Structural Engineering and Mechanics, Vol. 43, No. 4 (2012) 545-560
DOI: http://dx.doi.org/10.12989/sem.2012.43.4.545

Performance-based seismic analysis and design of code-exceeding tall buildings in Mainland China

Huanjun Jiang*[1], Xilin Lu[1a] and Jiejiang Zhu[2b]

[1]*State Key Laboratory of Disaster Reduction in Civil Engineering, Tongji University, Shanghai, P.R. China*
[2]*Department of Civil Engineering, Shanghai University, Shanghai, P.R. China*

(Received April 5, 2011, Revised May 29, 2012, Accepted August 7, 2012)

Abstract. Design codes provide the minimum requirements for the design of code-compliant structures to ensure the safety of the life and property. As for code-exceeding buildings, the requirements for design are not sufficient and the approval of such structures is vague. In mainland China in recent years, a large number of code-exceeding tall buildings, whether their heights exceed the limit for the respective structure type or the extent of irregularity is violated, have been constructed. Performance-based seismic design (PBSD) approach has been highly recommended and become necessary to demonstrate the performance of code-exceeding tall buildings at least equivalent to code intent of safety. This paper proposes the general methodologies of performance-based seismic analysis and design of code-exceeding tall buildings in Mainland China. The PBSD approach proposed here includes selection of performance objectives, determination of design philosophy, establishment of design criteria for structural components and systems consistent with the desirable and transparent performance objectives, and seismic performance analysis and evaluation through extensive numerical analysis or further experimental study if necessary. The seismic analysis and design of 101-story Shanghai World Financial Center Tower is introduced as a typical engineering example where the PBSD approach is followed. The example demonstrates that the PBSD approach is an appropriate way to control efficiently the seismic damage on the structure and ensure the predictable and safe performance.

Keywords: performance-based seismic design; tall buildings; performance objective; non-prescriptive; performance evaluation

1. Introduction

Conventional strength-based methods of seismic design have the basic objective to provide for life safety. There are uncertainties concerning the seismic demand and seismic capacity of the structure. One major drawback of this traditional design approach is that it does not directly address structural inelastic seismic responses and thus cannot effectively deal with damage loss due to structural and nonstructural failure during earthquakes (Zou1 *et al*. 2008). The progress that took place in the last two decades in the fields of computational mechanics and hardware technology made it possible to

*Corresponding author, Professor, E-mail: jhj73@tongji.edu.cn
[a]Professor, E-mail: lxlst@tongji.edu.cn
[b]Professor, E-mail: zhujjt@126.com

employ more realistic design procedures based on nonlinear analysis in place of procedures based on linear analysis. Performance-based seismic design (PBSD) concepts have been introduced over the last 15 years by various guidelines, such as SEAOC Vision 2000 (1995), ATC-40 (1996), FEMA-356 (2000), etc., for the assessment and rehabilitation of existing structures and the analysis and design of new ones. The need to improve seismic performance of the structures through the development of performance-oriented procedures has been repeatedly highlighted (Chandler and Lam 2001). Up to now, the philosophy of performance-based seismic design has been sufficiently developed. In several countries, seismic design is in the process of fundamental change, with the emphasis changing from "strength" to "performance". The next generation of seismic design codes is expected to incorporate, to some degree, the principles of PBSD (Bommer and Pinho 2006).

The general methodology for PBSD could be classified into two types, indirect PBSD and direct PBSD. For indirect PBSD, after the conceptual design phase traditional forced-based analysis is conducted to quantify the forces or stresses induced and initial design of structural components and systems is conducted at first, then the deformation or seismic damage is estimated and checked against pre-established limit, and the design should be modified until the pre-defined performance objectives could be achieved (Arzoumanidis *et al*. 2005). It can be easily applied and especially adequate to irregular structural form in current practice. The direct PBSD approach starts directly by predetermined displacement or damage index consistent with the design performance level, proportions the structure and then conducts the response analysis. In the latter type of approaches, the direct displacement-based seismic design is one of the most suitable procedures which can easily be incorporated into PBSD philosophy (Priestley and Kowalsky 2000). Although the direct displacement-based seismic design procedure appears promising, it has not been mature enough to be applied directly to various structures (Xue *et al*. 2008), and it is appropriate mainly to regular structures. Most research efforts in PBSD field in recent years involve seismic hazard and loss analysis, selection and modification of ground motions, development of nonlinear computer analysis model, seismic demands prediction, damage evaluation, etc. (Scotta *et al*. 2009, Vamvatsikos *et al*. 2010). Given the inherent uncertainty and variability in seismic response, it follows that a PBSD methodology should be formalized within a probabilistic basis. The Pacific Earthquake Engineering Research Center (PEER) has embarked on a more robust methodology, the second generation of PBSD procedures for performance-based earthquake engineering (Moehle 2004).

Since the 1980s, as a result of rapid economic growth and urbanization, many tall buildings have been constructed in Mainland China. The development of tall building structural systems has been accelerated in recent years. Owing to the wide variety of social requirement for commercial or aesthetic purposes, the limited availability of land, and the preference for centralized services, the height of tall buildings has grown taller, and the configuration as well as structural system has become more complex in recent years, resulting in a large number of code-exceeding tall buildings. The uniqueness in these structures beyond the scope of current design codes brings new challenges to engineers, since their structural behavior is difficult to predict and evaluate. The design codes typically provide minimum requirements for the design of code-compliant structures to ensure safety of life and property. As the code-exceeding buildings are concerned, although the use of alternate method which is non-prescriptive is permitted, the procedures and requirements of such non-prescriptive design have not been well defined. Usually, the details of design are required to work out in each case. In Mainland China in recent years, PBSD approach has been highly recommended to employ in the design of tall buildings with irregularity or height beyond the code specification in engineering practice in order to control the seismic damage and economic losses, to promote the implementation of the advanced technology in construction, and to meet the diverse needs and objectives of the owners, users and society. However, it has not been reached the agreement on generalized PBSD methodologies for tall buildings beyond the scope of design codes in engineering practice. For each project, a seismic peer review panel shall be convened and provide an independent, objective, technical review of those aspects of the structural design of the building that relate to seismic performance. It is often time consuming.

Urban regions along the west coast of the United States have been seeing the resurgence in construction of tall building in recent two decades. Many of these buildings use high-performance materials and framing systems that are not commonly used for building construction or that fall outside the height limits of current buildings codes (Moehle 2008). Design of these tall buildings is usually outside the limits of the code prescriptive provisions. Some guidelines, such as SEAONC (2007) and LATBSDC (2008), have been developed to provide an alternate, performance-based approach for seismic design and analysis of tall buildings. These guidelines are primarily based on the current design codes and engineering practice in the United States.

In order to achieve a level of consistency in the seismic design of code-exceeding tall buildings and reduce the case-by-case work involved in the seismic peer review in Mainland China, a general PBSD approach incorporating the experience gained from previous practice and current codes is summarized for code-exceeding tall buildings in this study. Enhanced seismic performance objectives are developed, taking the seismic protection category and code-exceeding type into consideration. The design-specific criteria and performance verification methods are outlined. The performance-based seismic design procedure is formed accordingly. A typical engineering example following this approach is provided later.

2. Performance objectives

Seismic performance objectives could be defined as the coupling of expected performance levels with expected levels of seismic ground motions. Three levels of seismic hazard, minor or frequent earthquake with the exceeding probability of 63.2% in 50 years (50 year return period), moderate or basic earthquake with the exceeding probability of 10% in 50 years (475 year return period), and strong or rare earthquake with the exceeding probability of 2% in 50 years (2475 year return period), are considered in Chinese national seismic design code (Ministry of Construction of China 2010). The minimum seismic performance objectives for ordinary buildings (including tall buildings) specified in the code are summarized as fully operational under minor earthquake, repairable under moderate earthquake, and collapse prevention under strong earthquake.

For tall buildings beyond the scope of design codes, the enhanced performance objectives are proposed herein. The relationships between the performance levels and earthquake design levels are summarized in Table 1.

The seismic protection category is classified into four grades according to the importance of the building and the consequence of earthquake disasters. Type I is the highest grade. For tall buildings, the lowest grade, Type IV, is excluded. In Chinese code for concrete structures of tall buildings (Ministry of Construction of China 2002), there are two classes of structural height specified, Class A and B. The height limit for Class B is much larger than Class A. If the height is larger than the limit for Class A or the extent of irregularity is violated, the building is classified as the type beyond the scope of design codes. The performance level of operational defined here means the

Table 1 Seismic performance objectives for code-exceeding tall buildings

Seismic protection category	Seismic performance level		
	Frequent earthquake	Basic earthquake	Rare earthquake
I	Fully operational	Fully operational	Operational
II	Fully operational	Operational	Repairable
III (RC structures with Height Class B and irregularity within the code)	Fully operational	Repairable	Collapse prevention
III (structures except above)	Fully operational	Operational	Repairable

post-earthquake damage state in which very limited structural damage occurs. The basic vertical and lateral force resisting structural systems retain most of their pre-earthquake characteristics and capacities. Although some minor structural repairs may be appropriate, there would generally not be required prior to reoccupancy.

3. Design philosophy

Different from the seismic design of ordinary code-compliant tall buildings, the most important task for seismic design of code-exceeding tall buildings is to demonstrate that the desired seismic safety and performance objectives can be assured in spite of the existence of unfavorable code-exceeding conditions by taking effective measures to counteract the negative impacts exerted by the code-exceeding conditions. The key components and potential weak positions related to the code-exceeding conditions should be identified and consequently additionally strengthened so that they no longer fail first or suffer severe damage. The common design philosophy, such as weak beam and strong column, weak flexural strength and strong shear strength, and weak member and strong joint, is also employed to adjust the strength and then the reinforcement of the structural component. Good understanding of structural behavior under the earthquake is prerequisite to accomplish this task. Good engineering practice and judgement are vital in some cases. Sufficient evidence for the rationality of the structural solutions and realization of the pre-defined seismic performance objectives should be provided by comprehensive analytical studies and/or testing.

4. Design criteria and seismic performance evaluation

4.1 Design criteria

The design criteria should be established corresponding to the desired performance objectives. These minimum acceptance criteria ascertain that the performance objective will be accomplished. The criteria are set in terms of limit values of configuration, distribution of stiffness and strength along the height of the structure, axial load ratio (specified for RC columns and shear walls), stresses, strains, inter-story drift ratio, the ratio between the maximum floor displacement (maximum lateral displacement of the vertical structural members) and the average floor displacement (floor displacement ratio for short later) to control the torsional response, etc. In particular, the identified key components and potential weak positions, and the corresponding limit values of responses should be addressed so as to enhance their seismic performance. In addition, the constructional measures, such as minimum reinforcement ratio, minimum material strength grade, reinforcement detailing, etc., are required to reduce structural damage.

The building configuration is generally defined as the size, shape and proportions of the three-dimensional form of the building. The extended definition of configuration also includes the nature, size and location of the structural members. The configuration establishing conceptual design is often determined by architectural design of the building, and is a subject of mutual agreement between architect and engineer. It plays an important role in determining the building's seismic performance. It has been well accepted that the configuration should be as regular as possible. In Chinese national design code, two types of irregularities, plan and elevation irregularities, are defined. The code approach reducing the detrimental effect of irregularity is to require more advanced methods of analysis and even carry out structural tests if necessary.

4.2 Seismic performance evaluation

4.2.1 Numerical analysis

Numerical analysis ranging from simple frame procedures to an elaborate finite element analysis is performed to verify the seismic performance of the structure. The performance is checked under individual level of earthquake. The elastic analysis is conducted under the frequent earthquake while nonlinear analysis, including nonlinear pushover (static) analysis and time history (dynamic) analysis, is done under other level of earthquake as nonlinear responses are predicted. The type of the nonlinear analysis required to be performed depends on the height and complexity of the structure. In mainland China nonlinear dynamic analysis should be performed for the buildings with the height more than 200 m or severe irregularities. Buildings higher than 300 m are required to be analyzed using two or more different computer programs to validate the results. Nonlinear analysis should be properly substantiated with respect to the seismic input, the constitutive model used, the method of interpreting the results of the analysis and the requirements to be met. In the time history analysis, at least two pairs of measured ground motions and one pair of simulated ground motions are utilized as input motions. For the important and large-scale code-exceeding tall buildings, the site-specific design spectra and simulated ground motions are required in the analysis. The earthquake responses, plastic mechanism, distribution of damage, etc., are estimated against the preset allowable limit.

4.2.2 Structural tests

Physical testing is important to structural engineering because it helps establish fundamental understanding about the behavior and failure mechanism of structures. For tall buildings it generally involves both static tests on joints and members and dynamic tests on full scaled structural models for overall assessment since the scale in the structural model test is too small to reflect the local behavior precisely. For joint and member test the challenge is to produce the real stress field and boundary conditions.

Structural model testing is often used to help structural engineers directly acquire the knowledge about the prototype, especially in the case of complex tall buildings for which the numerical simulations are considered unreliable. Shaking table model test has been considered an economical, accurate, and practical way to evaluate the seismic performance of structures. To ensure that the model behaves in a similar manner as the prototype, the model designed should meet the requirements of dynamic similitude theory. By shaking table tests, the earthquake responses and dynamic characteristics are derived, the failure process, yielding mechanism, and structural weak positions are discovered, and then the overall seismic performance of the prototype structure can be evaluated accordingly.

5. Design procedure

The performance-based seismic design procedure consists of two design phases. In the first phase, after the preliminary design is completed with the basic configuration and structural layout selected, the code-exceeding conditions are identified, and the seismic performance objectives are determined accordingly. Furthermore, the key structural components which are crucial to the seismic safety of overall structure should be identified and laid particular emphasis. The design criteria are established to achieve the desired performance objectives. Different performance requirements are proposed for different structural components. The seismic effects under the frequent earthquake and the effects of other actions are determined on the basis of linear-elastic behavior. The dimensions and reinforcement of structural members are derived by using the conventional strength-based design code. The general method for determining the seismic effects is the modal response spectrum analysis using elastic design spectra.

In the second phase, the seismic performance of the target building is evaluated by comprehensive numerical analysis. For tall buildings which greatly exceed the height limit or have very complex or unique as well as innovative structural system without design experience and referential bases, structural testing including joint, member, and full structural model test is highly recommended to conduct to study the structural behavior and check the seismic performance directly. If the pre-defined seismic objectives can not be satisfied, design iteration should be done until satisfied.

6. Case study

6.1 Structure description

The 101-story Shanghai World Financial Center Tower (SHWFC Tower for short later) is 492 m above ground, the tallest completed building in Mainland China. The perspective view of SHWFC Tower is shown in Fig. 1. The structure is diagonally symmetrical, as shown in Fig. 2. Three parallel structural systems, the mega-frame structure consisting of mega-columns, mega-diagonals, and belt trusses; reinforced concrete and braced steel services core tube; and outrigger trusses which create interactions between the core tube and the mega-structure columns, are combined to resist vertical and lateral loads. Perimeter reinforced concrete walls locate at lower levels from Floor 1 to 5, and mega-columns are positioned at the corners of the building from Floor 6. Several stiffened and transfer stories in the structure are regularly spaced throughout the height of the building. One-story high belt-trusses and core transfer trusses are placed at each 12-story interval, whereas three 3-story high outrigger trusses spanning between the mega-structure columns and the corners of the core tube are distributed evenly along the height. The perspective view of the main structure is shown in Fig. 3. The continuity of core tube along the height is broken by three different

Fig. 1 Perspective view of SHWFC Tower

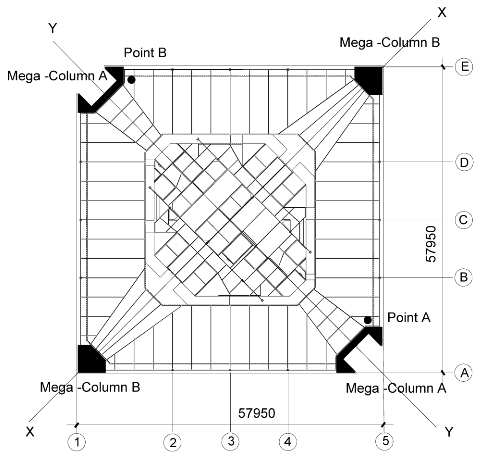


Fig. 2 Standard structural plan

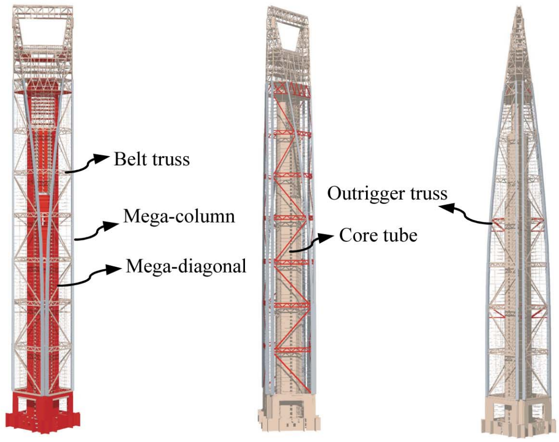


Fig. 3 Perspective view of main structure

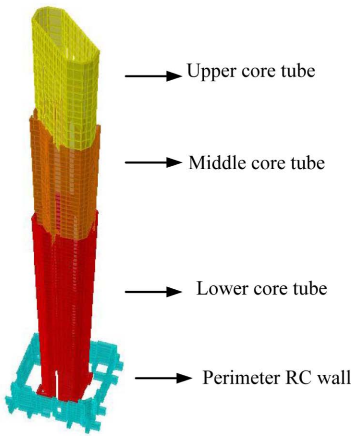


Fig. 4 Shear walls

configurations of core walls due to the requirement of setback in elevation, as shown in Fig. 4. The lower segment (from Floor 1 to 59) and the middle segment (from Floor 60 to 79) are made of reinforced concrete while the upper segment (above Floor 79) is made of structural steel with concrete encasement at the two ends. Strengthened floor diaphragms are provided from Floor 55 to 59 to enable proper shear transfer between the lower and middle core tube while a strengthened floor diaphragm is provided at Floor 79 to transfer shear force between the middle and upper core tube.

The following items are identified as code-exceeding: (1) the total height of 492 m exceeds the limit of 190 m stipulated for composite frame-RC core tube structures; (2) the overall aspect ratio of 8.5 to the top of the spire exceeds the limit of 7; (3) the elevation irregularity including several stiffened and transfer stories, and the discontinuity of core walls exceeds the code limit. The following components are identified as key components of the structure: core tube, mega-columns, mega-diagonals, outrigger trusses, and joints connecting mega-column, mega-diagonal, and belt truss.

6.2 Performance objectives and design criteria

In Shanghai the design earthquake level is specified as Chinese intensity 7 (roughly equivalent to a modified Mercalli intensity of 7). The seismic protection category of SHWFC Tower is II and the performance objectives could be determined according to Table 1. Due to both of the height and irregularity far beyond the code specification, the objective of collapse prevention under the rare earthquake of intensity 8 (extreme rare earthquake) was added. The performance objectives are expressed by four performance levels coupled with four earthquake levels. Namely, under the frequent earthquake of intensity 7, the building is fully operational; under the basic earthquake of intensity 7, the building is operational; under the rare earthquake of intensity 7, the building is repairable; under the rare earthquake of intensity 8, the building is collapse prevention. For the four earthquake ground motion level, the PGA is 35, 100, 200, 360 gal, respectively, and the design acceleration spectra specified in the design code are shown in Fig. 5.

To accomplish the pre-defined performance objectives, the following design criteria were established:

(1) Under the frequent earthquake of intensity 7, all structural components perform elastically. The maximum compressive stress of concrete in the mega-columns and core tube acting as the principal lateral force resisting members is less than 2/3 of its compressive strength. No significant nonlinear responses are believed to occur in compression members when the compressive stress of concrete is less than 2/3 of its compressive strength. The maximum inter-story drift ratio is less than 1/500. The ratio between the natural vibration period of the first torsional mode and that of the first translational mode (period ratio for short later) should not exceed 0.85, and the floor displacement ratio should not exceed 1.4.

(2) Under the basic earthquake of intensity 7, yielding of reinforcement bars in RC members should be prevented, and all the steel members keep elastic. Namely, all the members keep unyielding.

(3) Under the rare earthquake of intensity 7, yielding of mega-columns and secondary components, such as coupling beams in core tube and floor members, is permitted while core tube,

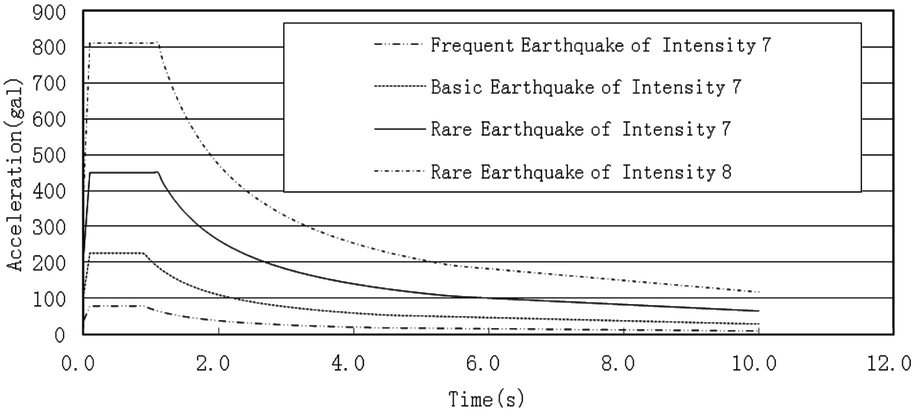


Fig. 5 Design acceleration spectra

mega-diagonals, outrigger trusses, and important joints connecting the primary members, should be prevented from yielding. The maximum inter-story drift ratio is less than 1/100.

(4) Under the rare earthquake of intensity 8, the structure remains standing, and no structural components fall off. The reinforcing bars and steels in the mega-columns, mega-diagonals, core tube, outrigger trusses, and their joints could enter strain hardening stage while fracture should be prevented so that these primary components will not fail. Especially the failure of the aforementioned joints should be avoided.

6.3 Experimental study

6.3.1 Shaking table model tests

Complying with the dynamic similitude law, a 1/50-scale model was designed and constructed by scaling down the geometric and material properties from the prototype structure. Micro-aggregate concrete and fine wires were used to construct the RC elements, and steel structural members were simulated by copper plates. The model structure as shown in Fig. 6 was tested on a shaking table, and was subjected to a series of one and two-dimensional base excitations with gradually increased acceleration amplitude for four earthquake levels, representing frequent, basic, and rare earthquake of Chinese intensity 7, and rare earthquake of intensity 8, respectively (Lu *et al.* 2007). El Centro wave (1940), San Fernando wave (1971), and Shanghai artificial wave were used as input motions. According to the similitude relationship between the tested model and the prototype structure, for individual pair of motions, the peak ground acceleration (PGA) was scaled with the amplification factor of 2.5, and the time was compressed with the scale factor of 1/11.18. Shanghai artificial wave is one-dimensional, and El Centro wave and San Fernando are two-dimensional. For two-dimensional motions, the ratio of PGA between the primary to secondary component was taken as 0.85 according to Chinese seismic design code. After each level of ground motions was inputted, the white noise was scanned to measure the natural frequencies and damping ratios of the model structure. The natural vibration periods of the first three modes at the end of inputting each level of earthquake motions are shown in Table 2. The earthquake responses under each seismic intensity level are outlined as follows:

(1) After the input of the frequent earthquake of intensity 7, no visible cracks were found in the test model. The natural frequency of the model was almost as the same as that measured at initial state, indicating that no damage occurred. The period ratio in two directions is both 0.426, lower than the limit of 0.85. The maximum inter-story drift ratio is 1/539 and 1/707 in two primary directions, lower than the limit of 1/500. The building could be fully operational.

(2) After the input of the basic earthquake of intensity 7, no visible cracks were found in the test model. The natural frequency of the first mode was decreased by 3.7%, indicating that minor damage occurred. The building could be operational.

(3) After the input of the rare earthquake of intensity 7, the test model cracked slightly. The natural frequency of the first mode was decreased by 9.7%, indicating that the damage developed but is still slight. The strains of key components are low. No weak positions related to the code-exceeding conditions were found. The maximum inter-story drift ratio is 1/127 and 1/151 in two directions, lower than the limit of 1/100. The building could be repairable.

(4) After the input of the rare earthquake of intensity 8, major damage occurred. Concrete in mega-columns between Floors 5 and 7 crushed and spalled, and the peripheral steel columns buckled at Floors 6 and 7, as shown in Fig. 7. Cracks were also found in mega-columns at other floor levels. No

Fig. 6 Tested model

(a) Mega-column

(b) Peripheral steel column

Fig. 7 Damage on SHWFC Tower model

joints were damaged and no components fell off. The natural frequency of the first mode was decreased by 27.8%. The test model remained stable. Collapse prevention could be assured.

In addition, from Table 2 it can be found that up to the input of the rare earthquake of intensity 7, the periods of the first two modes are equal. Due to the effect of control and mechanical system of the shaking table, the actual acceleration amplitude of the input motions may differ from the target one. In the tests, under the rare earthquake of intensity 8 this difference in X and Y direction is more significant than other earthquake levels. Therefore, the damage in X and Y direction caused by the earthquake motions of this level is more different, which results in the difference of the lateral stiffness then the natural vibration period in two directions, i.e., the period of the 1st and 2nd mode.

6.3.2 Joint tests

The structural behavior of the joint connecting mega-column, mega-diagonal, and belt truss is crucial to the overall structure. The typical joint between Floors 54 and 55 were tested under static loading. Four 1/7-scale specimens were constructed. Two specimens were made of steel, and the others were made of steel-concrete as shown in Fig. 8. The joint made of steel-concrete was applied in the building. The joint made of steel tested here was just for comparison. The letters denoting the individual component are illustrated in Fig. 9. The letter of “G”, “A” and “F” represents the mega-column at Floor 55, 54 and 53, respectively. “E” and “C” represent mega-diagonals. “D” and “B” represent the lower and upper chord of belt truss respectively. The specimens were tested under monotonic static loading. For the specimens made of steel, components A and F buckled out of plan when the load was increased to be 1.05 times of that induced by the frequent earthquake of intensity 7, as shown in Fig. 10. As to the specimens made of steel-concrete, when the load reached the quantity induced by the basic earthquake of intensity 7, no visible cracks were found in the joint zone. When the load reached the quantity induced by the rare earthquake of intensity 7, some cracks appeared in the joint zone, whereas, the steel in the joint zone remained elastic. When the load was increased to be 1.33 times of that induced by the rare earthquake of intensity 7, the concrete in component F crushed, as shown in Fig. 11, and a few points in the steel embedded in joint zone yielded, but the plastic deformation of the steel is small and the stirrups remained elastic. The buckling of the steel was prevented in this type of joint. The test stopped at this stage due to

Fig. 8 Joint made of steel-concrete

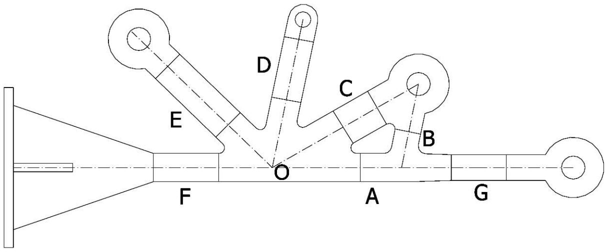


Fig. 9 Numbering of members

Fig. 10 Failure pattern of joint made of steel

Fig. 11 Failure pattern of joint made of steel-concrete

the limitation of loading capacity. The structural behavior of joints made of steel-concrete is testified to be much better than the joints made of steel. All the responses of the specimens made of steel-concrete met the requirements of the design criteria pre-established for the joint. The joint made of steel-concrete is adopted in the design.

The results of shaking table tests on the full scaled model structure and the static loading tests on the key joint demonstrate that the global as well as local responses under individual intensity level of earthquakes meet the pre-established design criteria, which indicates the desired seismic performance objectives could be realized.

6.4 Numerical analysis

6.4.1 Elastic finite element analysis

The commercial program ANSYS was adopted in the elastic analysis under the frequent earthquake of intensity 7. Considering the contributions from different structural components to the global structural behavior, different types of computational elements were employed. Solid element was employed for mega-columns below Floor 41. Ordinary beam element was applied for mega-diagonals, belt trusses and outrigger trusses. As to floor slabs and core walls, shell element was used. The finite element analysis model (FEAM) is shown in Figs. 12 to 14. The structural dynamic characteristics and earthquake responses were obtained as follows:

(1) The first two modes are translational, corresponding to the periods of 6.406 s and 5.401 s in two primary directions. The third mode is torsional with the period of 2.772 s. The period ratios in two directions are 0.433 and 0.513 respectively, less than the limit of 0.85.

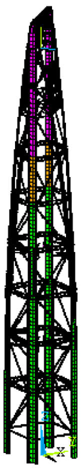


Fig. 12 FEAM of Mega-frame

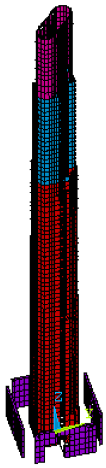


Fig. 13 FEAM of core wall

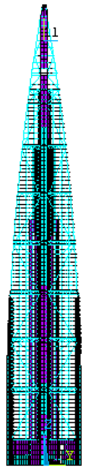


Fig. 14 Integrated model

(2) The maximum inter-story drift ratios in two directions are 1/1037 and 1/1067 respectively, less than the limit of 1/500.

(3) Below Floor 20, the floor displacement ratio is approximately equal to 1.0. From Floor 20 to 90, the displacement ratio is less than 1.2. Above Floor 90, the displacement ratio is less than 1.4.

The tested joint model was also analyzed by finite element analysis. The predicted maximum stress in the joint is 28 MPa under the frequent earthquake of intensity 7. Accordingly, the maximum stress in the joint is estimated as 68 MPa and 305 MPa under the rare earthquake of intensity 7 and 8 respectively, less than the yielding stress of 345 MPa. The buckling analysis results indicate that buckling will not occur in the joint under the rare earthquake of intensity 7 while the joint will just reach buckling state under the rare earthquake of intensity 8.

6.4.2 Nonlinear time history analysis

1. Shaking table test model

Due to the complex behavior of this tower under earthquakes, another different three-dimensional nonlinear structural analysis program TBNLDA developed by the authors was applied in the nonlinear time history analysis. The simplified structure, where only the three parallel structural systems, the mega-frame structure, the services core tube, and the outrigger trusses were kept, was modeled using macro elements so that nonlinear time history analysis could be performed efficiently with accuracy appropriate to engineering application. The shear walls were modeled by macro wall element which consists of vertical fibers simulating the bending and axial loading behavior and shear spring simulating the shear behavior. Other structural members were modeled by beam element whose cross-section is simulated by the fiber model. The interaction between the axial and flexural behavior can be considered in the numerical model. In the simplified model the structure was divided into 14 stories in vertical direction. For the sake of validating the simplified model, the shaking table test model was firstly analyzed.

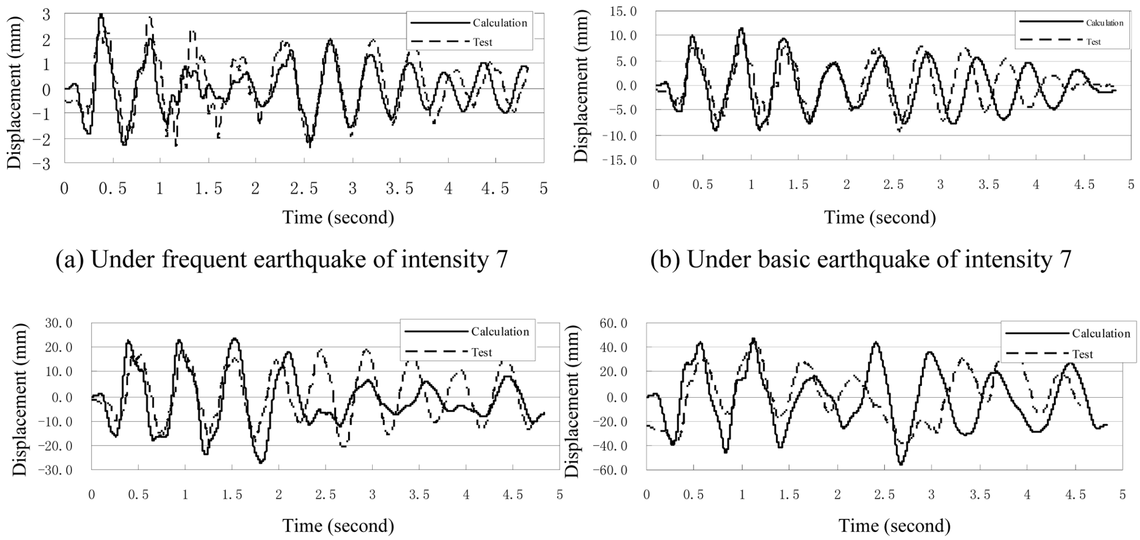
At the end of inputting each level of earthquake motions, the natural vibration periods of the test model were calculated. The comparison of calculated natural vibration periods of the first three modes with the test results at different stage are shown in Table 2. The calculated ratios of the base

Table 2 Comparison of natural vibration periods for shaking table test model (unit: second)

Mode	Initial state		Frequent 7		Basic 7		Rare 7		Rare 8	
	Calculate	Test	Calculate	Test	Calculate	Test	Calculate	Test	Calculate	Test
1	0.4476	0.4437	0.4962	0.4437	0.5768	0.4608	0.6714	0.4914	0.9173	0.6143
2	0.4009	0.4437	0.3691	0.4437	0.5578	0.4608	0.6394	0.4914	0.8509	0.5672
3	0.1775	0.1890	0.1622	0.1890	0.2108	0.2001	0.2261	0.2391	0.2538	0.2600

Table 3 Comparison of the ratio between base shear and gravity load

Earthquake level	Frequent 7		Basic 7		Rare 7		Rare 8	
	X direction	*Y* direction	*X* direction	*Y* direction	*X* direction	*Y* direction	*X* direction	*Y* direction
Calculate	5.11%	6.08%	15.02%	17.30%	25.38%	30.86%	44.93%	46.28%
Test	6.60%	8.56%	16.01%	19.28%	26.63%	26.92%	48.44%	39.45%



(a) Under frequent earthquake of intensity 7 (b) Under basic earthquake of intensity 7

(c) Under rare earthquake of intensity 7 (d) Under rare earthquake of intensity 8

Fig. 15 Roof displacement in the *Y* direction subjected to El Centro wave

shear to gravity load are compared with the test results, as shown in Table 3. The comparison of the calculated displacement responses in the y direction at the roof with the test results at different stage subjected to El Centro wave are shown in Fig. 15. The calculated results agree with the test results roughly. The difference becomes larger with the increase of ground motions.

2. Prototype structure

The simplified analytical model proved to be valid above was then applied for the nonlinear time history analysis of the prototype structure. The main earthquake responses at different earthquake level were obtained as follows:

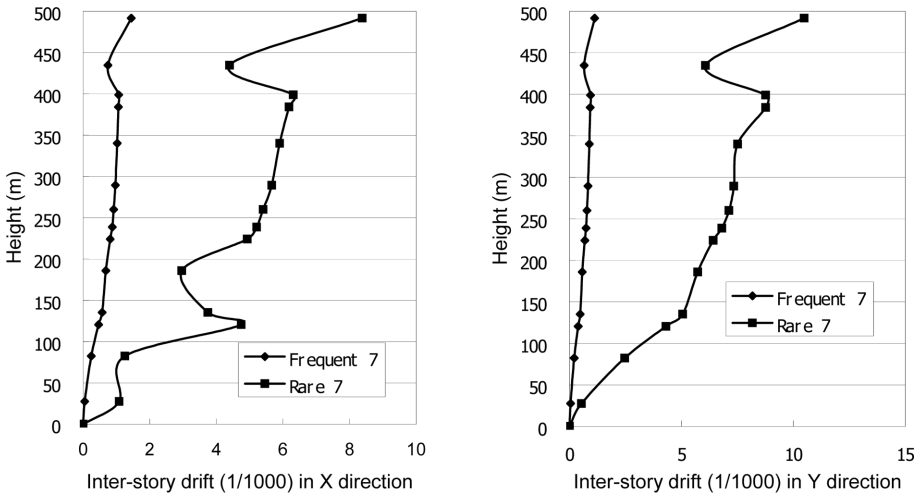
(1) After the input of the frequent earthquake of intensity 7, micro concrete cracks on mega-columns at the levels of Floors 5 and 6 occur first. The maximum compressive stress of concrete in mega-columns and core tube is 13 MPa and 10 MPa respectively, less than 2/3 of concrete compressive strength. The maximum stress of the reinforcing bars or steels is 190 MPa, 180 MPa and 80 MPa in mega-columns, core tube and mega-diagonals respectively, less than the steel yielding strength. The distribution of inter-story drift ratio is shown in Fig. 16. The maximum inter-story drift ratio is 1/934 and 1/1066 in two primary directions, less than the limit of 1/500.

(2) After the input of the basic earthquake of intensity 7, micro concrete cracks occur on the mega-columns at other levels. No structural members yield.

(3) After the input of the rare earthquake of intensity 7, cracks develop in many RC structural members extensively. The stress of the reinforcing bars and steel embedded in mega-columns from Floor 1 to 5 are still low, whereas, in the mega-columns at Floor 6 some steels embedded yield and the maximum concrete stress arrives at the ultimate strength. The reinforcing bars and steels in the core tube, mega-diagonals, and outrigger trusses have not yet yielded. The distribution of inter-story drift ratio is shown in Fig. 16. The maximum inter-story drift ratio is 1/158 and 1/115 in two directions, less than the limit of 1/100.

(4) After the input of the rare earthquake of intensity 8, the strains in many structural components increase rapidly. The maximum stress of the reinforcing bars is 290 MPa and 350 MPa at the bottom of mega-columns and core tube, still lower than the steel yielding strength. In the mega-columns at Floor 6 the reinforcing bars yield, the embedded steel has been in hardening stage with the maximum stress of 410 MPa, and the cover concrete spalls with the maximum strain of 0.009. In the middle level of the structure, the steels in mega-diagonals have also yielded. The strains of the longitudinal bars and steels in all the above members are lower than the fracture strain.

The numerical analysis results also show that all the responses of the structure as well as the main structural members and important joints under the four design earthquake levels comply with the design criteria, ensuring the implementation of performance objectives.



(a) *X* direction (b) *Y* direction

Fig. 16 Distribution of inter-story drift ratio of prototype structure

7. Conclusions

A general performance-based seismic analysis and design approach for code-exceeding tall buildings in Mainland China is proposed in this study. It is an indirect PBSD, the conventional strength-based design with upgrades by selection of performance objectives, establishment of performance criteria, and comprehensive validation. At the current state of arts, it is more practical as well as feasible than the direct PBSD in engineering application.

A typical example of PBSD for an ultra-tall building, Shanghai World Financial Center Tower, is presented. The seismic performance objectives of SHWFC Tower are taken as fully operational under minor earthquakes, operational under moderate earthquakes, repairable under rare earthquakes, and collapse prevention under extreme rare earthquakes. The design criteria to accomplish the performance objectives, different from those specified in the current design code for ordinary buildings, were proposed, laying particular emphasis on key structural components. Experimental study including shaking table tests on full scaled structural model and static loading tests on key joints was carried out to evaluate the seismic performance and validate the design comprehensively. Both the global responses of full structural model and the local responses of joints under individual intensity level of earthquake meet the design criteria. Furthermore, detailed numerical analysis including elaborate elastic finite element analysis by ANSYS and nonlinear time history analysis for the simplified structure by TBNLDA were performed to obtain a complete picture of the structural behavior of the tower suffered by the earthquake of different intensity. The simulation results of the model structure agree roughly with the shaking table test results, validating the nonlinear numerical analysis. Both of the experimental study and numerical analysis demonstrate that the designed structure system is an efficient solution to resist earthquakes and the pre-selected seismic performance objectives could be accomplished with sufficient confidence.

Acknowledgements

The authors are grateful to the support from the Ministry of Science and Technology of China through Grant No. SLDRCE09-B-10 and the National Natural Science Foundation of China through Grant No. 51078272 and 51021140006, the owner of Shanghai World Financial Center Tower, Leslie E. Robertson Associates, and Shanghai Xian Dai Architectural Design (Group) Co., Ltd.

References

Applied Technology Council (1996), *Seismic Evaluation and Retrofit of Concrete Buildings*, ATC-40, Report No. SSC 96-01, Redwood City, California.

Arzoumanidis, S., Shama, A. and Ostadan, F. (2005), "Performance-based seismic analysis and design of suspension bridges", *Earthq. Eng. Struct. D.*, **34**(4), 349-367.

Bommer, J.J. and Pinho, R. (2006), "Adapting earthquake actions in Eurocode 8 for performance-based seismic design", *Earthq. Eng. Struct. D.*, **35**(1), 39-55.

Chandler, A.M. and Lam, N.T.K. (2001), "Performance-based design in earthquake engineering: a multi-disciplinary review", *Eng. Struct.*, **23**(12), 1525-1543.

Federal Emergency Management Agency (2000), *Prestandard and Commentary for the Seismic Rehabilitation of Buildings*, FEMA-356, Washington, DC.

Lu, X.L., Zou, Y., Lu, W.S. and Zhao, B. (2007), "Shaking table model test on Shanghai World Financial Center Tower", *Earthq. Eng. Struct. D.*, **36**(4), 439-457.

Los Angeles Tall Building Structural Design Council (2008), *An Alternative Procedure for Seismic Analysis and Design of Tall Buildings Located in the Los Angeles Region*, Los Angeles.

Ministry of Construction of China (2010), *Code for Seismic Design of Buildings*, Beijing, China Architecture & Building Press. (in Chinese)

Ministry of Construction of China (2002), *Technical Specification for Concrete Structures of Tall Buildings*, Beijing, China Architecture & Building Press. (in Chinese)

Moehle, J.P. (2004), "A frame work for performance-based earthquake engineering", *Proceedings of 1st International Conference on Urban Earthquake Engineering*, Tokyo, March.

Moehle, J.P. (2008), "Performance-based seismic design of tall buildings in the US", *Proceedings of 14th World Conference on Earthquake Engineering,* Beijing, October.

Priestley, M.J.N. and Kowalsky, M.J. (2000), "Direct displacement-based seismic design of concrete buildings", *Bulletin, NZNSEE*, **33**(4), 421-444.

Scotta, R., Tesser, L., Vitaliani, R. and Saetta, A. (2009), "Global damage indexes for the seismic performance assessment of RC structures", *Earthq. Eng. Struct. D.*, **38**(8), 1027-1049.

Structural Engineers Association of California (1995), *Performance Based Seismic Engineering of Buildings*, Vision 2000, Sacramento, California.

Structural Engineers Association of Northern California (2007), *Recommended Administrative Bulletin for San Francisco*, San Francisco.

Vamvatsikos, D. and Fragiadakis, M. (2010), "Incremental dynamic analysis for estimating seismic performance sensitivity and uncertainty", *Earthq. Eng. Struct. D.*, **39**(2), 141-163.

Xue, Q., Wu, C.W., Chen, C.C. and Chen, K.C. (2008), "The draft code for performance-based seismic design of buildings in Taiwan", *Eng. Struct.*, **30**(6), 1535-1547.

Zou1, X.K., Chan, C.M., Li, G. and Wang, Q. (2008), "Multiobjective optimization for performance-based design of reinforced concrete frames", *J. Struct. Eng.*, **134**(9), 1569-1581.