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A Comparative Case Study on Seismic Design of Tall RC Frame-Core Tube Structures in China and USA

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SUMMARY

To evaluate the major differences between the Chinese and the United States (US) seismic design codes from a structural system viewpoint, a comparative case study is conducted on a tall frame-core tube building, a typical type of reinforced concrete (RC) system widely constructed in both countries. The building, originally designed using the US seismic design code, is firstly re-designed according to the Chinese seismic design code based on the information provided by the Pacific Earthquake Engineering Research Center (PEER). Secondly, the member dimensions, the dynamic characteristics, the seismic design forces and the material consumptions of the two designs are compared in some detail. Subsequently, nonlinear finite element models of both designs are established to evaluate their seismic performances under different earthquake intensities. Results indicate that the seismic design forces determined by the Chinese response spectrum are larger than those determined by the US spectrum at the same seismic hazard level. In addition, the upper-bound restriction for the inter-story drift ratio is more rigorously specified by the Chinese code. These two aspects have led to a higher level of material consumption for a structure designed by the Chinese code. Despite of the above, the two designs yield roughly similar structural performances under earthquakes.

KEY WORDS**:** RC frame-core tube building; American code; Chinese code; seismic design; nonlinear analysis; design comparison

1. Introduction

Tall building constructions have become increasingly popular in China over the recent two decades. However, seismic safety of these tall buildings presents a critically important issue because China is an earthquake-prone country being located at the intersection of the Pacific and Eurasian seismic belts. Although considerable progress has been made to the major seismic design codes in China, viz. the latest Code for the Seismic Design of Buildings GB50011-2010 (CMC, 2010a) and the Technical Specification for Concrete Structures of Tall Building JGJ3-2010 (CMC, 2010b), further improvement of the design philosophies is a challenging task. This is because none of the tall buildings in China has experienced a very strong earthquake. As such, limited structural information is available due to the lack of exposure of these structures to strong earthquakes. Therefore, it is important to study through the efforts made in other countries with substantial experience in effective seismic design of tall buildings.

The United States (US), Japan and Europe have a long history of tall building construction and as a result have developed characteristic and comprehensive seismic design philosophies. Their tall buildings have proven to exhibit good seismic performances during strong earthquakes. Various comparisons have been performed between the seismic design codes of the US, Japan, Europe and China. These include comparing the site classifications and the lateral earthquake loads in different codes (Luo and Wang, 2004, 2006; Duan and Hueste, 2012; Song and Zheng, 2012; Zhao and Jiang, 2012); comparing the reinforcement design, detailing and ductility of concrete members in earthquake-resistant structures (Zhuang and Li, 2006; Sun et al., 2011; Bai and Au, 2013a, b); as well as comparing the deflection limits of tall buildings (Smith, 2011), the load combinations (Guan, 2012) and the near-fault effect factors (Zhou and Fang, 2012). In addition, the seismic design codes were also compared between China and other countries, such as New Zealand (Dong, 2011) and Canada (Zhang and Christopoulos, 2011). Despite of the above research efforts, most of these comparative studies mainly focused on several design parameters, a design formula or a particular phase within the entire seismic design procedure. Such are insufficient to fully evaluate the design philosophies and the safety margins of different design codes, because the seismic performance of a structure is governed by the entire system of seismic design codes. Therefore, an effective research methodology should involve selecting a building with a specified seismic design objective, designing it based on different code systems and then comparing the performances of the different designs. In this regard, only limited studies (Tang et al., 2013) have been conducted to date.

In view of the above, this study aims to conduct a case study to comprehensively compare the design outcomes resulted from the entire systems of the Chinese and the US seismic design codes. Subsequently, the major differences between the two outcomes are identified and discussed. A typical reinforced concrete (RC) frame-core tube tall building, which is a widely used structural form in both China and the US, is selected to conduct the case study. The building, originally designed using the US seismic design code, is firstly re-designed according to the Chinese seismic design code based on the information provided by the Pacific Earthquake Engineering Research Center (PEER) (Moehle et al., 2011). Secondly, the member dimensions, the dynamic characteristics, the seismic design forces and the material consumptions of the two designs are compared in some detail. Subsequently, nonlinear finite element models of both designs are established to evaluate their seismic performances under different earthquake intensities.

2. Background Information for the Case Study

To evaluate and improve performance-based seismic designs of tall buildings, PEER launched the Tall Buildings Initiative (TBI) research program in 2006. A case study project on tall buildings (Moehle et al., 2011) was conducted as part of the TBI program. One of such buildings is an RC frame-core tube structure, Building 2A, which can be served as a representative benchmark.

Located in Los Angeles, Building 2A is a 42-story residential building including a 6.1-m tall penthouse on the top and four stories below the ground. The total height of the building is 141.8 m above the ground. Figure 1a reproduces the three-dimensional (3D) view and the typical floor plan of Building 2A as presented in the published report of Moehle et al. (2011). In this report, Building 2A was designed based on the International Building Code (IBC) (ICC, 2006), which requires the use of ASCE 7-05 (ASCE, 2005) and ACI 318-08 (ACI, 2008). Table 1 summarizes the seismic design parameters used for Building 2A.

To compare the differences in the seismic performances of the same building based on the Chinese and the US seismic design codes, Building 2A is re-designed herein to the Chinese codes, mainly including GB50011-2010 (CMC, 2010a), JGJ3-2010 (CMC, 2010b) and the Code for Design of Concrete Structures GB50010-2010 (CMC, 2010c). The Chinese PKPM design software (CABR, 2010) is employed and the re-designed building is referred to as Building 2N, as detailed in Figure 1b. All design details for Building 2N, including the structural configuration and dimensions, the vertical design loads, the site conditions and the seismic hazard level are identical to those for Building 2A. Note that the detailed design information of the basement of Building 2A is not given in the published report of Moehle et al. (2011), which restricts the design of the basement of Building 2N. Preliminary analysis of the basement in Building 2A indicates that the area and stiffness of the basement are large enough to satisfy the fixed boundary requirements in Section 6.1.14 in the Chinese code GB50011-2010 (CMC, 2010a). As such, the building can be modeled as fixed at the top of the basement for the superstructure design of Building 2N. In addition, this study focuses on the differences in the design results and seismic performances of superstructures of the two buildings. Thus, the effect of the basement is not taken into account in the re-design, implying that the structure is fixed at the ground level, also not included in the seismic evaluation for both Buildings 2A and 2N. In addition, Building 2A was originally designed to have a post-tensioned flat slab system. Though this flat slab system has been widely used in US, its application in Chinese is not so much. The slab-column connections are prone to brittle punching failure under gravity and/or earthquake loads without enough warning. Prior to punching failure, the lateral drift capacities of the connections are also limited (Chen, 2003; Khaleel et al., 2013; Rha et al., 2014; Ruiz et al., 2013; Yi et al., 2014). Such a failure mode can hardly produce an overall ductile yield mechanism in the structure. Many flat slab collapse scenarios have been reported in the literature (Chen, 2003; Ruiz et al., 2013). In China, very few flat slab buildings have suffered real strong earthquakes. Thus, the application of flat slab system is strictly restricted in high seismic intensity regions as specified in the Chinese code (no more than 40 m in height in zones of 8.5 degree seismic intensity). Alternatively, additional beams are provided to connect its concrete core tube to the perimeter frame columns for Building 2N. Taking this into consideration, the thickness of the floor slabs in Building 2N is thus reduced to 140 mm accordingly.

3. Vertical Design Load

To maintain consistency of the design conditions, identical superimposed dead loads and live loads, as listed in Table 2, are considered for both buildings, except for the self-weight of the structure. Whilst Building 2A adopts the strength design load combinations given in ASCE 7-05 (ASCE, 2005), load combinations for Building 2N follow the provisions of 5.6.1 and 5.6.3 in JGJ3-2010 (CMC, 2010b). Guan (2012) compared the load combinations between ASCE 7-05 and the Chinese code. The comparison indicates that similar load combination concept is adopted by the Chinese and the US codes which results in similar overall structural effects, although the specific load combination coefficients are slightly different.

4. Seismic Design Load

4.1 Chinese and US seismic design methods

The seismic design methods adopted in the Chinese and the US codes are different in some aspects. In the Chinese code, the fortification level earthquake (i.e., 10% probability of exceedance in 50 years) is used to define the Seismic Ground Motion Parameter Zonation Map of China. A two-stage design method is used for the structural seismic design of buildings. The first design stage refers to an elastic design procedure under frequent earthquakes (i.e., 63% probability of exceedance in 50 years). For all buildings, this stage of design is required. In this stage, the design seismic forces are calculated using the acceleration spectrum at the level of frequent earthquakes, and the corresponding load carrying capacity and elastic deformation are evaluated. For some special buildings, such as structures with irregular plane or with obvious weak stories, the second stage is required. The second stage refers to an inelastic deformation check procedure under severe earthquakes (i.e., 2~3% probability of exceedance in 50 years), where the seismic inelastic deformation needs to be assessed to prevent serious damage or collapse.

The seismic design method employed in the US IBC 2006 (ICC, 2006) represents an inelastic design procedure under the design earthquake, which means that a structure can be economically designed according to the reduced elastic seismic design forces, while the structural elements are detailed to reliably exhibit ductile behavior thereby maintaining the basic life safety performance objective. IBC 2006 (ICC, 2006) utilizes Maximum Considered Earthquake (MCE, 2% probability of exceedance in 50 years) ground motion maps to define the earthquake intensity in different regions in the conterminous United States. The design procedure is as follows: the MCE spectrum is firstly calculated according to the mapped acceleration parameters and site coefficients, and the corresponding design spectrum is 2/3 times the MCE spectrum. The design spectrum is then reduced by the response modification coefficient, R, to calculate the seismic design lateral force or base shear, which is used in the subsequent elastic structural analysis. The internal forces in structural components can be obtained from the elastic analysis. The design lateral force-induced drift from the elastic analysis should thus be multiplied by a deflection amplification factor, C_d, to estimate the maximum inelastic drift.

As the above description, the seismic design philosophies adopted in the Chinese and the US codes are indeed different. The seismic design method employed in the US code is an inelastic design procedure. The seismic design load is obtained from the 2/3 MCE spectrum and the anticipated damages in response to the design level earthquakes are acceptable in structural components. On the other hand, the seismic design strategy adopted by the Chinese code is an elastic design procedure and the seismic design forces are calculated from the frequent earthquake spectrum. No damage should occur in buildings at the design level. In addition, the Chinese code also specifies that for some special buildings, the secondary deformation assessment should be conducted at the MCE level to ensure the structural safety.

4.2 Seismic design load

This study focuses on the differences in seismic performances between the two buildings respectively designed according to the Chinese and the US codes, so it is important to ensure consistency of the site classification and the seismic hazard level between Buildings 2A and 2N.

Luo and Wang (2006) have conducted a comprehensive comparison on the site classification and seismic hazard characteristics between these two countries and suggested the conversion relationships of the site classification and ground motion parameters between the Chinese and the US codes. Building 2A is located on an NEHRP site class C, with an equivalent shear-wave velocity of 360 m/s for 30 m soil (V_{S30}). The characteristic period of the site is 0.455 s. This site condition is approximately equal to Site Class II and the 3rd Group in GB50011-2010 (CMC, 2010a) according to the findings of Luo and Wang (2006).

As there are several differences between the calculation methods for seismic design loads in the Chinese and US codes, a key challenge in this study is to determine a proper earthquake intensity for the seismic design of Building 2N using the Chinese code, and achieve an identical seismic hazard level between Buildings 2N and 2A. Note that the exceedance probability of MCE as defined in the US design code is approximately equivalent to that of a severe earthquake as defined in the Chinese design code. The response spectra for a severe earthquake in an 8.5 degree and a 9 degree seismic intensity zones in China are plotted in Figure 2 against the site-specific MCE spectrum (Moehle et al., 2011), which was used for the design of Building 2A. Notably, the corresponding peak ground acceleration (PGA) values of the fortification level earthquake (i.e., 10% probability of exceedance in 50 years) are 300 cm/s^2 and 400 cm/s^2 in the 8.5 degree and 9 degree seismic intensity zones, respectively.

Figure 2 indicates a reasonable agreement between the two Chinese response spectra and the site-specific MCE spectrum. For short periods, the response spectrum for the 9 degree seismic intensity zone clearly agrees better with the MCE spectrum; for moderate periods (approximately 2.5 s), the spectrum for the 8.5 degree zone agrees better; and for long periods (beyond 2.5 s), the values of both Chinese response spectra are greater than that of the MCE spectrum.

In view of the above, the 8.5 degree seismic intensity specified in the Chinese seismic design code is selected as the design intensity for Building 2N for the following reasons: (1) As specified in the Chinese code JGJ3-2010 (CMC, 2010b), the height for a RC frame-core tube structure, such as Building 2N, is strictly limited to no more than 60 m in zones of 9 degree seismic intensity. As such, a 9 degree seismic intensity is not suitable for the design intensity requirement of Building 2N. (2) The estimated fundamental period of Building 2N is approximately 2.52 to 5.04 s, based on the empirical formula for RC frame-core tube structures in China. For such a long period range, the response spectrum in the 8.5 degree seismic intensity zone is closer to the site-specific MCE spectrum, as evident in Figure 2.

5. Comparison of the Design Outcomes

5.1 Effective seismic weight and design periods

The effective seismic weight and the design periods of the two buildings are compared in Table 3. The seismic weight of Building 2N is the sum of the self-weight of the structure plus 0.5 times the live load, in accordance with the provisions of 5.1.3 of the Code for Seismic Design of Buildings GB50011-2010 (CMC, 2010a). The effective seismic weight of Building 2A includes the total dead load and four other loads required by Section 12.7.2 in ASCE 7-05 (ASCE, 2005), viz. (1) in areas used for storage, a minimum of 25 percent of the floor live load; (2) the weight of partitions; (3) the total operating weight of permanent equipment; and (4) where the flat roof snow load exceeds 1.44 kN/m^2, 20 percent of the uniform design snow load, irrespective of the actual roof slope.

Table 3 shows that the design periods of Building 2A, which are provided by the published report of Moehle et al. (2011), are much larger than those of Building 2N. It should be noted that the two buildings are different in structural arrangement and member dimension. In addition, there are another two reasons for the larger design period of Building 2A.

(1) The Chinese code adopt an elastic design procedure under frequent earthquakes and the design periods of Building 2N are calculated using the gross elastic section stiffness provided by the PKPM software. IBC 2006 (ICC, 2006), on the other hand, adopts an inelastic design procedure under the design earthquake in which effective component stiffness values (e.g., $0.7EI_g$ for columns and $0.35EI_g$ for beams) are used when developing the analysis model for design, with consideration of the anticipated cracking and damage. The published report of Moehle et al. (2011) provides the stiffness assumptions used in the design of Building 2A.

(2) The basement is included in the analysis model of Building 2A (Moehle et al., 2011), which also lengthened the design periods.

Regardless of the above two reasons, if the periods of Building 2A are calculated using the same method for Building 2N without including the basement, the elastic fundamental period of Building 2A of approximately 2.9 s is still longer that of Building 2N.

5.2 Material properties and dimensions of the main structural members

The material properties and dimensions of the main structural members in Buildings 2A and 2N are compared in Table 4. More detailed design information for Building 2N is listed in Table 5. Building 2N evidently contains larger columns and more internal walls in the core tube than Building 2A. Such a difference is mainly due to the fact that the seismic design forces determined by the Chinese response spectrum are larger than those governed by the US spectrum at the same seismic hazard level. In addition, the Chinese code specifies a higher requirement for inter-story drift ratio, which leads to a higher structural stiffness and hence larger seismic design forces. A more detailed discussion will be presented in Section 6.

5.3 Design lateral forces and inter-story drift ratio

The design seismic forces of Building 2N are calculated with the acceleration spectrum for frequent earthquakes. Thus, the load carrying capacity and the elastic deformation are evaluated based on these corresponding seismic forces. The design seismic forces of Building 2A, on the other hand, are calculated with a reduced design acceleration spectrum according to the response modification coefficient, R. The internal forces in structural components can subsequently be calculated from an elastic analysis, and the drift corresponding to the design lateral forces can be obtained by multiplying by C_d.

The seismic design forces along the building height of Buildings 2A (Moehle et al., 2011) and 2N are displayed in Figure 3a (in which the response modification coefficient, R, is already considered). Obviously, the seismic base shear force of Building 2N in the Y direction is 1.47 times that of Building 2A. The seismic response coefficients in the Chinese and US design codes are shown in Figure 3b. The design information for Building 2A (Moehle et al., 2011) indicates that the combined response for the modal base shear determined directly via modal response spectrum analysis, V_t, is smaller than 85 % of the calculated base shear, V, using the equivalent lateral force (ELF) procedure. However, Section 12.9.4 in ASCE 7-05 (ASCE, 2005) clearly specifies that the design modal base shear force shall be scaled with 0.85 V/V_t if the modal base shear force, V_t, is less than 0.85 V. Thus, the design base shear of Building 2A is governed by 0.85 V. The seismic response coefficient (Eq. 12.8-2 in ASCE 7-05), as well as the upper limit (Eq. 12.8-3 in ASCE 7-05) and lower limit (Eq. 12.8-5 in ASCE 7-05) used in the ELF procedure of Building 2A, are all shown in Figure 3b. The base shear force of Building 2A determined by the ELF procedure, V, is constrained by the lower limit of Eq. 12.8-5 in ASCE 7-05 shown as the red dash-dot line in Figure 3b, whereas the equivalent seismic response coefficient to the 0.85 V is shown as the blue dash line. Thus, the comparison made in Figure 3b indicates that the seismic response coefficient in the Chinese design code (i.e., the black line) is evidently larger than the value in the US code (i.e., 0.85 Eq. 12.8-5 in ASCE 7-05, the blue dash line). In addition, the effective seismic weight of Building 2N is also larger than that of Building 2A. These two reasons lead to the seismic design forces of Building 2N being noticeably larger than that of Building 2A.

The design inter-story drift ratio in each direction of the two buildings and the corresponding upper limits are shown in Figure 4. The maximum story drift ratio of Building 2N for frequent earthquakes is approximately 1/809, which marginally satisfies the allowable limit of 1/800 for the elastic inter-story drift ratio specified in the Chinese code. On the other hand, the maximum story drift ratio of Building 2A at the design level is approximately 1/152, which is much smaller than the allowable limit of 1/50 for the inelastic inter-story drift ratio specified in ASCE 7-05 (ASCE, 2005). Therefore, the story drift limit for frequent earthquakes specified in the Chinese code plays an important role in the seismic design of Building 2N. However, the design of Building 2A is not governed by the story drift limit specified by the building code.

5.4 Material consumptions

The material consumptions of the two buildings are compared in Figure 5. The comparison reveals that the total concrete consumption of Building 2N is roughly the same as that of Building 2A. However, such a consumption of the main lateral-force-resistance system, including the beams, columns and shear walls of Building 2N, is substantially higher than that of Building 2A. Similarly, the amount of reinforcement used in Building 2N is clearly higher than that in Building 2A, and the additional reinforcement is mainly distributed in the shear walls. Note that the design shear forces of Building 2N are larger than that of Building 2A, which contributes to the higher reinforcement usage in the shear walls of Building 2N.

6. Critical Factors in Seismic Design of Building 2N

Several critical factors have influenced the seismic design of Building 2N using the Chinese code. They include the seismic design force, the inter-story drift limit, the shear capacity design of the shear walls and coupling beams, and the axial compression ratio limit. If Building 2N adopts the same member dimensions and materials as Building 2A, its inter-story drift ratio in the X direction would be approximately 1/750 for frequent earthquakes, which does not satisfy the limit of 1/800 specified in the Chinese seismic design code. Moreover, its load-bearing capacities of the lateral-force-resisting components would not satisfy the strength requirements of the Chinese code. For example, the axial compression ratio of the columns located at the bottom 10 stories would exceed the allowable limit, with a maximum axial compression ratio reaching 0.89; furthermore, the shear capacities of many of the coupling beams and some of the shear walls would also be inadequate to satisfy the Chinese code. Therefore, for Building 2N to satisfy the allowable limit of the inter-story drift ratio and the axial compression ratio specified in the Chinese code, two measures must be adopted to increase the lateral stiffness of Building 2N, by enlarging the cross-sections of the columns and adding several shear walls inside the core tube. Nonetheless, many coupling beams and some shear walls are still unable to meet the demand of the maximum design shear force of their cross sections. To rectify this situation, the thicknesses of the shear walls must be increased and the spans of the coupling beams enlarged to ensure the shear capacity design of the core tube meeting the strength requirements of the Chinese code.

7. Nonlinear Finite Element Analysis of Buildings 2A and 2N

Based on the design outcomes of Buildings 2A and 2N, 3D nonlinear finite element models of the two buildings are established using the commercial software MSC.Marc, which has powerful nonlinear computational capacity. Because the inelastic responses including the flexural yield of the frame beams and columns, the shear behavior and flexural yield of shear walls and coupling beams were considered in the 3D nonlinear analytical model for the seismic evaluation of Building 2A in the TBI report (Moehle et al., 2011), the same modeling strategy is also adopted in the seismic evaluation of Buildings 2A and 2N in this study. The frame beams and columns are modeled with the fiber beam element model, which is capable of simulating the axial-flexural coupling of RC frames. The core tube and coupling beams are simulated using the multi-layer shell element model, which exhibit superior nonlinear performance when replicating the bending and shear coupling behaviors both in-plane and out-of-plane. Truss elements are adopted to simulate the longitudinal reinforcement in the boundary elements of core walls and the longitudinal or diagonal reinforcement in the coupling beams. The details of the fiber beam element model and the multi-layer shell element model have been introduced in the published papers (Miao et al., 2011; Lu et al., 2013). The former works of the authors (Miao et al., 2011; Li et al., 2011; Lu et al., 2013) have validated the feasibility and accuracy of the fiber beam element model and the multi-layer shell element model used in this study at the component levels by comparing the experimental and simulated results of a number of specimens test, including RC columns, shear walls and core tubs. The degradation of strength and stiffness resulting from the cyclic loading and the performance of structural components at collapse or near-collapse levels are accurately represented. In addition, the accuracy of this modeling approach has been confirmed at the structural level through a comparison of a shaking table test and the corresponding FE analysis (Jiang et al., 2014). Furthermore, Lu et al. (2012) conducted the collapse simulations of the typical RC frames in Xuankou School during the Wenchuan Earthquake and the simulation results agreed well with the actual seismic damage. These works confirm that the proposed model is reliable to predict the nonlinear behavior of the buildings even at collapse or near collapse level. The post-tensioned slabs in Building 2A are modeled using the equivalent beams (Yang et al., 2010). The expected material properties previously defined in Tables 4 and 5 are used to develop the nonlinear finite element models of the two structures.

The inherent structural eccentricities resulting from the distribution of mass and stiffness can be directly reflected by the two 3D structural models. The accidental eccentricities resulting from some uncertain factors, such as variation in material strength, tenant build-out, furniture, and storage loads, are very complex. LATBSDC (2008) indicates that, if the torsional amplification factor A_x during the serviceability evaluation as described in ASCE 7-05 is less than 1.5, the accidental eccentricities can be ignored during the collapse prevention analysis. The published report (Moehle et al., 2011) shows that the building studied is regular in plan and elevation and the factor A_x is less than 1.5. Hence, the accidental eccentricities are not considered in these 3D nonlinear models.

7.1 Pushover analysis

The provisions of Section 3.11.4 in the Chinese code JGJ3-2010 (CMCb, 2010) specify that static nonlinear procedures (pushover) can be adopted for the preliminary evaluation of tall buildings no more than 150 m. As both Buildings 2A and 2N are 141.8 m in height and have regular plans and elevation, a pushover analysis of Buildings 2N and 2A is conducted first as the preliminary seismic evaluation because it is easy to implement and able to approximately get a quick view of the nonlinear structural performances. More reliable seismic evaluations of these two buildings have been conducted by using nonlinear dynamic history analysis in subsequent sections. In the pushover analysis, the structure is subjected to an inverted triangular distribution of lateral forces. The resulting base shear force versus displacement relationships of Buildings 2A and 2N are provided in Figure 6. Figure 6a shows that the initial stiffness and lateral strength of Building 2N are higher, but its lateral resistance decreases rapidly after reaching the peak shear force. Although Building 2A exhibits slightly smaller initial stiffness and lateral strength, its ductile behavior is better than Building 2N. Figure 6b indicates that for Building 2N, the trends of the shear forces carried by the frame and core-tube are similar to the total shear force pattern. The core-tube bears a larger proportion of the total base shear. Therefore, the decrease in the lateral strength of the core-tube causes the decline of the global lateral strength of Building 2N. In contrast, the core-tube of Building 2A bears a larger proportion of the base shear force than that absorbed by the frame at the initial stage. However, when the core-tube reaches its maximum strength and begins to yield, the shear force taken by the frame begins to gradually increase. If the maximum displacement exceeds 0.67 m, the shear force taken by the frame will be larger than that taken by the core-tube. Ultimately, although the shear forces taken by the frame and the core-tube are different for these two buildings, their global structural seismic performances are still comparable.

7.2 Nonlinear dynamic time-history analysis

The nonlinear dynamic time-history analysis is intended to estimate and compare the performances of Buildings 2A and 2N for different earthquake intensities. To achieve this, three earthquake intensities are selected, including frequent earthquakes (i.e., 63% probability of exceedance in 50 years), fortification level earthquakes (i.e., 10% probability of exceedance) and severe earthquakes (i.e., 2~3% probability of exceedance) in the Chinese code. The popularly used 22 records of far-field ground motions recommended by FEMA P695 are adopted in the following structural seismic evaluation. The PGA of these selected ground motion records is scaled to 110 gal, 300 gal and 510 gal for frequent, fortification level and severe earthquakes, respectively, which is specified in the Chinese seismic design code GB 50011-2010 (CMC, 2010a) for the Intensity 8.5 region. The scaled ground motion records are input along the X direction of the buildings and classical Rayleigh damping is adopted with a damping ratio of 5% for the nonlinear time history analysis.

The mean values of displacement responses of Buildings 2A and 2N with the standard deviation subjected to the 22 selected ground motion records are shown in Figure 7. Figures 7a, c and e indicate that both the negative and positive mean displacement responses of the two buildings are very similar for all the three hazard levels. Furthermore, Figures 7b, d and f also demonstrate that the mean story drift ratios of Buildings 2A and 2N are comparable for all three intensities, except those above the 33rd story, where the story drift ratio of Building 2A is clearly larger than that of Building 2N. The maximum story drift ratio of Building 2A occurs at the 34th story with a value of 1/899, 1/320 and 1/184 for frequent, fortification level and severe earthquakes, respectively. The maximum story drift ratio of Building 2N occurs at the 33rd story with a value of 1/932, 1/348 and 1/197 for the three earthquake intensities. Therefore, the maximum story drift ratio of Building 2A is larger than that of Building 2N at all hazard levels. In addition, Building 2N has a smaller dispersion in displacement responses than Building 2A.

Taking one of the 22 ground motion records, i.e., CHICHI_CHY101-N, for instance, Figure 8 shows the plastic hinge distribution in Buildings 2N and 2A under the severe earthquake. It reveals little difference in the entire structural damage degree of the two designs. In both buildings, the plastic hinges at the beam ends form a uniform distribution along the building height and a significant number of column hinges occur at the bottom stories of the structure. The only difference is that a number of column hinges also form at the upper stories in Building 2A whereas beam hinges are dominant in Building 2N. To evaluate the detailed structure responses, the peak values of beam and column rotations, the normalized column axial forces, core wall compression strains, as well as coupling beam rotations in Buildings 2A and 2N subjected to CHICHI_CHY101-N at the severe earthquake level are summarized in Table 6. As Table 6 shows, the peak values of normalized column axial forces and shear wall compression strains in Building 2N are smaller than those in Building 2A, while the peak values of beam and coupling beam rotations are larger than Building 2A. The results indicate that Building 2N has stranger columns and core walls, but weaker beams and coupling beams. For other ground motions, both buildings suffer a lesser degree of damage, and similarly the damage degree of the two designs is comparable and more column hinges appear in Building 2A.

Overall, the two designs exhibit generally similar structural performances under different levels of earthquakes.

8. Conclusions

Based on a typical case study of a core-tube frame structure Building 2A provided by PEER (Moehle et al., 2011), Building 2N is generated through a redesign process according to the Chinese seismic design code. The design procedures of these two buildings and their seismic performances under different earthquake intensities are compared and evaluated in some detail. The study indicates that the seismic design forces determined by the Chinese response spectrum are larger than the US counterparts at the same seismic hazard level. In addition, a higher requirement for the inter-story drift ratio is specified by the Chinese code, thereby resulting in larger seismic design forces. These two aspects together have led to a higher level of material consumption for Building 2N than Building 2A. Nonetheless, the global level performance assessment, including the story drift ratio and plastic hinge distribution, indicates that the two designs exhibit approximately similar structural performances under different levels of earthquakes. The preliminary comparison at the component level indicates that Building 2N has stranger columns and core walls, but weaker beams and coupling beams.

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List of Tables

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Table 1 Seismic design parameters used for Building 2A (Moehle et al., 2011)

S_s	1.725 g
S_l	0.602 g
F_a	1
F_v	1.3
S_{MS}	1.718 g
S_{Ml}	0.782 g
S_{DS}	1.145 g
S_{Dl}	0.521 g
R	7.0
Site Class	C
C_d	5.5
C_s	0.051
Seismic weight (W)	45372 kN
Modal combination method	Complete quadratic combination (CQC)
Redundancy factor (ρ)	1.0
Accidental eccentricity	5%
Base shear "V" (See section 12.8 in ASCE 7-05)	23140 kN
Modal Base shear "V_t" (See section 12.9.2 in ASCE 7-05)	V_{tx}=50870/R=7267 kN V_{ty}=52311/R=7473 kN
Modal base shear scaled to match 0.85V	0.85×23140=19669 kN

Table 2 Vertical design load (Moehle et al., 2011) for both buildings

Application	Location	Superimposed dead load (units: kN/m^2)	Live load (units: kN/m^2)
Parking	4 stories below ground	0.1435	2.392
Retail	Ground level inside area	5.263	4.785
Cladding	Tower perimeter	0.7177	0
Outside plaza	Ground level outside area	16.747	4.785
Corridors and exit areas	Inside elevator core	1.340	4.785
Residential	2nd-42nd floor	1.340	1.914
Mechanical	At roof floor only	444.528 kN	1.196
Roof	Roof floor	1.340	0.9569

Table 3 The effective seismic weight and design periods

		Building 2N	Building 2A (Moehle et al., 2011)	
Effective seismic weight (units: ton)		57,306.0	46298.0	
Period (units: s)	T_1	2.565	4.456	Translation mode in the X direction
	T_2	2.383	4.026	Translation mode in the Y direction
	T_3	1.992	2.478	Torsion mode

Note: The X and Y directions of Buildings 2N and 2A are illustrated in Figure 1.

Table 4 The material properties and dimensions of the main structural members in Buildings 2A and 2N

			Building 2A (Moehle et al., 2011)	Building 2N
Beams	Material property (units: MPa)	Specified strength	34.5	26.8
		Expected strength	44.8	36.1
	Dimension (units: mm)		762×914	250×500, 450×900
Columns	Material property (units: MPa)	Specified strength	34.5, 41.4, 55.2, 69.0	26.8, 32.4, 38.5
		Expected strength	44.8, 53.8, 71.7, 89.6	36.1, 42.9, 50.1
	Dimension (units: mm)		1170×1170 - 915×915	1500×1500 - 800×800
Shear walls	Material property (units: MPa)	Specified strength	34.5, 41.4	26.8, 32.4, 38.5
		Expected strength	44.8, 53.8	36.1, 42.9, 50.1
	Thickness (units: mm)		610, 460	400 - 600

Table 5 Material properties and dimensions for the main structural members in Building 2N

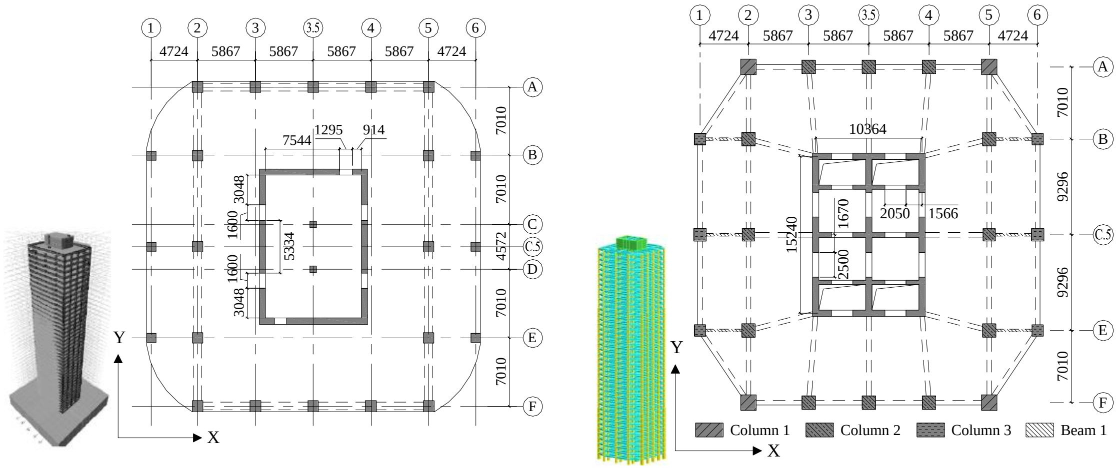
Element	Member location (Figure 1b)	Floor	Specified strength of concrete (units: MPa)	Dimension (units: mm)
Slabs	All positions of 1st-41th floor and inside core tube of 42nd floor		36.1	140
	Outside core tube of 42nd floor and all positions of 43rd floor		36.1	150
Moment Frame beams	Beam 1	All floors	36.1	250×500
	All Beams except Beam 1	All floors	36.1	450×900
Moment Frame Columns	Column 1	1^{st}-10^{th} floor	50.1	1500×1500
		11^{th}-20^{th} floor	50.1	1300×1300
		21^{st}-30^{th} floor	42.9	1200×1200
		31^{st}-42^{nd} floor	36.1	1000×1000
	Column 2	1^{st}-10^{th} floor	50.1	1300×1300
		11^{th}-20^{th} floor	50.1	1200×1200
		21^{st}-30^{th} floor	42.9	1100×1100
		31^{st}-42^{nd} floor	36.1	900×900
	Column 3	$1^{st-}10^{th}$ floor	50.1	1100×1100
		11^{th}-20^{th} floor	50.1	1000×1000
		21^{st}-30^{th} floor	42.9	900×900
		31^{st}-42^{nd} floor	36.1	800×800
Core Walls	Internal walls in the X direction	1^{st}-10^{th} floor	50.1	470
		11^{th}-20^{th} floor	50.1	400
		21^{st}-30^{th} floor	42.9	400
		31^{st}-42^{nd} floor	36.1	400
	External walls in the X direction	1^{st}-20^{th} floor	50.1	600
		21^{st}-30^{th} floor	42.9	600
		31^{st}-43^{rd} floor	36.1	500
	All walls in the Y direction	1^{st}-10^{th} floor	50.1	550
		11^{th}-20^{th} floor	50.1	450
		21^{st}-30^{th} floor	42.9	400
		31^{st}-43^{rd} floor	36.1	400
Coupling Beams	Same as core walls	Same as core walls	Same as core walls	600 in depth

Note: All reinforcement consists of HRB400 reinforcing bar, whose specified strength is 400 MPa and expected strength 455.7 MPa.

Table 6 Peak values of component level responses in Buildings 2A and 2N

	Beam rotation (units: rad)	Normalized column axial force	Column rotation (units: rad)	Core wall compression strain	Coupling beam rotation (units: rad)
Building 2A	0.012	0.80	0.0043	0.0018	0.019
Building 2N	0.016	0.55	0.0045	0.0011	0.025

Note: The normalized column axial force represents the column axial force which is normalized by $A_g f_c$, where A_g is the column cross sectional area and f_c the expected concrete strength.



(a) Building 2A (Moehle et al., 2011)

(b) Building 2N

Figure 1 3D view and typical floor plan of Buildings 2A and 2N (units: mm)

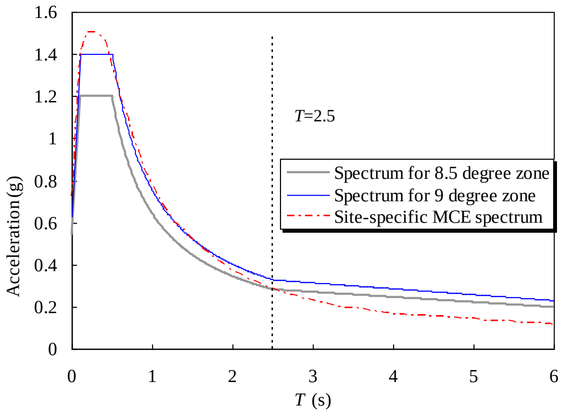
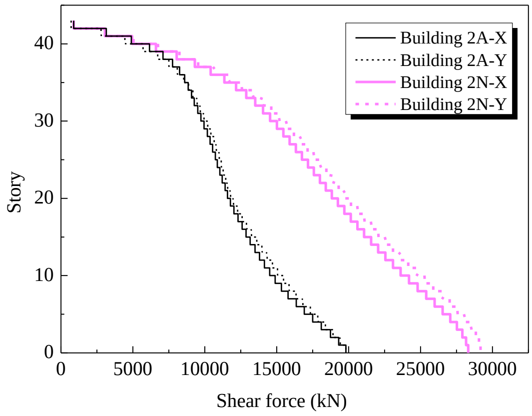
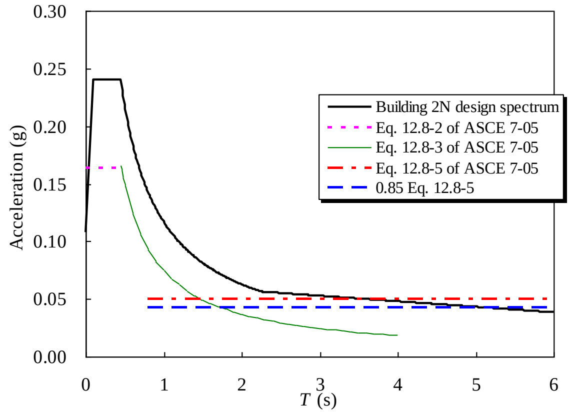


Figure 2 Comparison between the site-specific MCE spectrum and Chinese response spectra

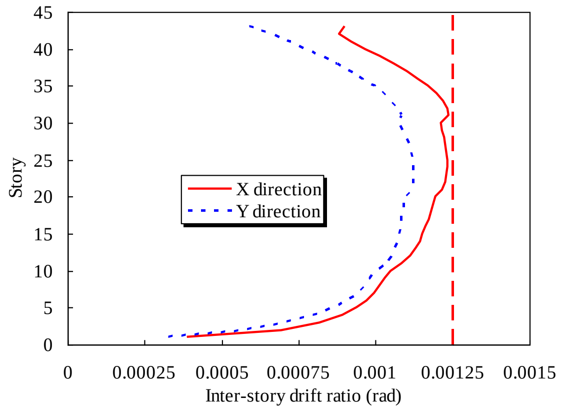


(a) Design lateral force

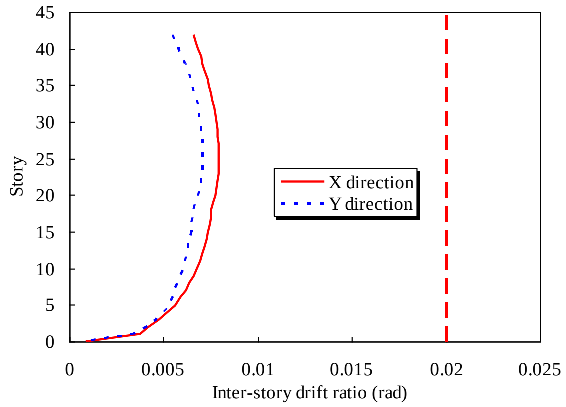


(b) Seismic response coefficient

Figure 3 The design lateral forces and seismic response coefficients in Buildings 2A and 2N

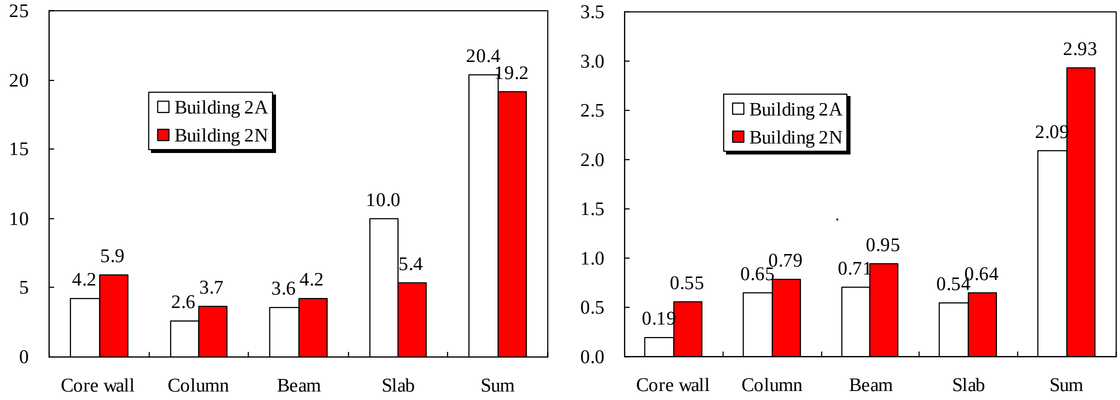


(a) The design story drift ratio of Building 2N



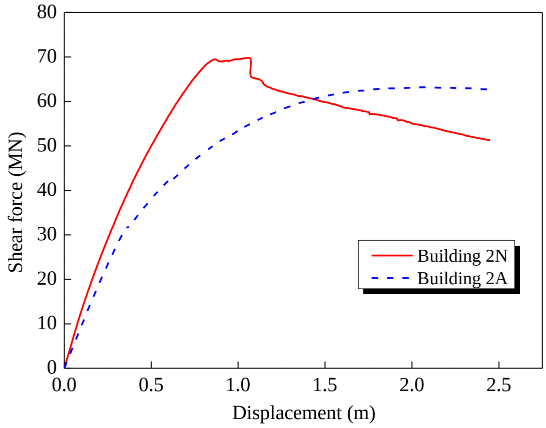
(b) The design story drift ratio of Building 2A (Moehle et al., 2011)

Figure 4 The design story drift ratio of Buildings 2A and 2N

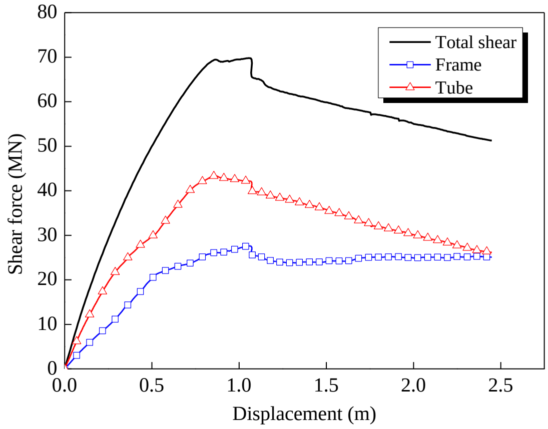


(a) Concrete ($\times 10^3$ m^3) (b) Reinforcement steel ($\times 10^3$ ton)

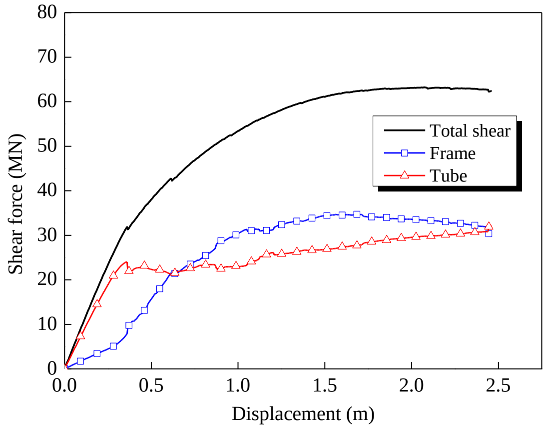
Figure 5 The material consumptions in Buildings 2A and 2N



(a) Pushover capacity curves of Buildings 2N and 2A

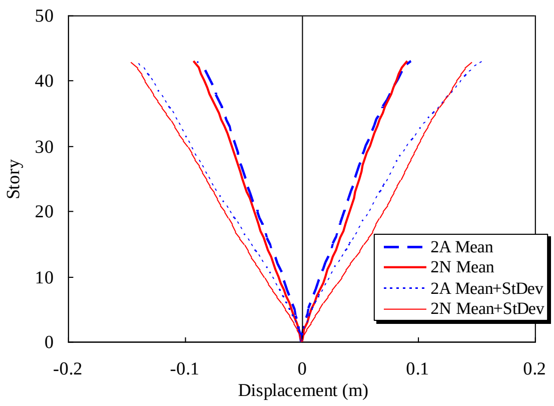


(b) Base shear distribution of Building 2N

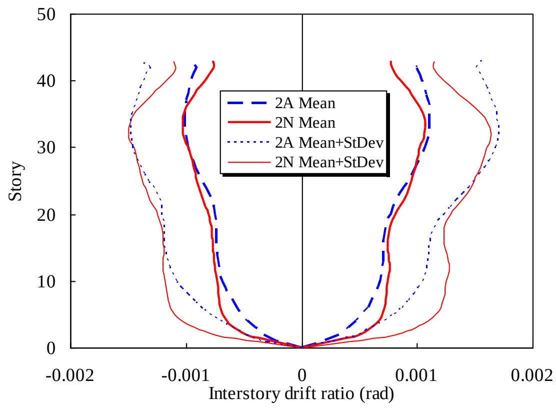


(c) Base shear distribution of Building 2A

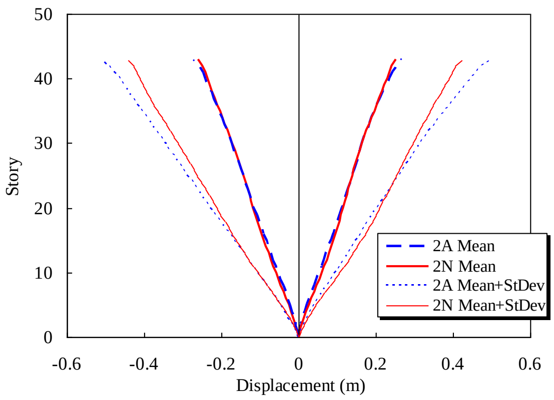
Figure 6 Base shear force-displacement relationships of Buildings 2A and 2N



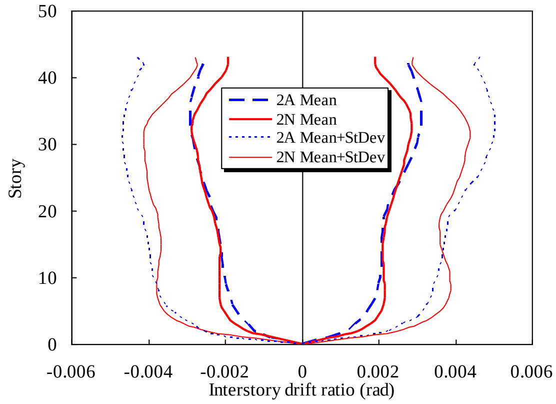
(a) Story displacement under frequent earthquakes (PGA = 110 gal)



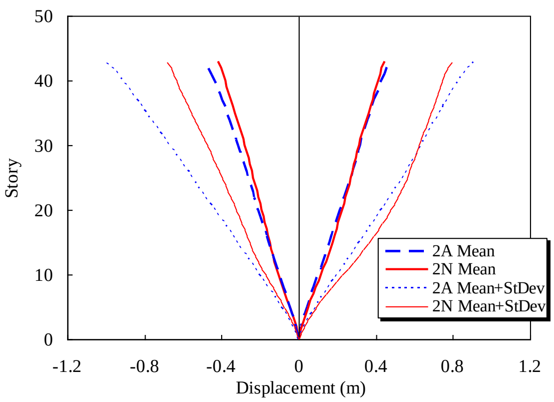
(b) Story drift ratio under frequent earthquakes (PGA = 110 gal)



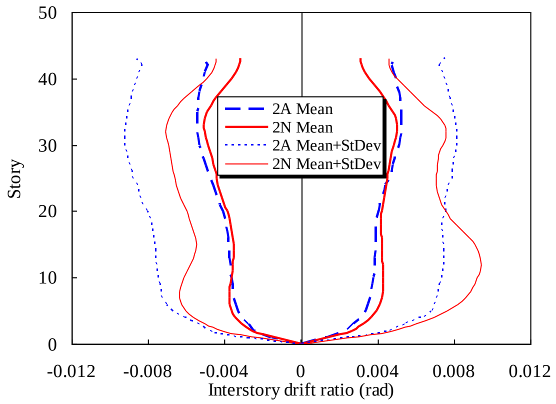
(c) Story displacement under fortification level earthquakes (PGA = 300 gal)



(d) Story drift ratio under fortification level earthquakes (PGA = 300 gal)

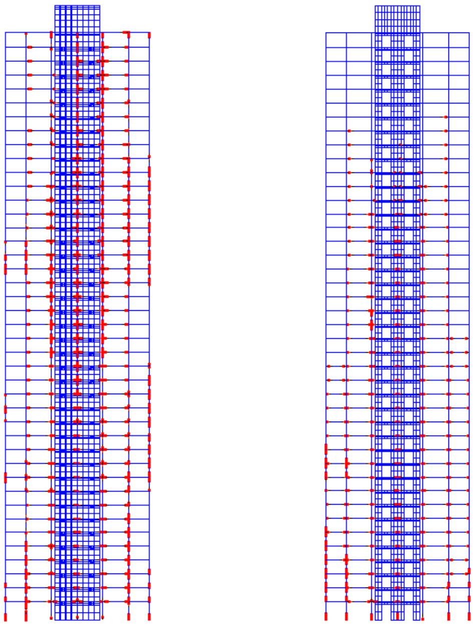


(e) Story displacement under severe earthquakes (PGA = 510 gal)



(f) Story drift ratio under severe earthquakes (PGA = 510 gal)

Figure 7 Displacement responses of Buildings 2A and 2N



(a) Building 2A (b) Building 2N

Figure 8 Plastic hinge distribution of Buildings 2A and 2N subjected to CHICHI_CHY101-N

(PGA=510 gal)