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Title: **The ASeismic Design and Nonlinear Dynamic Analysis of a 350m High Braced Steel Frame**

Authors: Ziguo Xu, Director of Supertall Building Research Center, China Academy of Building Research
Chongcui Ren, Director of Structure Engineering and Consulting Department, China Academy of Building Research
Congzhen Xiao, Director of Structure Engineering and Consulting Department, China Academy of Building Research

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The ASeismic Design and Nonlinear Dynamic Analysis of a 350m High Braced Steel Frame

Ziguo Xu
Director of Super-tall Building Research Center

China Academy of Building Research, Beijing, China

Dr. Ziguo Xu is a Professor of China Academy of Building Research in Beijing, China. His field of specialty is in the performance based seismic design and evaluation of tall buildings. Since serving work in the academy, Dr. Ziguo Xu and his team have completed over 90 real high-rise buildings' seismic performance evaluation tasks including ShangHai center(632 meters high), Beijing China Zun(536 meters high) with elastoplastic dynamic analysis method and/or shaking table test. Owing to the efforts on tall building structure research, Ziguo was assigned as first director of Supertall Building Research Center and adjunct professor of Guangzhou University.

Chongcui Ren
Director of Structure Engineering and Consulting Department

China Academy of Building Research, Beijing, China

Dr. Ren Chongcui was the engineer of the China Academy of Building Research. She was in the Structure Consulting Department of CABRTECH Engineering Consulting & Design Institute. Her field of speciality is the analysis and consulting, and in particular the dynamic elasto-plastic analysis and consulting, of high-rise buildings. She graduated from Harbin Institute of Technology in structure engineering in 2008 and obtained the master degree. After that she came to work in CABR till now.

Congzhen Xiao
Director of Structure Engineering and Consulting Department

China Academy of Building Research, Beijing, China

Dr. Congzhen Xiao is deputy chief engineer in China Academy of Building Research, he is experienced in the research and consulting of tall and complex structures.

He has been involved in structural design of a number of innovated high-rise building in China such as Beijing Linked Hybride, Shenzhen Vanke Center, Raffles City in Chengdu, and so on. He also takes part in consulting of seismic design of many super high-rise building such as 632m Shanghai Tower and 660m Shenzhen Ping An Finance Center, meantime, he and his team have accomplished most shaking table test for building over 400m in China.

Abstract

Using nonlinear time history analysis to investigate the seismic performance of tall building structures has been more widely implemented in recent years as china new generation national codes for seismic design were promulgated in 2010, in which the systematic methodologies of performance-based seismic design are first presented. This paper firstly presents a brief introduction about the performance-based seismic design of a 350-meter high braced steel frame structure according the requirements of Chinese codes, including the determination of seismic action, the performance objectives and the methods to achieve the performance. And then the authors emphasize the investigation of the nonlinear dynamic responses of the structure under seven sets of 7 intensity rare level earthquake motions acting with elastoplastic time history analysis, each set of that is consisted of two transitional. The nonlinear responses of displacement, interstory drift, story & base shear forces and the yield mechanism are studied. The failure modes

Keywords: Braced steel frame structure, Nonlinear time history analysis, Performance Based Design

Introduction

Since emerging in the late nineteenth century in the United States of America, tall building has developed rapidly worldwide. With the development of the economics and building technologies in recent years, many tall buildings are built worldwide, especially in China.

Based on the newest statistic data published by the Council on Tall Buildings and Urban Habitat (CTBUH), the number of completed or under construction tall buildings whose total height are over 300 meters increases to 221 in the world and about 50% of which are located in Mainland China.

It is worth noting that such tall buildings with heights over 300 meters are virtually beyond the serviceable range of current Chinese structural design codes and specifications and are named code-exceeding high-rise buildings. The structural designs of code-exceeding high-rise buildings, especially aseismic designs, need adopt some special measures and meet more strict requirements. The design also should be submitted for passing the experts panel review as requested by the National Law of Administrative Licensing.

This paper herein firstly presents a brief introduction about the performance-based seismic design of a 350-meter high braced steel frame structure according the requirements of Chinese codes and comments of the experts panel, including the determination of seismic action, the performance objectives and the methods to achieve the performance. Then the detailed investigation of the nonlinear dynamic responses of the structure under seven sets of 7 intensity rare level earthquake motions acting with elastoplastic time history analysis, each set of that is consisted of two transitional. The nonlinear responses of displacement, interstory drift, story & base shear forces and the yield mechanism are studied. The failure modes under different strong earthquake acting were also discussed in this paper herein. The authors also proposed one method to evaluate the seismic performance of the structure based on capacity-demand criterion.

Description of Hanking Center Tower

The Hanking Center project (see Figure 1) is a new 70 stories office and residential tower with associated retail podium and parking located in Shenzhen China. The tower's height is approximately 320m with a screen wall extending an additional 30m. A 6 stories retail podium surrounds the tower above grade with 5 levels of basements containing parking, truck docks, and mechanical functions below ground. As the consequence of the innovative architectural design of rethinking the traditional commercial office building, the primary movement and

Figure 1. Hanking Center Tower Project (Source: Morphosis)

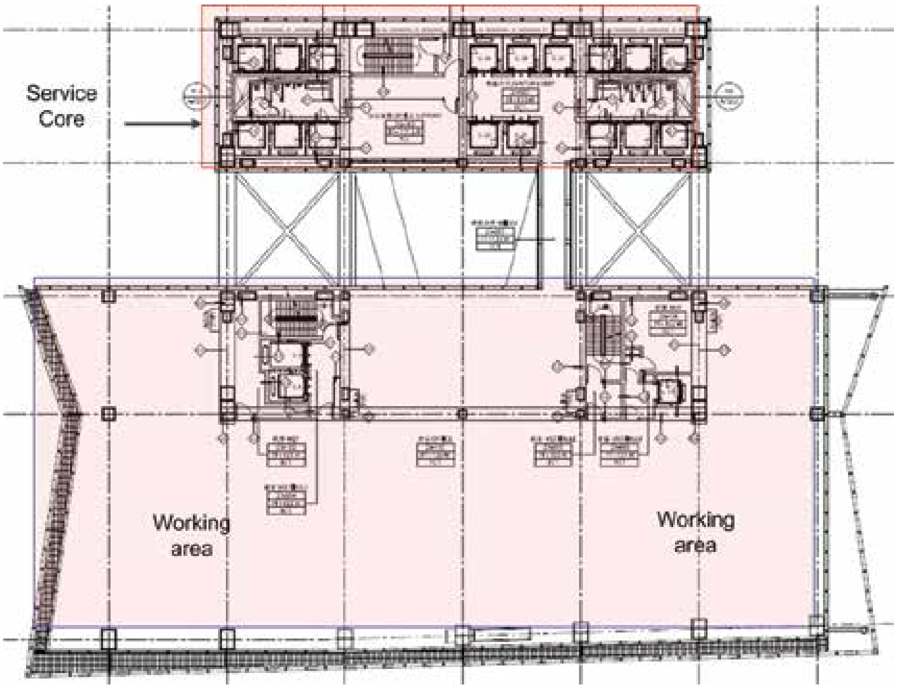

Figure 2. The Architetural plane (Source: China Academy of Building Research)

service core was offset to the exterior of the floor plate (see Figure 2). Offsetting the core increases space-planning flexibility, offers healthier working environments with enhanced natural light and airflow and allows for a public-to-private gradient of activity on each floor plate.

Taking architectural demands for space, panoramic views and structural cost into consideration, the structural form of the tower is intended to be steel structure. The lateral system of the tower is a mega-brace frame system, which consists of rectangular concrete filled tube (RCFT) columns, built-up steel mega bracing and steel beams on four column lines (line T-E on the north, the lines T-2 and T-6 on the west and east respectively) and the sloped front face of the building (see Figure 3). It should be noted that a closed tube is formed by columns, steel mega braces and beams, which is followed by a four story module between the intersection nodes of bi-way braces for the entire height of the tower, so that the integrity of whole structure is enhanced. Additional bracing is provided at every level surrounding the stair and elevator cores to compliment the mega-braced system by providing additional lateral strength and stiffness between the nodes. Further linkages are obtained with belt trusses at three mechanical levels, which provide additional load paths when a column or brace failed.

The floor system consists of steel-bar truss deck and composite steel beams. The thickness of the truss deck is 250 mm for refuge and mechanical floors and 120 mm for other floors.

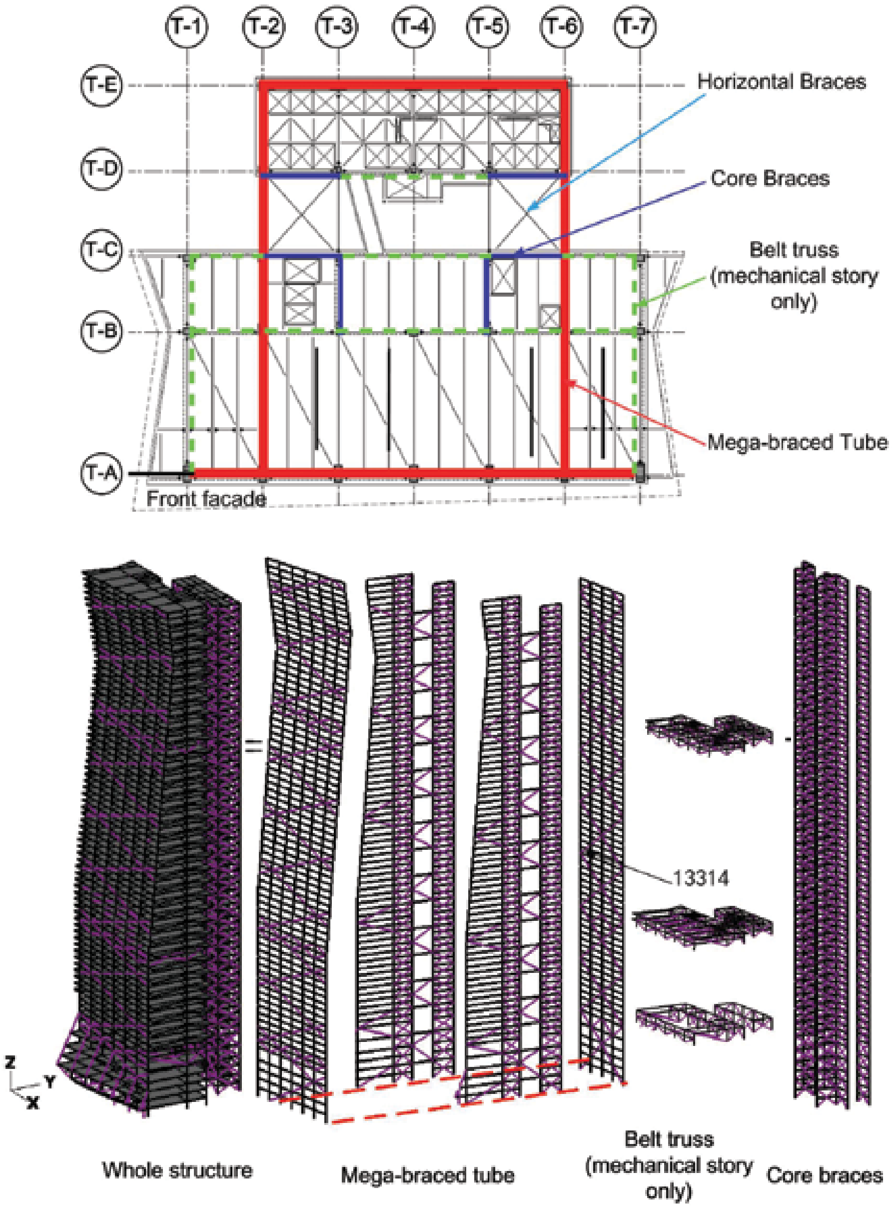

Figure 3. The Constitution of Structural System (Source: China Academy of Building Research)

		Performance Objectives			
		A	B	C	D
Earthquake Levels	Frequent Earthquake	1	1	1	1
	Medium Earthquake	1	2	3	4
	Rare Earthquake	2	3	4	5

Table 1. Overall Structure seismic performance objectives (Source: China Academy of Building Research)

Performance Levels	Damaged Conditions	Damaged Parts			Function Sustainability
		Key components[1]	Ordinary vertical components[1]	Energy Dissipating components[1]	
1	Good, No damage	No damage	No damage	No damage	Immediate occupancy without repair
2	Fine, Minor damage	No damage	No damage	Minor damage	Occupancy, a little of repair
3	Mild damage	Minor damage	Minor damage	Minor damage, part mild damage	Mal-function, need repair
4	Moderate damage	Mild damage	Part moderate damage	Moderate damage, part serious damage	Function interrupt, Need repair or fortification
5	Serious damage	Moderate damage	Part serious damage	Serious damage	Out of function, Overhaul

Table 2. The Structural Components performance Levels (Source: China Academy of Building Research)

It should be noted that Hanking Center Tower is a code-exceeding high-rise building. There are several other code-exceeding items besides structural height (350 meters, code limit is 300 meters), which are height-width ratio (7.3, code limit is not greater than 6), aspect ratio (1.83, code limit is not greater than 1.5) and reentrant irregularity respectively.

The Performance-based Seismic Design of Hanking Center Tower

Description of the performance-based seismic design in Chinese codes

The systematic methodology of performance-based seismic design was first presented in Code for Seismic Design of Buildings (GB 50011-2010, China) and Technical Specification for Concrete Structures of Tall Building (JGJ 3-2010, China), which were promulgated in 2010. There are four objective grades for overall structure (A~D, see Table 1) and five performance levels for structural members (1~5, see Table 2) recommended by the methodology of performance-based seismic design. Actually, such classification is the detailed expression about the essential ideology of china seismic design, which can be briefly addressed as "No damage under frequent earthquake (63% probability of exceedance in 50 years), Repairable under medium earthquake (10% probability of exceedance in 50 years) and No collapse under rare earthquake (2%~3% probability of exceedance in 50 years)". Corresponding to the general performances descriptions of overall structure and structural components that were listed in the table1 and Table 2, some specific requirements and design approaches were also prescribed in Chinese codes.

The Setting of Performance Objectives

The aseismic performance objectives of structural members should be determined by taking the seismic precautionary intensity, the structural seismic fortification category, situ seismic safety, the complexity and irregularity of structure, the importance of structural components for seismic-forces-resisting system and structural performance hierarchy into consideration. Refer to the regulations of Chinese Code for Seismic Design of Buildings (GB50011-2010) and the Chinese Standard for Classification of Seismic Protection of Building Constructions (GB50223-95), the precautionary seismic intensity and the structural seismic fortification category of Hanking Center project is 7 degree and class C respectively. Furthermore, Hanking Center Tower is a code-exceeding high-rise building, as stated above, and will be the tallest steel structure in China when it is built. The performance-based design methods shall be adopted in its structural aseismic design and the preliminary design needs be submitted for passing experts panel review. The final affirmatory performance objectives (Halvorson and Partners, 2013) of structural components are listed below (see Table 3).

As showed in Table 3, the overall seismic performance objective of Hanking Center Tower structure is C and the performance levels of structural components are between 1 and 4 based on their importance.

Besides the seismic forces, the wind loads are also dominant actions for the performance-based design of Hanking Center Tower. As prescribed by Chinese codes, the strength checks for structural members should utilize 100-year wind loads and all structural components should keep elastic. The design values for base shear and total overturning moment of wind loads (the reference pressure is 0.90 kPa, damping ratio is 2% and load factor is 1.4) are 69.6/106.7 MN (X/Y direction) and 15,100/23,800 MN-m respectively, which are larger than those of medium earthquake (the base shear and total overturning moment are 61.9/60.9 MN and 12,246/11,960 MN-m) and even larger than those of rare earthquake in Y direction (the base shear and total overturning moment are 90.0/95.0 MN and 17,800/19,200 MN-m).

The determination of seismic action

The mode-decomposition response spectrum method is the popular approach for evaluating the seismic forces sustained by structure for high-rise buildings. The Chinese Code for Seismic Design of Buildings gives the expression form of seismic response spectrum and provides the general values of parameters, such as maximum seismic influence coefficient (represented as αmax), site characteristic period (represented as Tg) and structural damping ratio (represented asζ), which are used to calculate the seismic design spectrum curve as well. In order to obtain more appropriate parameters' values, the investigation of site-specific probabilistic seismic hazard is usually required for important buildings, such as code-exceeding buildings, as a supplementary means. For Hanking Center project, the seismic design spectrum curve defined by the code's proposal values of parameters (αmax is 0.08 for precautionary seismic intensity 7 degree, Tg is 0.35 seconds andζ is 0.02 for steel structure under frequent and medium level earthquake and 0.035 for rare level earthquake) is different from that proposed by site seismic hazard report (see Figure 4). After consulting the experts of code-exceeding review panel, the formal frequent earthquake response spectra used for design was created by using the Chinese code equations, but the maximum seismic influence coefficient

1: Key components refer to those structural members whose failure could cause progressive damage or life-threating serious damage. Ordinary vertical components refer to other vertical structural members except key components. Dissipative components include framed beams, coupling beams, braces and dissipative devices.

and site characteristic period should be amended by taking account of the site seismic hazard report and geotechnical investigation report respectively. Besides multiplying the adjustment coefficient of structural damping ratio (calculated as $\eta_1=1+(0.05-\zeta)/(0.08+1.6*\zeta)=1+(0.05-0.02)/(0.08+1.6*0.02)=1.27$), the maximum seismic influence coefficient was calculated using the peak ground acceleration, 38gal, from site seismic hazard report. $\alpha_{max}=(2.25*PGA)*\eta_1/g=(2.25*0.38m/s2)*1.27/9.8\ m/s2=0.113$. The site characteristic period (represented as Tg) was determined to be 0.38 seconds by referring to the regulations in the code with a shear wave velocity of 225 m/s2.

Furthermore, the seismic response spectra of medium earthquake and rare earthquake needed to be defined for checking the performance objectives of structural components. According the comments of experts of code-exceeding review panel, the maximum seismic influence coefficients for medium and rare level earthquakes should be amplified by the ratio of the site seismic hazard report value to that of code for frequent earthquake besides multiply adjustment coefficients of respective structural damping ratios, that are 0.23 (code value)*1.27*0.089/0.08=0.32 and 0.5*1.16*0.089/0.08=0.65 respectively.

The methods in seismic performance design

Corresponding to the status of overall structure and structural components under different level earthquakes (see Table 3), the analysis methods used in the seismic performance design of Hanking Center Tower involves elastic analysis, equivalent elastic analysis and elastoplastic analysis.

The elastic analysis methods mainly implemented for frequent level earthquake, including linear response spectrum analysis and linear time history analysis, are mainly used to check the structural regularities (such as vertical regularity, torsional irregularity, weak story check, soft story check, etc.), inspect the requirements of overall structural performances (such as inter story drift, minimum shear-weight ratio, story stiffness-weight ratio) and design those structural components without higher seismic performance objectives.

For medium and rare level earthquakes, as showed in Table 3, some structural components of Hanking Center Tower would yield and the overall structure would not be keeping elastic anymore. For considering the effects of such situations, there are two measures in Hanking structure performance design:

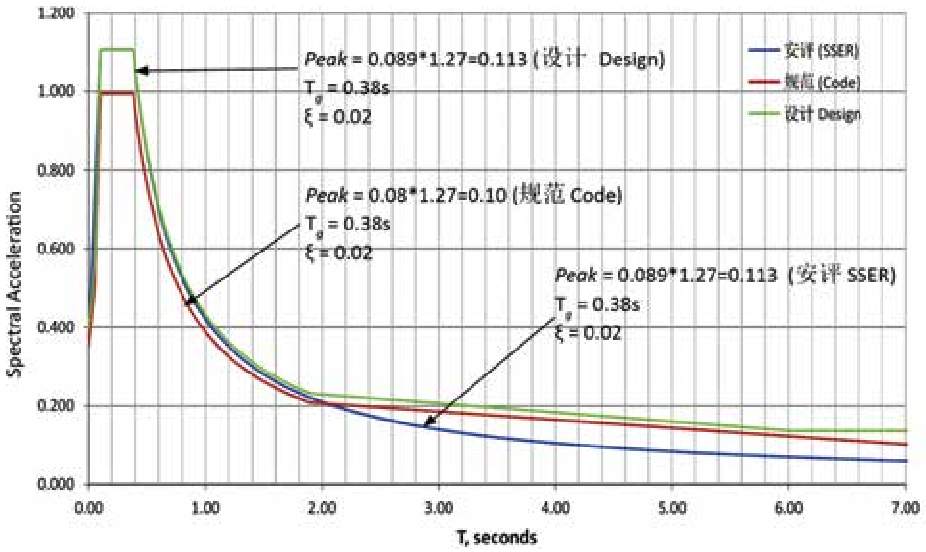

Figure 4. The comparison of seismic spectra (frequent earthquake) (Source: H+P, 2013)

	Frequent earthquake	Medium Earthquake	Rare Earthquake
Basic Performance Description	No damage; Occupancy without repair	Mild damage; Occupancy after repair	Moderate damage; Fortification need for occupancy
Overall structure performance	Maintain elastic	Yielding on Core/Perimeter Beam and Core Bracing members only	Inelastic; Limited Yielding and Buckling within LS deformation limit
Story drift limit	H/300	---	H/50
RCFT Columns	No Damage; Remain Elastic	No Damage; Remain Elastic	Mild Damage; Inelastic, limited yielding within IO deformation limit
Mega Bracing	No Damage; Remain Elastic	Not yielding	Mild Damage; Inelastic, limited yielding within IO deformation limit
Belt truss	No Damage; Remain Elastic	Not yielding	Mild Damage; Inelastic, limited yielding within IO deformation limit
Corridor/Diaphragm Brace at every 4 levels	No Damage; Remain Elastic	No Damage; Remain Elastic	Not yielding
Core/Perimeter Beams	No Damage; Remain Elastic	Mild Damage; Inelastic, limited yielding within IO deformation limit	Moderate Damage; Inelastic, Limited yielding within LS deformation limit
Core Bracing	No Damage; Remain Elastic	Mild Damage; Inelastic, limited yielding within IO deformation limit	Moderate Damage; Inelastic, Limited yielding within LS deformation limit

Table 3. The Performance objectives of overall structure and components of Hanking Center Tower (Source: H+P, 2013)

1) Using larger structural damping ratio for creating seismic response spectra (i.e. 3.5% for rare level earthquake) and for performing the response spectrum analysis. 2) In structural component strength performance objectives of NOT YIELDING and No Damage checking, using standard combinations (1.0 Dead Load+0.5 Live Load±1.0 Earthquake Load) and design combinations (1.2 Dead Load+0.6 Live Load±1.3 Earthquake Load±0.7 Wind Load, 1.0 Dead Load±1.3 Earthquake Load±0.7 Wind Load) respectively for determining load effects and material standard strength for evaluating bearing capacity of structural component. Although the response spectrum analysis is still elastic, the analysis/design method mentioned above is named equivalent elastic analysis because of considering the energy dissipation effect due to inelasticity.

Furthermore, inelastic behavior is expected to occur under rare level earthquakes as described in Table 3. Therefore, nonlinear analysis for Hanking Center Tower is required to model the inelastic effects, and to demonstrate that the applicable performance standards are meeting. In addition to evaluating the building's performance according to the above performance standards, the nonlinear analyses are also used to achieve the followings: 1) Verify overall structural behavior and ductility of the system. The adopted criterion for steel ductile limits is the maximum allowable plastic strain at any section of element is 0.05. 2) Verify interstory drifts are within prescribed code limits h/50 for rare earthquakes. 3) Identify any weak points and address undesirable failure mechanisms that may exist based on rare earthquake demand. 4) Confirm sufficient strength of critical elements for rare earthquake loading.

The Nonlinear Dynamic Analysis of Hanking Center Tower

Descriptions about the analysis method

To provide deeper insight into the dynamic responses of the structure, a serial nonlinear

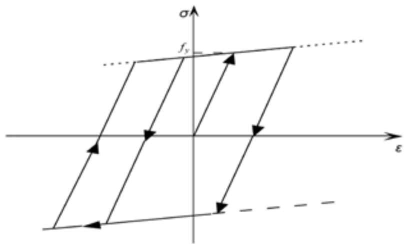

Figure 5. Bilinear kinematic hardening model for steel (Source: China Academy of Building Research)

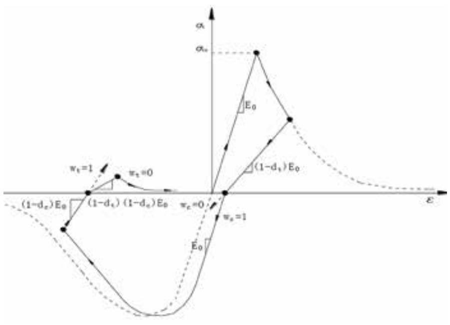

Figure 6. Plastic damage model for concrete (Source: China Academy of Building Research)

history analysis for Hanking Center Tower structure were executed. The analysis was carried out by using the commercial finite element analysis program - Abaqus.

The selected constitutive law for steel material is the bilinear kinematic hardening model without stiffness degradation during the loading and unloading cycles (see Figure 5). The selected constitutive law for concrete is uniaxial cyclic law with monotonic envelope governed by the recommended curve in the Chinese Code for Design of Concrete Structure (GB 50010-2010). The implemented cyclic behavior is characterized by linear unloading–reloading branches with progressively degrading stiffness (see Figure 6). After each unloading/reloading sequence, the monotonic envelope is reached again when the absolute value of the largest compressive strain attained so far is surpassed. The concrete in tension follows the same loading/unloading/reloading rules as in compression with the same initial stiffness and appropriate values for the other parameters.

It should be noted that all material strengths are adopted the standard values instead of averaged values or expected values recommended by the Chinese code and the confinement effects for concrete caused by steel tubes are not considered. The reasons for such conservativeness are the quality of concrete casted in situ could be imperfection and to keep consistent with steel strength value, which is no expected value in Chinese code.

In the structural finite element model, different finite element formulations for structural members were adopted based on the deform behavior: quadrilateral or triangular shell element is used for representing the slab and three-node space beam-column element based on fiber model is used for representing column, beam and brace. In the calculation process, the gravity analysis is firstly performed with construction sequence, and then the dynamic inelastic analysis continued on that basis. The adopted time integration scheme is the explicit central different method.

Moreover, the mechanism of energy dissipating in the calculation was supposed to be contributed by two sources: the structural damping and the hysteresis of plasticity. The former was introduced into the calculation with 3% modal damping form and remained constant whether the structure was elastic or not. The amount of latter is automatic counted as the extent of plasticity.

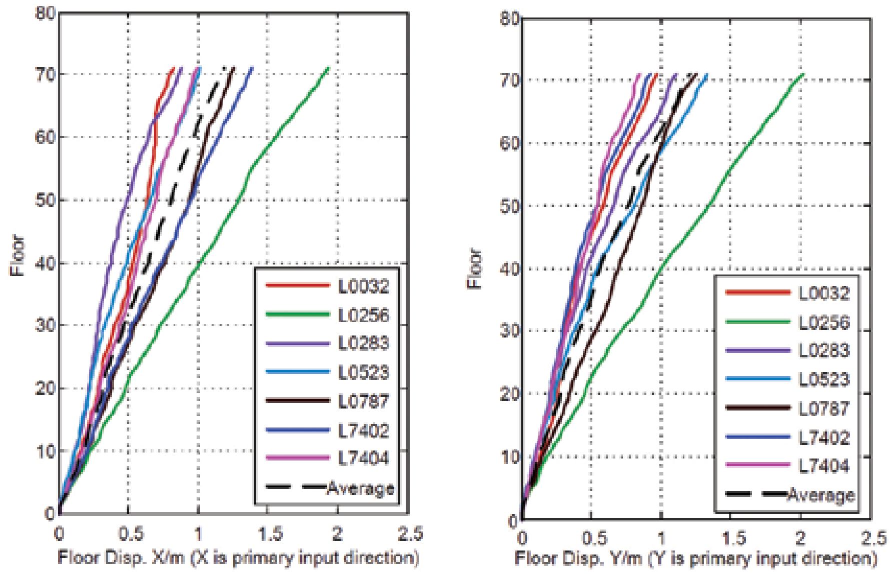

Figure 7. Peak floor disp. distributions (Source: China Academy of Building Research)

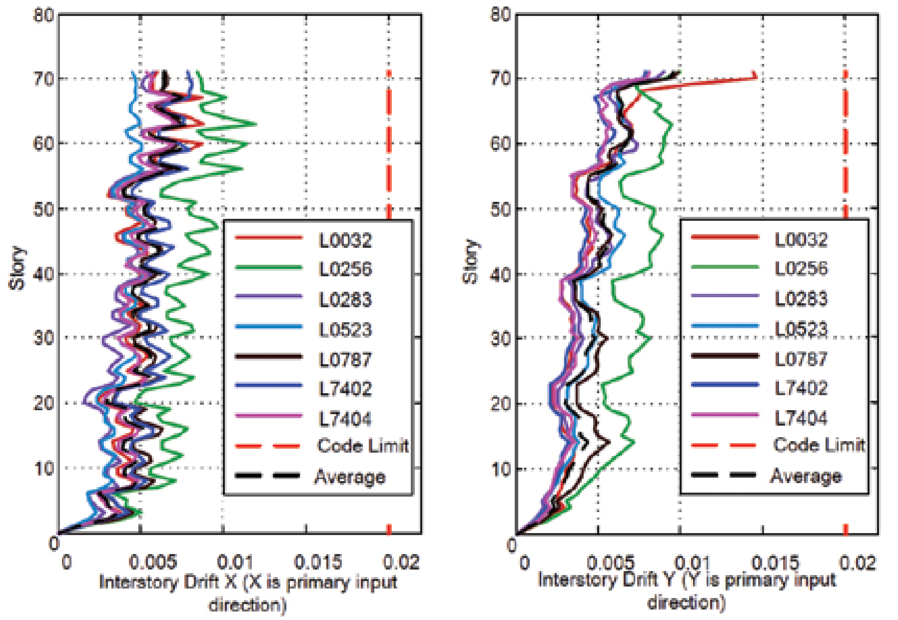

Figure 8. Peak interstory drift distributions (Source: China Academy of Building Research)

Selection and Input Method of Accelerograms

Total 14 waves, seven sets of bidirectional ground motion records (2 sets of artificial records and 5 sets of actual earthquake records) based on the rare earthquake level were selected for response history analysis. All earthquake records were scaled (in amplitude) to have a peak ground acceleration (PGA) of 220 cm/s2. Two schemes of ground motion records input were determined by the first two translational mode shapes of vibration: one combination is taking the directions of first and second mode vibration shapes as primary and secondary input directions respectively, the other combination is taking the directions of second and first mode vibration shapes as primary and secondary input directions respectively. The adjustment factors of peak ground acceleration for primary and

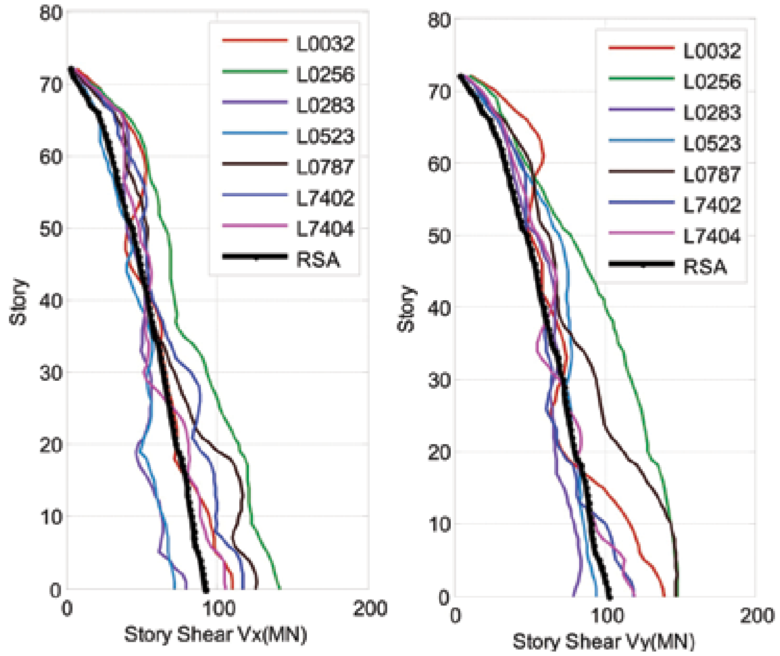

Table 9. Peak story shear distributions (Source: China Academy of Building Research)

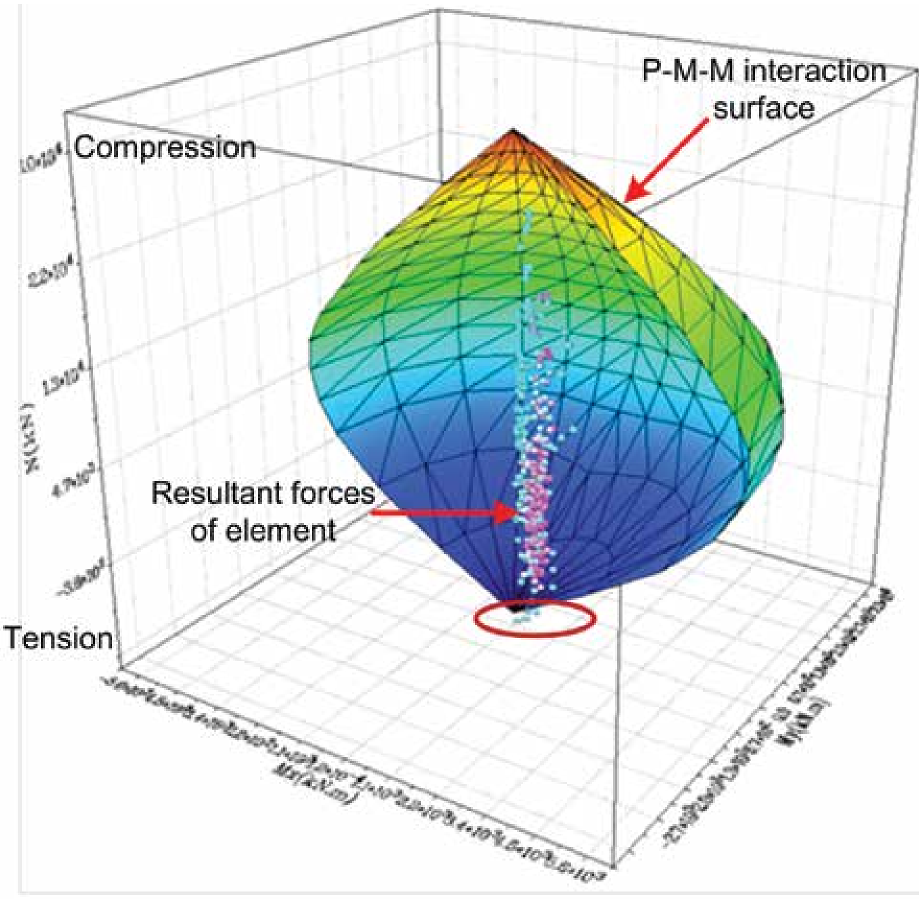

Table 10. The yield mechanism of CFT column at top of the structure (Source: China Academy of Building Research)

secondary input direction are 1.0 and 0.85 respectively. So the total amount of analysis cases is 7 (sets of records)*2 (schemes of records input) =14.

Analysis Results
Self-Weight and Vibration analysis

The first 6 natural vibration periods and the vibration characteristics are 6.46 (X-translation), 5.60 (Y-translation), 2.96 (Z-torsion), 2.13 (X-translation, 2nd order), 1.81 (Y-translation, 2nd order), 1.16 (Z-torsion, 2nd order) respectively. It can be seen that the natural vibration shape is fine. The structure has good torsion-resisting capacity and the ratio between the first torsion period and the first translational period is 2.96/6.46=0.46.

The equivalent self-weight (1.0Dead Load+0.5Live Load) of overall structure is about 154,211 tons.

For economy of space, the dynamic responses of structure are only presented and discussed with the results in primary input directions described in the next paragraph.

Structure deformations

In figures 7 and 8 we can see the distribution curves of the maximum floor displacements and interstory drifts in primary input direction under selected records acting.

The averages of the maximum roof displacements and the maximum peak interstory drifts from seven sets of records are 1187 mm, H/126 as X is primary input direction and are 1209 mm, H/104 as Y is primary input direction respectively.

Story & base shears

It could be seen from the distributions of maximum story shear forces under seven sets of records (see Figure 9) that the shear forces of upper stories are obviously larger than those of elastic response spectrum analysis due to higher mode effects.

The averages of the X and Y as primary input direction maximum base shear forces from seven sets of records are 107,361kN and 121,101kN respectively.

The seismic performance and yield mechanism of structure

It is one effective way to evaluate the seismic performance of overall structure by comparing the basic results (i.e. base shear, roof displacement, etc.) of nonlinear history analysis with those of elastic history analysis.

The ratios of average base shear force and roof displacement from elastoplastic history analyses to those from elastic history analysis are 97.2%, 99.2% as X is primary input direction and 91.7%, 97.5% as Y is primary input direction respectively. Commonly, the base shear responses will decrease as the plasticity in structure increases. The average base shears of elastoplastic analyses are only slightly smaller than those of elastic analyses.

As state above, general speaking, the performance of overall structure under the selected seven sets of rare earthquake records acting was essentially elastic.

According to the distributions of plastic strain in critical components, the behaviors of structural components are resumptively

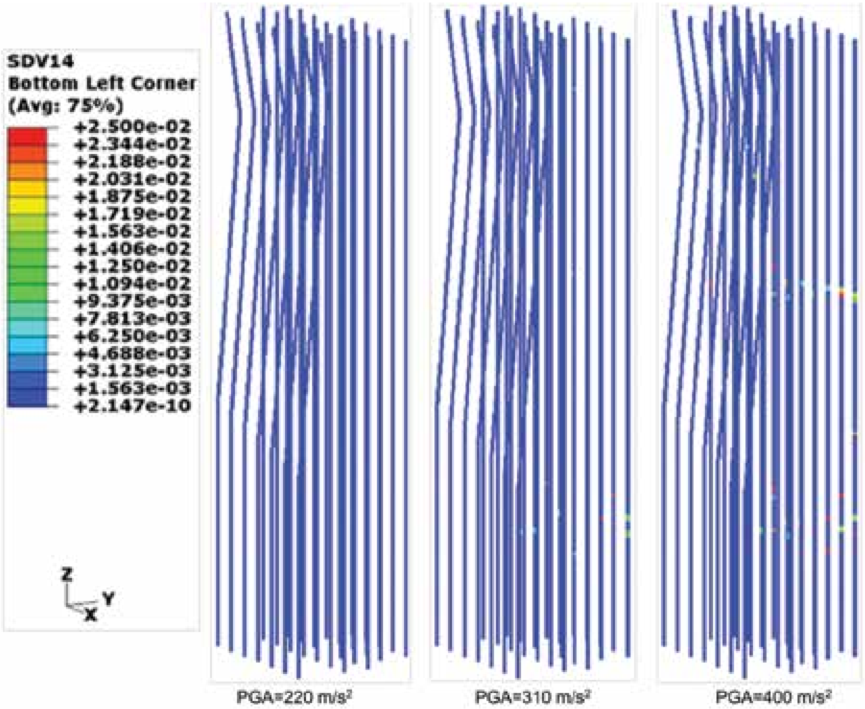

Figure 11. the distributions of plastic strain in CFT columns under different PGA earthquakes (Source: China Academy of Building Research)

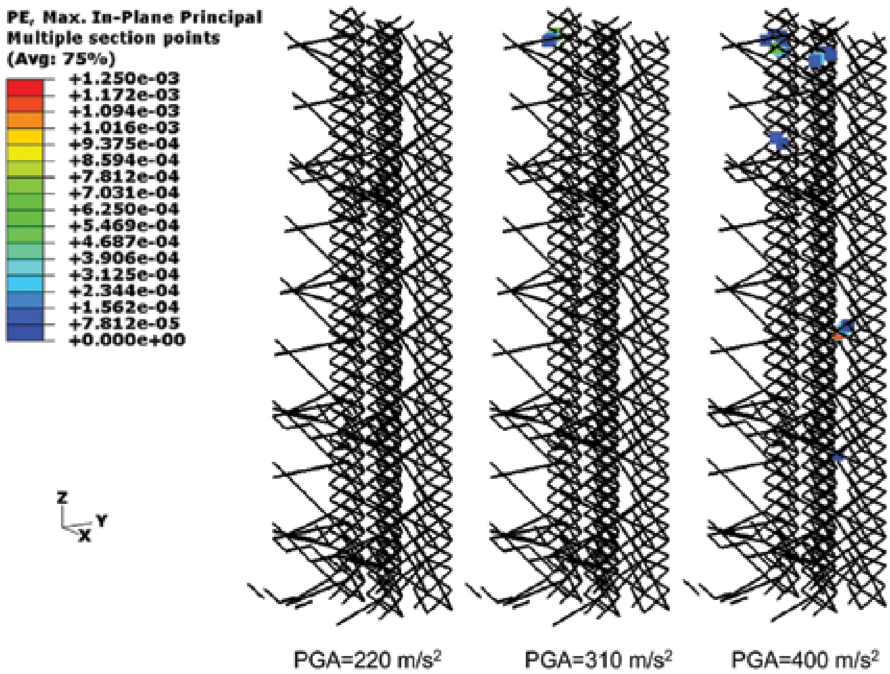

Figure 12. The distributions of plastic strain in braces under different PGA earthquakes (Source: China Academy of Building Research)

described as follows: 1) Some steel beams at those floors where the intersection nodes of braces located yielded. 2) Several columns located at upper structure went into plastic range. 3) All braces kept elastic.

Despite the fact that the braces go plastic prior to columns and controlled brace yielding is a favorable energy dissipation strategy for seismic engineering, the real performance for bracing were actually determined by the wind loads.

The further investigations about the yield mechanism of CFT column atop were taken as showed in Figure 10. The resultant forces (pairs of axial forces, x-moment and y-moment) which were represented as tiny balls at every time interval and the P-M-M interaction surface (was defined by moment-curvature analyses with changing axial forces and flexural angles) of member' section were plotted together.

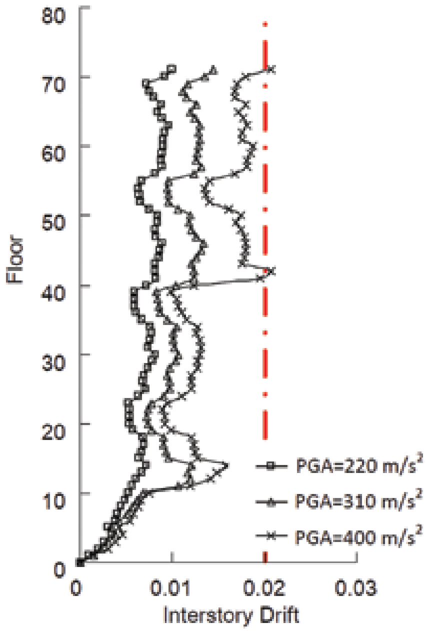

Figure 13. The distributions of interstory drift under different PGA earthquakes (Source: China Academy of Building Research)

It can be clearly seen that the axial force is the primary loading mode for the CFT column and the yield mechanism was mainly tension-yielding type.

The failure mode of structure under higher intensity earthquake

To investigate structural responses under intensive earthquakes, especially the structural failure modes, are of importance for aseismic engineering all the time. In a sense, to grasp the potential failure modes of structure means to seize the soul of seismic design for structure. In order to investigate the seismic performance of Hanking Center Tower structure and seek the potential failure mode under higher intensity earthquakes, additional nonlinear history analyses were performed with increasing the peak ground acceleration of natural records L0256 to 310 cm/s2 (7.5 intensity rare level earthquake in code) and 400 cm/s2 (8 intensity rare level earthquake in code) respectively.

It could be seen that the plasticity in columns (see Figure 11) and braces (see Figure 12) developed more intensive and extensive as the PGA increasing. In terms of magnitude only, there is more plasticity in columns than that in braces under high intensity earthquakes acting. Refer to the distributions of interstory drift under different PGA earthquakes (see Figure 13), it would be likely to obtain following conclusions: 1) the seismic performance of structure is excellent. The maximum interstory drift is still less that the code limit value of 1/50, 2) the weak story of the structure are 11th story and 41th story, 3) the potential failure mode of structure, as the PGA increasing, would be: the columns at 11th story and 41th story yielded intensively and lost bearing capacity finally.

It is technically inappropriate for preventing progressive collapse that columns go yielding even lost bearing capacity, although the building would be still standing under earthquake of intensity higher than that of precautionary. This confusing situation might be caused by the current strength design method, enveloping of multiple loads combinations, and the complexity or divergence of nonlinear dynamic problem.

Summary

This paper herein firstly presents a brief introduction about the performance-based seismic design approach in Chinese codes and its applications in a 350-meter high braced steel frame structure — Hanking Center Tower, including the determination of seismic action, the performance objectives and the methods to achieve the performance. The nonlinear history analysis implemented in the structure design was stressed. The nonlinear responses of displacement, inter-story drift angle, story & base shear forces and the yield mechanism are studied. The failure mode under higher intensity earthquakes acting was also discussed. It has been demonstrated by the results of nonlinear time-history analysis that the performance-based seismic design approaches proposed in the Chinese codes are effective in improving the seismic performance of Hanking Center Tower.

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