# Performance Based Seismic Design of a 75 Story Buckling Restrained Slender Steel Plate Shear Wall Tower

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**ABSTRACT**
The Jinta Tower is a 75-story building located in Tianjin, China, with slender steel plate shear walls (SPSW) used as the primary lateral load resisting system. Construction detail, method and schedule constraints result in the steel plates being subject to gravity loads. Chinese codes require that steel plates not buckle in frequent (50 year) seismic events in addition to the typical performance requirements in the moderate (475 year) and rare (2000 year) seismic events. To address these constraints, a buckling restrained slender steel plate shear wall system - SPSWs with vertical stiffeners to enhance gravity load carrying capacity of the plates and delay buckling is developed. The approach for stiffener design included use of buckling and interaction formulae as well as pushover, parametric and sensitivity studies. The rare earthquake performance was studied and prescribed performances were verified by nonlinear dynamic analyses. The SPSWs were fine meshed to a 0.1m × 0.1m grid incorporating geometric and material nonlinearities. Geometric imperfections were also taken into account. Other structural members such as concrete filled tube columns, floor slabs, steel beams/braces and stiffeners were also meshed to a similar size to accurately model their nonlinearities. A ten million degree of freedom nonlinear model of the structure was created for three 40s earthquake time history analyses on the ABAQUS parallel platform. Despite the high initial gravity stresses, the stiffeners were demonstrated to push the inception of plate buckling out beyond the frequent events and provide great ductility and ability to absorb the earthquake input energy through significant tension field action in the larger events.

**KEYWORDS**
Buckling Restrained Slender Steel Plate Shear Walls; ABAQUS; Performance Based Design; Tension Field Action.

## INTRODUCTION

The Jinta Tower (Tower) is a 330 meter tall 75-story office building in Tianjin, China, with steel plate shear walls (SPSW) used as the primary lateral load resisting system (Wang 2008 and Mathias 2008)**.** It has an elliptical footprint approximately 42m by 81m at the base which changes with height to create an "entasis" effect, as shown in Fig. 1.

**Fig. 1**. The architectural impression of Jinta

Given the tower's slender form, the SPSW structural system was selected because: (1) a SPSW system's elastic stiffness is much higher than a brace structure's for the same steel tonnage, equivalent to that of a concrete shear wall; (2) research and testing data for SPSW's demonstrate very significant ductility under moderate and rare earthquakes resulting from tension field action.

SPSWs have been used as the primary lateral load resisting system in many modern and important structures (Astaneh-Asl 2006). Apart from their stiffness and ductility benefits, they are much lighter compared to reinforced concrete shear walls, resulting in less weight to be carried by the foundations as well as lower seismic loads due to the reduced seismic mass.

SPSWs can be either stiffened or un-stiffened. Under moderate and rare earthquake events, stiffened SPSWs yield in shear prior to elastic buckling while un-stiffened SPSWs buckle in diagonal tension field action before they yield and thus resist horizontal loads. Due to their great ductility, post-buckling strength, and material efficiency, many un-stiffened SPSWs structures have been designed and constructed in the United States and Canada. Researchers and codes in these countries typically recommend using un-stiffened SPSWs over stiffened SPSWs (Astaneh-Asl 2006, CAN/CSA S16-01 2001).

The design of un-stiffened SPSWs requires that no gravity load be considered to be carried by un-stiffened SPSW panels. It is practical in most low and medium rise buildings for un-stiffened SPSW panels to be installed after the whole structural frame is built. But for super high-rise buildings such as Jinta, the construction schedule would not permit for SPSW panels to be installed after the frame is topped out. After exploration of the construction schedule in detail, the latest that the installation of SPSW panels could begin was determined to be when the 15th story of the frame finished construction. As a result, the SPSW panels would have to carry substantial amounts of gravity load as the rest of the tower was constructed. Elastic buckling checks showed that the un-stiffened SPSW panels would buckle under gravity and frequent earthquake load combinations. This would violate the requirement of the Chinese code JGJ 99-98 that SPSWs not buckle in 50 year return events.

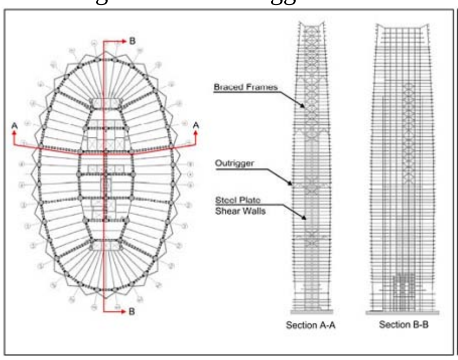
A modified slender SPSW structural system - SPSWs with vertical stiffened channels was hence developed to address this challenge. The use of the vertical stiffeners increased the SPSW panel's elastic buckling capacity so that they would not buckle under gravity load and 50 year return events, while still retaining significant ductility as a result of tension field effects in moderate and rare earthquake events.

Lacking specific code and research references for the proposed buckling restrained slender SPSW system, initial design of the SPSW panels and stiffeners was based on theory of mechanics fundamentals. The preliminary design was then subject to nonlinear analysis, and the analysis results were used to check if the desired performance objectives were achieved. Optimal design was obtained by this trial and error method. In this paper, emphasis is placed on the nonlinear analysis of the structure under earthquake actions and the use of its results to verify desired performance. As a result, the goals of prevention of buckling in frequent seismic and wind events, and tension field action to dissipate the earthquake energy in moderate and rare seismic events—even in the presence of high gravity stresses—are verified.

The paper is organized as follows: In the next section, the lateral structural system and performance objectives are described in detail. As nonlinear dynamic earthquake analysis involves a tremendous engineering effort, understanding the structure's behavior in advance of embarking on this effort is essential to its success. In the following section, the nonlinear models for the structural system are presented. The material properties and the beam and shell elements are discussed. In the following section, use of a simplified nonlinear push-over analysis approach as a preliminary design tool to check that the stiffened SPSWs do not buckle in frequent seismic events is described. Next, the nonlinear dynamic seismic analysis methodology used for verification of structural performance of the full building in rare earthquakes is described. Multiple nonlinear analysis steps are used to simulate the construction sequence. Geometric imperfections are applied to the SPSWs as an initial state to run the nonlinear dynamic earthquake analyses. The major analysis results are then presented and discussed. Tension field action is confirmed to occur in many of the SPSWs, dissipating energy. Finally, conclusions are drawn based on the findings.

## STRUCTURAL LATERAL SYSTEM DESIGN AND PERFORMANCE OBJECTIVES

The lateral force resisting system for the tower can be classified as a frame-shear wall system, with perimeter and core ductile moment-resisting frames, and core SPSWs linked together with outrigger and belt trusses (Fig. 2).



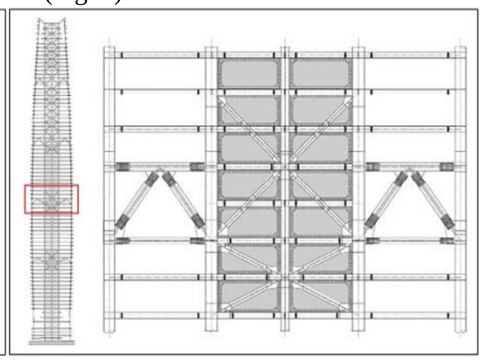



**Fig. 2**. Typical plan and section

**Fig.3**. Outrigger truss

The ductile moment-resisting frames consist of concrete filled steel pipe composite (CFT) columns and structural steel wide flange beams. The SPSWs consist of the CFT columns—Vertical Boundary Elements (VBE)—and structural steel wide flange beams—Horizontal Boundary Elements (HBE)— in-filled with stiffened structural steel plates. Outrigger trusses, which are placed in the short direction of the tower plan, are used to engage the perimeter columns in resisting overturning (Fig. 3). Four sets of outrigger and belt trusses are provided and located at the mechanical levels of the tower. Strengthened diaphragm slabs are used at the outrigger levels.

Buckling restrained SPSWs are used in the middle and lower portions of the building and are located in service core areas around the elevators, staircases and mechanical rooms. At the upper levels, where demands drop off sufficiently, concentric braces are used in lieu of SPSW panels in the core along with the CFT columns and wide flange girders.

Fig.4 shows a buckling restrained SPSW under construction. Gaps were introduced between the HBEs and the ends of the vertical stiffeners for the following reasons:
(1) For ease of fabrication.
(2) To minimize vertical gravity loads and thus prevent local buckling of the vertical stiffeners.
(3) To enable the formation of tension field effects when the structure experiences moderate and severe earthquakes.

The structural system is initially designed using elastic design (frequent seismic events) methods in the Chinese code and AISC 341-05. The maximum story damage drift performance limit is set to 1/600 which is within the limit prescribed by the codes. The elastic analysis methods do not, however, reliably predict whether the stiffened SPSWs remain un-buckled as required by the Chinese code under this service level of drift. Sophisticated nonlinear dynamic earthquake analysis is necessary to demonstrate satisfaction of this performance goal.

**Fig. 4**. Buckling Restrained SPSWs

Because a single nonlinear response history dynamic earthquake analysis run takes more than a day to finish, a trial and error study of the entire structure to design the stiffeners is not practical. Instead, a simplified nonlinear push over analysis methodology is implemented to establish the stiffening requirements. The most critical (highly stressed under gravity loads) one story bay for a given plate thickness is extracted from the structure to perform a nonlinear push over analysis with extreme boundary conditions. If the stiffened gravity loaded SPSWs buckle at story drift at or less than 1/600, then the sizes of the stiffeners are increased until the no buckling occurs. In this way the optimal design of stiffeners is obtained.

Once the initial design is complete, nonlinear dynamic response history earthquake analyses are undertaken to test if the performance objectives for a rare (2000 year return) earthquake are achieved. The performance objectives of this project were as follows:

（1）Overall building performance objectives

- The entire elasto-plastic time-history analysis is able to finish without divergence.
- The building’s is still standing (no collapse) at the end of the analysis.
- The building’s maximum inter-story drift angle is less than the code’s limit of 1/50

（2）Member performance objectives

- SPSWs: Allow steel shear wall plates to yield and buckle out of plane, but limit the value of the maximum plastic strain to 0.025.
- Concrete filled steel tube (CFT) columns: The CFT columns below the first outrigger floor (L15) must not show signs of plastic strain in a code stipulated simulated rare ground motion. If the effects of the measured ground motions exceed the code’s energy requirements, the CFT columns are allowed to have minor plastic strain.
- Outrigger trusses: Maximum plastic strain in members shall not exceed 0.025.

According the Chinese seismic code (GB50011-2001), earthquake ground motion time histories used for rare nonlinear dynamic earthquake analysis must meet the following requirements:

- The response spectrum of the ground motion should match the code response spectrum at the building's fundamental periods (maximum discrepancy 20%).
- The peak ground acceleration value of the ground motion should match the code prescribed value.
- The duration of the ground motions must be at least 5 times the building's first period.
- Two sets of measured and one set of simulated ground motions must be used in the analysis.

For this project, measured ground motions 1 (GTHX1+GTHY1), 2 (GTHX2+GTHY2) and simulated ground motion (GRHX+GRHY) provided by the Building Research Institute were used to conduct the non-linear analysis. The ground motion duration is required to be at least 40s and the accelerated peak value 310Gal. The analysis used bi-directional wave inputs. For each set of ground motions, the main directional waves are GTHX1, GTHX2, and GRHX. The secondary ground motions are GTHY1, GTHY2, and GRHY. The main to secondary directional ground motion acceleration peak values have the ratio of 1:0.85. Fig. 5 shows one of the ground motion records and its spectrum compared to the code specified response spectrum. The envelope of the three groups of results is used to evaluate the performance of the building.

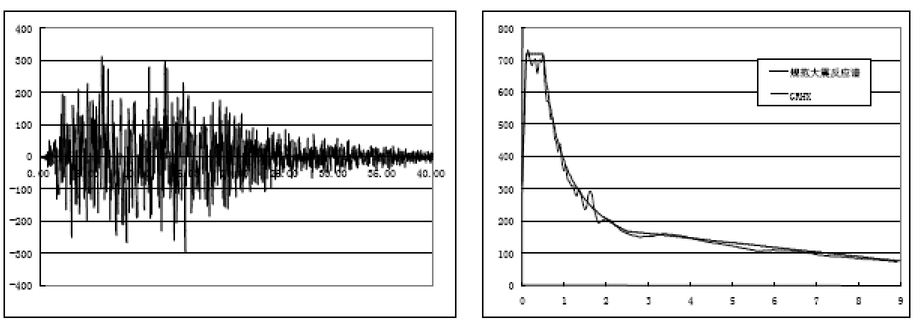



**Fig. 5**. Earthquake record and its spectrum

## NONLINEAR MODELING

A number of nonlinear modeling methods for building structures have been proposed, from simple lumped mass stick models (single equivalent column) to complex models of the entire structure that get down to the material constitutive relationship level. The simplified models provide fair results in nonlinear static push over analyses, but nonlinear dynamic response history analysis shows that there is a significant discrepancy between the results from simplified versus complex 3-D constitutive models (Krawinkler 2006). In the Jinta project, post-buckling behavior of the SPSWs is of primary interest; therefore a complex model capable of simulating the structure down to the material stress-strain level was essential (Roe, 2002).

The basic materials used in the tower are steel and concrete. An isotropic kinematic hardening model is used for the steel material. The Bauschinger effect has been taken into account, and no stiffness degradation occurs in the cycling (ABAQUS 2005). A plastic-damage model (Lee J, 1998) is used for the concrete material. It assumes that the two main failure mechanisms are tensile cracking and compressive crushing of the concrete material. It captures the three major characteristics of concrete used in buildings (Lubiner 1989): (1) strength in compression is larger than that in tension; (2) stiffness degrades when it goes into the plastic range; (3) stiffness is recovered when reversal occurs from tension to compression.

All structural components are modeled with either line elements or shell element in ABAQUS. Line elements are used to model beams and columns, while shell elements are used to model the steel or reinforced concrete shear walls and slabs. For line elements, plane section strains are assumed. As a plastic zone model is adopted, the stiffness of a line element is dynamically obtained by integration in the sectional and longitudinal directions. The hysteric behaviors of the members are represented by the cyclic features of the materials (Lee S. 2008). A general-purpose, three-dimensional, first-order shell element that uses reduced integration with plastic-damage concrete material and rebar layers is used to model concrete shear walls and slabs, while the same shell element with steel material is used to model the steel plate shear walls.

**Comment [MSOffice1]:** If only one layer used, replace with "a rebar layer"

Each node of the shell element has six degrees of freedom that are easy to connect to the line elements. To accurately model the shear walls and slabs, the shell elements are meshed to a 0.1m by 0.1m size. The distributed rebar layer is also taken into account in the reinforced concrete slabs.

## NONLINEAR PUSH OVER ANALYSIS FOR STIFFENER SIZING

A critical one story one bay SPSW model is shown in Fig. 6. The width $L_0$ is 7.4m, the height $H_0$ is 3.4m, the thickness of the steel plate is 32mm. The CFT size is φ1700x65t, the story beam section is H800x400x25x35, and the vertical stiffener channel section is

300x300x28x28. The gaps between the ends of the vertical stiffeners and the HBEs are 100mm. Boundary conditions for the nonlinear analysis are as shown in Fig. 6.

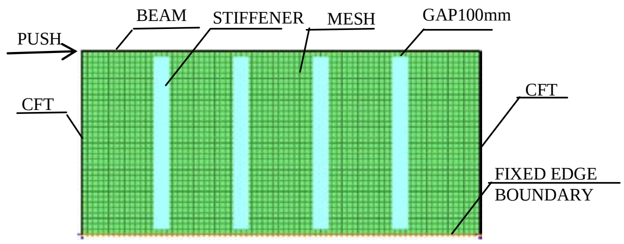



Fig. 6 Push over model

The pre-load for each CFT is 121,000kN and the pre-stress in the steel plate is 110Mpa. Geometric out of plane plate imperfections are taken to be 0.005m. The top edge is then monotonically displaced by a total of 0.01m (1/340 of the $H_0$).

The deformed shape of the SPSW is shown in Fig.7. The maximum out of plane displacement of the plates at a story drift of 1/600 of the $H_0$ is 5.6mm (5mm initial deformation plus 0.6mm). This shows that the stiffened SPSW is in a stable state under a frequent (50 year) event. Fig. 8 shows that when the gap at stiffener ends is increased to 400mm, the plates buckle (the out of plane deflections increase abruptly) under frequent events.

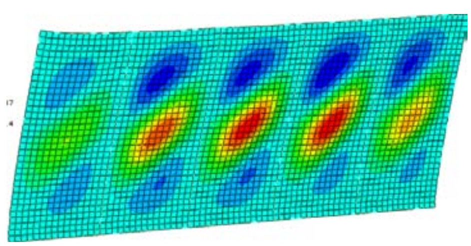


Fig. 7 Deformed SPSW shape

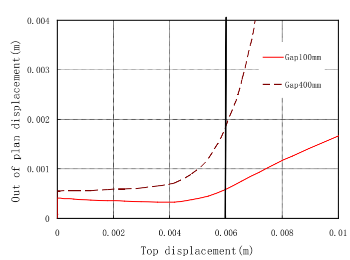



Fig. 8 Out of plane displacement

Using this type of trial and error analysis, the optimum sizes, spacing and end gap size of the stiffeners for different SPSW conditions are obtained.

Fig. 9 shows that when the steel plate pre-stress is 32Mpa, the maximum out-of-plane displacement is substantially smaller than for 110 Mpa. This

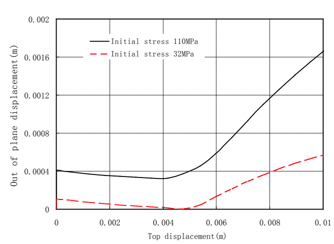



Fig. 9 Out of plane displacement

confirms the importance of minimizing axial stesses due to gravity on unstiffened SPSWs.

## NONLINEAR DYNAMIC EARTHQUAKE ANALYSIS

Before the nonlinear response history dynamic earthquake analysis is run, the following issues need to be noted:

(1) From the results of the elastic response spectrum analysis for the frequent seismic event, it is seen that the steel plates have already attracted more than 80 percent of the shear they attract in the rare event. By gradually scaling up from the frequent to the rare event, one sees that the steel plates buckle and become inelastic first, dissipating energy earlier than other structural members.

(2) After SPSW plate buckling and consequent tension field action occurs, the lateral stiffness of the structure gets reduced and earthquake forces induced also get reduced. Any gravity load carried by the SPSWs transfers upon plate buckling to the adjacent CFTs. The CFTs in the core must thus be capable of supporting the all gravity loads of the structure without the help of steel plates.

**Comment [MSOffice2]:** Sam: Considering the shear weight ratio in the rare EQ is around 4.5 and frequent earthquake around 1.5, this means that a lot of shear is taken by the frame - I assume this includes the CFT VBEs. I have thus changed SPSW to steel plate.

A model of the structure is first built using the ETABS program and after the seismic mass and dynamic characteristics are ascertained, a matched ABAQUS model is created.

In the ABAQUS platform, the follow features are incorporated:
(1) Initial geometric imperfections of the steel plates are introduced. As shown in Fig. 10, a 1.5% (approximately 5mm) out-of-plane displacement is applied to each steel plate segment between the stiffeners channels. The initial stress in the structure is set to zero.

**Comment [MSOffice3]:** Boxes or channels?

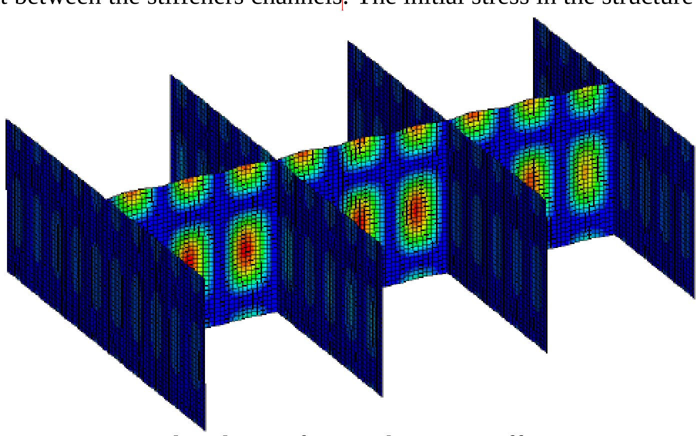


Fig. 10 Introduced imperfections between stiffeners at SPSWs

(2) The construction sequence is then simulated. Buildings are typically built one story at a time. The analysis should simulate this procedure using sub-steps; each sub-step representing a construction step. An element birth and kill technique is utilized to simulate this process as shown in Fig. 11.

This tower is built in the following manner to minimize vertical stresses induced in the steel plates and speed-up construction: (a) Floor-by-floor construction of frames below level 15. (b) Floor-by-floor installation of steel plates and diagonal bracing below level15 while simultaneously constructing the frame for the floors above level 15. (c) Finish loads for floors below level 15 (superimposed dead loads) added, and so on and so forth up the building. (d) Live loads added to each floor. It should be pointed out that this simulation is a highly nonlinear procedure and that the stress states differ from those of the linear gravity analysis using ETABS.

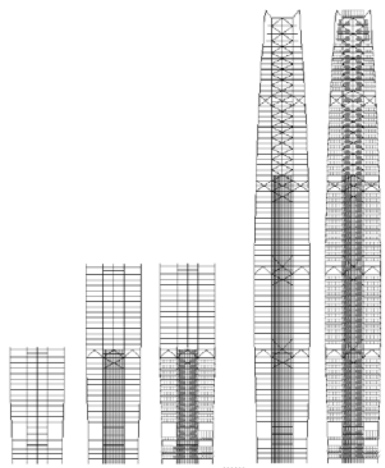


Fig.11 Simulation of building construction

Using the service stress state obtained in the construction steps described as the initial condition, ground motions in two horizontal directions (X and Y) are applied to the structure. Because of the nonlinearity of the structure, mode-based dynamic analysis is not suitable and direct integration dynamic analysis is performed. Implicit and explicit direct integration methods are available. Due to the large scale of the problem (10million DOFs) and the buckling behavior of the SPSWs, unconditional implicit integration typically used in elastic analysis cannot easily solve the equations. Instead, a conditional stable explicit integration method is used. The step length is less than the period of the minimum element which is approximately $2\times10^{-5}$s.

Three sets of earthquake ground motion analyses are run smoothly and without divergence to completion. The building satisfies the "no-collapse" requirement at the end of the rare earthquake simulations. Overall results are presented in Table 1.

The first few periods of the ABAQUS analysis are slightly longer than those from ETABS due to the finer meshing of the SPSWs in ABAQUS. One can observe that the base shear/weight ratios are 4.5 and 5 times those for the frequent earthquake in the X and Y directions respectively, which are smaller than 6 - the ratio of rare to frequent earthquake peak ground acceleration stipulated in the code. This confirms that the structure undergoes inelastic action with a consequent reduction of forces compared to a fully elastic response. The ratios would have been lower and energy dissipation greater if more inelastic action were permitted by the performance goals in the CFT columns.

Table 1 Overall Analysis Results

| Bi-directional earthquake waves | Simulated Motion 1 (GRHX+GRHY) | Measured Motion 1 (GRHX1+GRHY1) | Measured Motion 2 (GRHX2+GRHY2) |
|---|---|---|---|
| The first three period (s) from ETABS analysis | 7.55(X direction), 6.89(Y direction), 5.77(torsion) | | |
| The first three period (s) from ABAQUS analysis | 8.1(X direction), 7.61(Y direction), 6.67(torsion) | | |
| X-direction maximum base shear/weight ratio (1.3% for frequent EQ) | 5.83% | 5.24% | 5.66% |
| Y-direction maximum base shear/weight ratio (1.3% for frequent EQ) | 6.61% | 6.50% | 6.47% |
| Max. roof drift (m)-X | 1.749 | 1.883 | 1.573 |
| Max. roof drift (m)-Y | 2.337 | 2.578 | 1.756 |
| Max. inter-story drift angle-X (floor #) | 1/117 (42) | 1/83(72) | 1/107(56) |
| Max. inter-story drift angle-Y (floor #) | 1/110(27) | 1/100(35) | 1/83(71) |

The maximum inter-story drift computed is 1/83 at Level71 under measured ground motion 2. It is less than the 1/50 limit prescribe by the Chinese code.

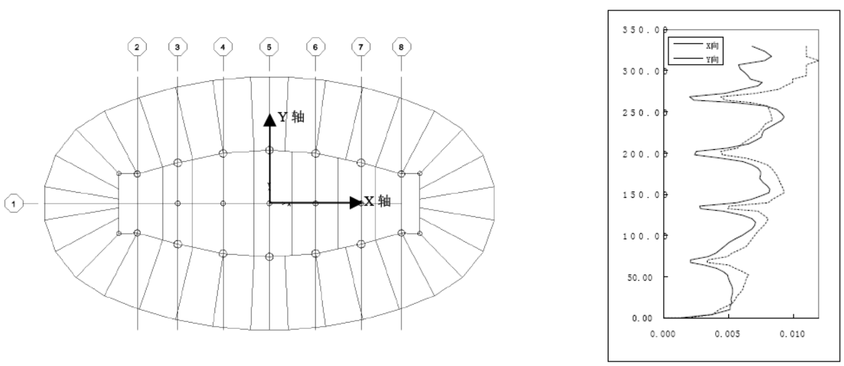



Fig.12 Story drifts along the height of building

The inter-story drifts along the height of the buildings are shown in Fig. 12.

Member performance evaluations:

(1) All the steel plastic strains of floor beams, CFTs and outrigger truss members are less than 0.025.
(2) Fig. 13 shows an example of plastic buckling and tension field effects. Seismic energy in a large earthquake is efficiently dissipated by the SPSWs.

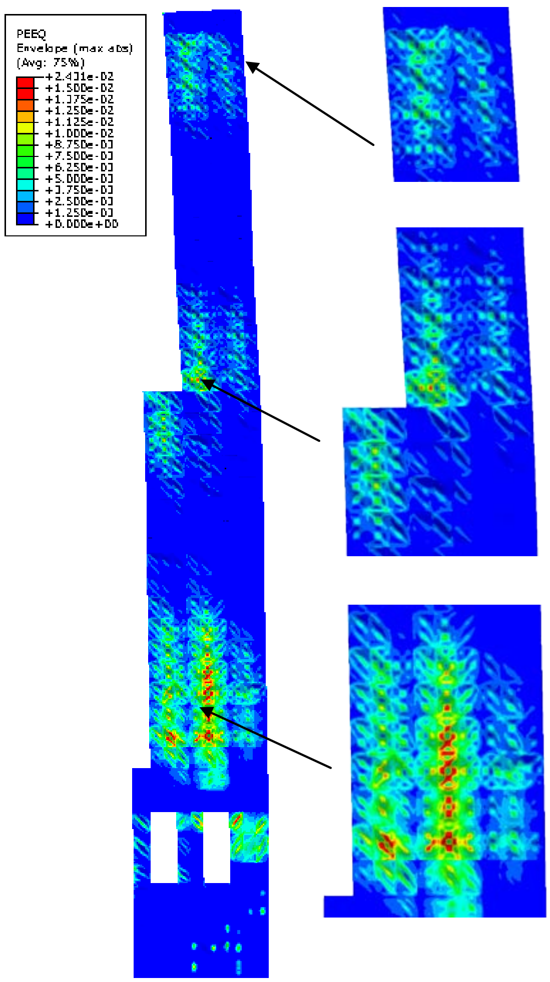



Fig.13 SPSW plastic buckling (tension field action)

## CONCLUSIONS

Un-stiffened steel plate shear walls (SPSWs) are an efficient and economical lateral load resisting system solution for high rise buildings. Their high initial stiffness very effectively limits story drifts in frequent earthquake and wind events. Moreover, they are very ductile and have large energy dissipation capacity; a characteristic vital for resisting large earthquake events. However, using SPSWs in super tall buildings like the 330m Jinta Tower presents big challenges. The SPSWs end up carrying gravity loads by virtue of their connection to the vertical boundary elements (columns) if details that are efficient (time and material) and economical to construct are used. This leads to premature buckling of steel plates in frequent seismic events and hence raises serviceability concerns (loss of stiffness and acoustic effects).

To address this concern, a buckling restrained slender steel plate shear wall system—SPSWs with vertical stiffeners to enhance the gravity load carrying capacity of the plates and delay buckling—is developed. Different from and requiring less steel material than traditionally stiffened SPSWs which yield in shear, they incorporate a 100mm gap between the ends of the vertical stiffeners and the horizontal boundary elements to ensure that they act as buckling stiffeners primarily and not as columns. The discontinuous stiffeners are designed to satisfy the performance objectives prescribed by the Chinese code; no buckling in frequent wind or seismic events.

Nonlinear analysis of the structure is used as a primary tool in the performance based design. Fine meshing of the structural members is used to pick up the energy dissipating features (tension field action effects) accurately. Simplified one bay-one story nonlinear push over models were used to verify that the stiffened SPSWs would not buckle under frequent seismic or wind events, while a fine detail nonlinear dynamic earthquake analysis of the entire structure is used to check if the performance objectives under the rare earthquake are achieved.

After a couple of trial and error iterations, the new buckling restrained SPSW system with discontinuous vertical stiffeners is demonstrated to push the inception of plate buckling out beyond the frequent seismic and wind events as required by the Chinese code and provide great ductility and ability to dissipate energy through significant tension field action in the larger seismic events. The prescribed performance objectives for the structure are achieved.

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