

k : appropriate critical buckling coefficient

b: clear steel panel height.

CONSTRUCTION CONSIDERATIONS

On the Jinta project, the contractors recommended the use of slip-critical bolted connections between the steel panels and the boundary elements (Figure 8). 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 is accomplished by providing vertically slotted or oversized holes for the bolted connections along the top and sides of the steel plates. The steel panels are installed as the tower construction proceeds, but the bolts are only tightened after the tower has reached its full height and most of the dead loads have been imposed.. The stability of the structure in the temporary condition is provided by the core and perimeter ductile moment resisting frames. The buckling checks described in section 8 above consider the gravity loads applied after the bolts are tightened.

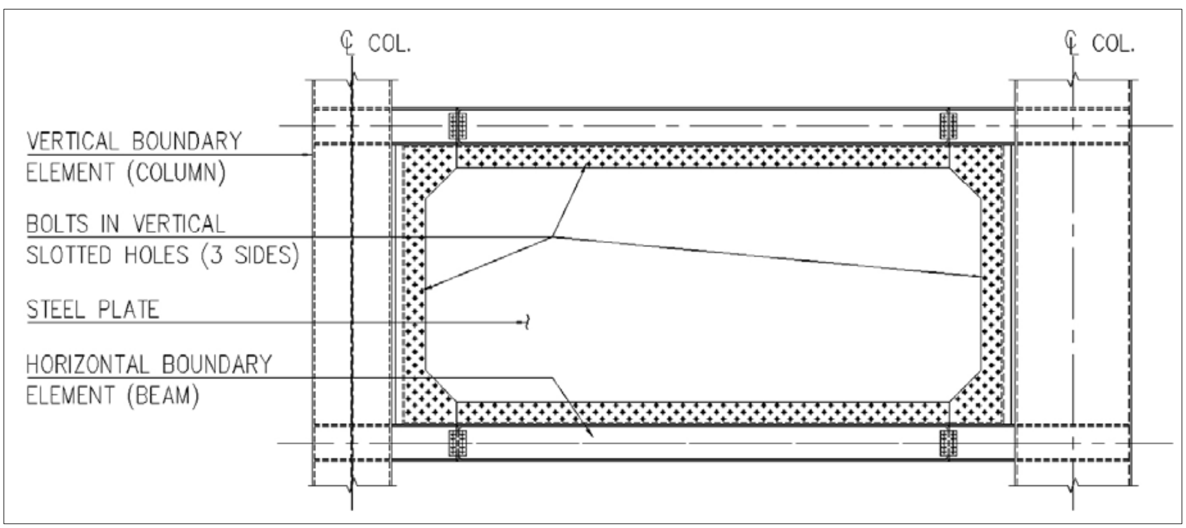

Figure 8. Typical SPSW Elevation

An elasto-plastic model was developed using the program SAP-Non Linear. The elasto-plastic model analysis demonstrated satisfaction of the seismic expert's performance requirements which are much more stringent than the collapse prevention goals typical for such buildings in rare earthquakes. The maximum inter-story drift was 1/76, a value less than the code limit of 1/50.

MOMENT CONNECTIONS

Drift requirements in China are typically limited to 2% in rare earthquakes. The ductility demands on beam-column joints of moment frames in China are not as high as those on special moment resisting frames in the US. Moment connection details commonly in use in China have been developed considering these code ductility demands. Chinese moment connection detailing practices are hence used on the tower structure.

TESTING OF TYPICAL SPSW ASSEMBLY

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. Photographs of the test set-up and a typical panel at the end of the test are provided in Figures 9 and 10, and graphs of story shear versus inter-story drift are shown in Figure 11. Tension field action was clearly evident and the system demonstrated excellent ductility reaching inter-story drift ratios of 1/22, the limit of the testing apparatus, with evidence of ample reserve ductility.

Figure 9. SPSW Assembly Test Set Up

Figure 10. SPSW Panel after Test

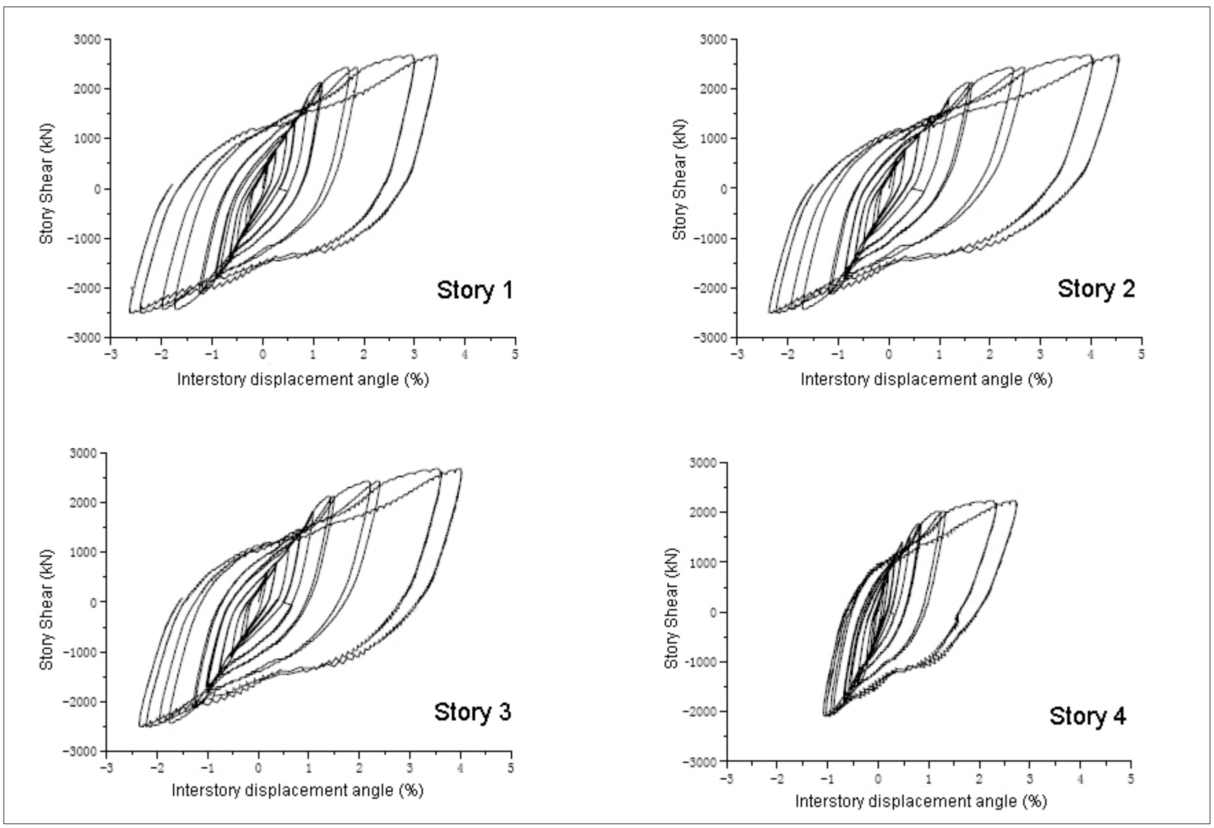

Figure 11. Story Shear vs Inter-Story Drift

CONCLUSIONS AND ACKNOWLEDGEMENTS

Steel plate shear walls acting in tandem with ductile moment resisting frames were determined to be the optimal solution to structuring the 329.6m tall Tianjin Jinta Tower with an aspect ratio of 1:8. The design solution was subjected to rigorous review by panels of seismic and wind experts in China at the end of the “Design Development” phase. This review resulted in the adoption of enhanced analysis and design procedures and performance goals; and testing. The project is nearing completion of the “construction documents” phase. Excavation of the site is complete and construction of the foundations is under way.

The authors wish to acknowledge the efforts of the Skidmore, Owings and Merrill LLP, San Francisco (SOM) design team, particularly those of Shihua Nie PhD, PE, who was responsible for performing the analysis and design computations for the lateral system of the tower. Design responsibilities for the project were shared by SOM and the East China Architectural Design Institute (ECADI) of Shanghai, China with SOM in the primary role through the design development phase and ECADI in the primary role thereafter. Thanks are given to Prof. A. Astaneh-Asl of the University of California, Berkeley and Prof. Y. J. Shi of Tsinghua University, Beijing, China for their invaluable advice during the various design phases of the project.

Finally, and most importantly, thanks are due to the client, Finance Street Holdings of China together with their local expert consultants Prof. Baisheng Rong, Prof. D. X. Wang and Dr. W. B. Yang for their vision in supporting this innovative structural solution.

REFERENCES

1. AISC-341, *Seismic Provisions for Structural Steel Buildings*, 2005.
2. NEHRP *Recommended Provisions for New Buildings and Other Structures*, FEMA 450, 2003 .
3. R. Sabelli , M. Bruneau, AISC *Steel Design Guide 20: Steel Plate Shear Walls*, 2007
4. JGJ 99-98, *Technical Specification for Steel Structure of Tall Buildings*, 1998.
5. ASTANEH-ASL A., *Seismic Behavior and Design of Steel Plate Shear Walls*, 2nd edition, 2006
6. CAN/CSA S16-01, *Limit States Design of Steel Structures*, 2001.
7. ATC 40, *Seismic Evaluation and Retrofit of Concrete Buildings*, Vol. 1, 1996.
8. FEMA 356, *Prestandard and Commentary for the Seismic Rehabilitation of Buildings*, 2000