SEISMIC LOSS ASSESSMENT OF TYPICAL RC FRAME-CORE TUBE TALL BUILDINGS IN CHINA AND US USING THE FEMA P-58 PROCEDURE

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ABSTRACT

Reinforced concrete (RC) frame-core tube buildings are widely constructed both in China and the United States (US). Their seismic performances greatly influence the economic loss of earthquakes. This study aims to compare the seismic losses of two typical RC frame-core tube tall buildings designed following the Chinese and the US seismic design codes. The prototype building is originally designed using the US seismic design codes, provided by the Tall Building Initiative (TBI) Project. Then the prototype building is redesigned according to the Chinese seismic design codes with the same design conditions and seismic hazard level. Detailed nonlinear finite element (FE) models are established for both designs. These models are used to evaluate their seismic responses at different earthquake intensities, including the service level earthquake (SLE), the design based earthquake (DBE) and the maximum considered earthquake (MCE). In addition, the collapse fragility functions of these two buildings are established using the incremental dynamic analysis (IDA). Subsequently, the seismic loss consequences (repair costs, repair workload, and casualties) of these two designs are calculated using the procedure proposed by FEMA P-58. The comparison shows that the Chinese design exhibits better seismic performances in most cases with smaller total repair cost, shorter repair time and a smaller number of casualties, except slightly longer repair time at the MCE level. For both designs, the repair cost of nonstructural components accounts for the majority of the total cost. The ceilings and elevators are the major causes of casualties at the MCE level.

KEYWORDS

Frame-core tube, tall buildings, nonlinear analysis, collapse fragility, seismic loss

INTRODUCTION

RC frame-core tube buildings are widely constructed both in China and the US. Their seismic performances greatly influence the economic loss of earthquakes. Comparative studies of the seismic performances of RC frame-core tube tall buildings in China and the US will provide good examples to understand the differences between the Chinese and the US seismic design systems, which may be helpful for further optimization of the design of tall buildings in China. Nonlinear time history analysis (THA) is the most widely used seismic performance assessment method of tall buildings currently. However, the limitations of this method have been identified in terms of: (1) questions regarding the accuracy and reliability of this method in predicting actual building responses; (2) the lack of performance measures that are more meaningful and useful for decision making purposes; (3) the inability to reliably and expediently consider the non-structural systems. The next-generation performance-based seismic design project (FEMA, 2006) developed a new methodology for seismic performance assessment of individual buildings, namely the FEMA P-58 procedure (FEMA, 2012a; FEMA, 2012b), to address these limitations. The technical basis of this methodology is the framework for performance-based earthquake engineering developed by the Pacific Earthquake Engineering Research Center (PEER) (Moehle and Deierlein, 2004). The FEMA P-58 procedure accounts for the uncertainties associated with current ability to accurately predict the actual response, the intensity of ground shaking, the actual occurrence of the damage state, etc. This procedure uses a probabilistic approach to characterize performance so that the reliability with which performance may be attained can be fully understood. Unlike the nonlinear THA, which describes the building performance in terms of structural responses such as story drift ratio, acceleration and material stress, the FEMA P-58 procedure describes the structural performance in terms of the consequences of earthquake damage including: casualties; repair costs, repair time, etc.

This study aims to compare the seismic losses of two typical RC frame-core tube tall buildings designed following the Chinese and the US seismic design codes. The prototype building is originally designed using the US seismic design codes, provided by the TBI Project (Moehle et al., 2011). Then the prototype building is redesigned according to the Chinese seismic design codes with the same design conditions and seismic hazard level (Lu et al., 2015). Detailed nonlinear FE models are established for both designs. These models are used to

evaluate their seismic responses at different earthquake intensities. In addition, the collapse fragility functions of these two buildings are established using the IDA. Based on the above analysis, the seismic loss consequences (repair costs, repair workload and casualties) of these two designs are calculated using the procedure proposed by FEMA P-58.

DESCRIPTION OF THE STUDIED BUILDINGS

The prototype RC frame-core tube building, Building 2A, is one of the case study buildings in the TBI research program launched by PEER in 2006 (Moehle et al., 2011). Building 2A is a 42-story residential building including a 6.1-m tall penthouse on the top. The total height of the building is 141.8 m above the ground. Figure 1(a) shows the three-dimensional (3D) view and the typical floor plan of this building. Building 2A was designed based on the International Building Code (ICC, 2006). The published report of Moehle et al. (2011) provided detailed design information and outcomes of Building 2A.

Building 2A is redesigned following the Chinese codes by the authors, mainly including the Code for the Seismic Design of Buildings GB50011-2010 (CMC, 2010a), the Technical Specification for Concrete Structures of Tall Building JGJ3-2010 (CMC, 2010b), and the Code for Design of Concrete Structures GB50010-2010 (CMC, 2010c). The redesigned building is referred to as Building 2N. All design details of 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. A detailed comparison of the two designs can be found in Lu et al. (2015). Lu et al. (2015) indicates that Building 2N has larger columns and more internal walls in the core tube. 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 story drift ratio is specified by the Chinese codes, thereby resulting in larger seismic design forces. These two aspects together have led to a higher level of material consumption (concrete and reinforcement) for Building 2N than Building 2A.
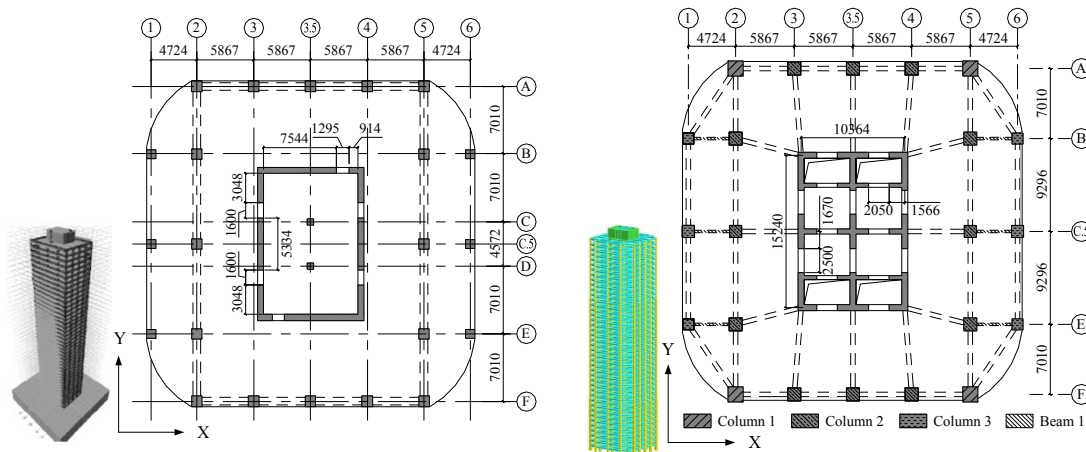
(a) Building 2A (Moehle et al., 2011)   (b) Building 2N (Lu et al., 2015)
Figure 1 Three dimensional views and typical floor plans of Buildings 2A and 2N (unit: mm)

NONLINEAR FE ANALYSIS OF BUILDINGS 2A AND 2N

Based on the design outcomes of Buildings 2A and 2N, 3D nonlinear FE models of the two buildings are established using MSC.Marc software. Modeling details can be found in Lu et al. (2015). The nonlinear THA is intended to assess and compare the performances of Buildings 2A and 2N for different earthquake intensities. To achieve this object, three earthquake intensities are selected, including SLE (i.e. a 63% probability of exceedance in 50 years), DBE (i.e. a 10% probability of exceedance in 50 years) and MCE (i.e. a 2-3% probability of exceedance in 50 years) in the Chinese code. The popularly used 22 records of far-field ground motions recommended by FEMA P695 (FEMA, 2009) are adopted in the following structural seismic evaluation. The corresponding peak ground acceleration (PGA) values of these selected ground motion records are scaled to 110, 300 and 510 cm/s^2 for SLE, DBE and MCE, 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 the classical Rayleigh damping is adopted with a damping ratio of 5% for the nonlinear THA.

(a) Story drift ratio
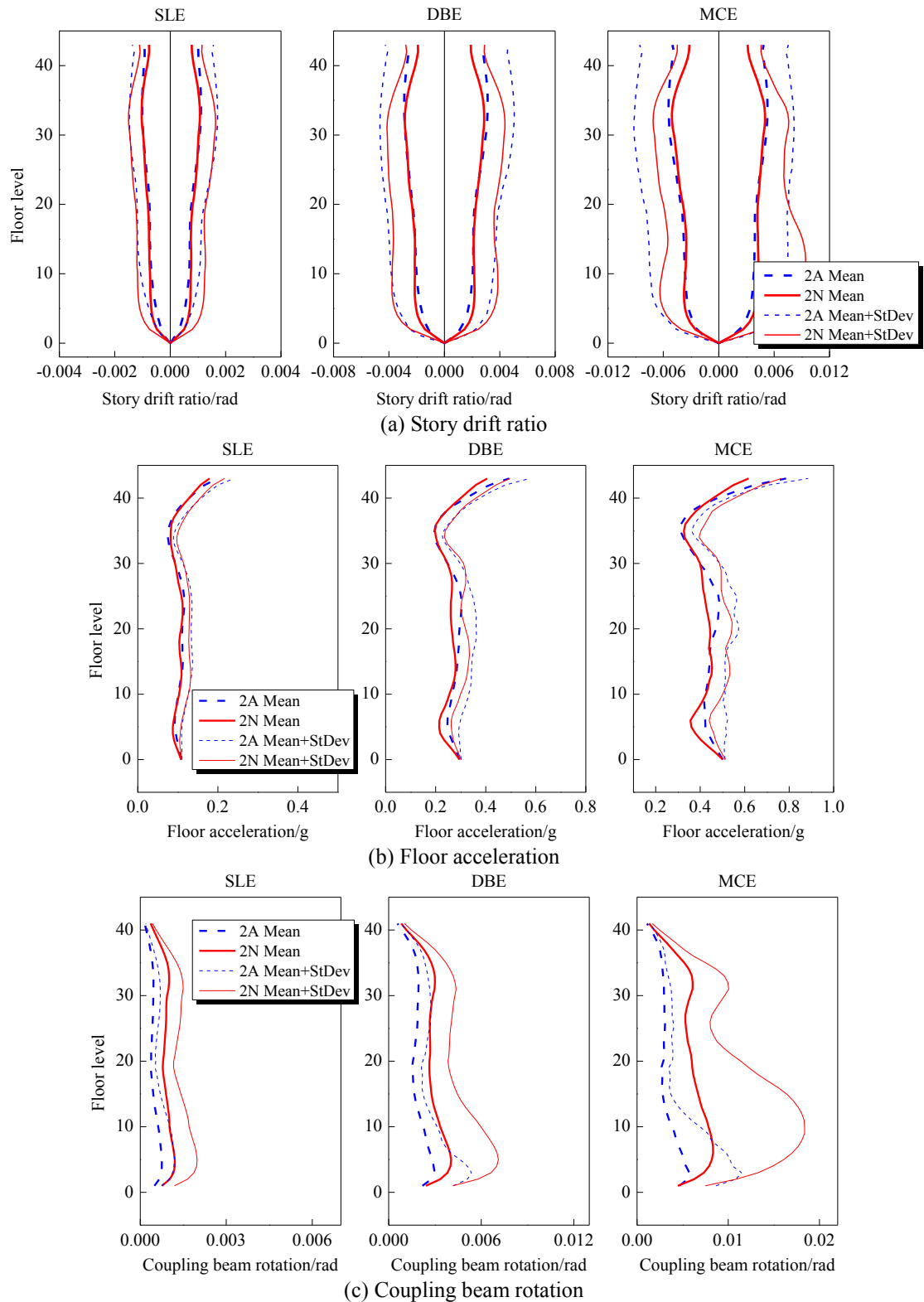

(b) Floor acceleration

(c) Coupling beam rotation

Figure 2 Structural responses of Buildings 2A and 2N

The mean values and standard deviations of story drift ratio, floor acceleration and coupling beam rotation of Buildings 2A and 2N subjected to the selected ground motion records are shown in Figure 2. Figure 2(a) indicates that both the negative and positive mean story drift ratios of the two buildings are very similar for all the three earthquake intensities, while the maximum story drift ratio of Building 2A is slightly larger than that of Building 2N. Figure 2(b) demonstrates that the mean acceleration responses of Building 2A are also larger than that of Building 2N at most floors. In addition, the maximum acceleration response occurs at the top floor in both buildings. Figure 2(c) shows that the coupling beam rotations are obvious greater for Building 2N, as the

coupling beams in Building 2N have larger span-depth ratio than those in Building 2A (Lu et al., 2015).

IDA (Vamvatsikos and Cornell, 2002) is an effective method to evaluate the seismic performance of structures. Therefore, IDA is performed to evaluate the collapse resistance capacities of Buildings 2A and 2N. The ground motion intensity is increased gradually in IDA until the collapse occurs. Then, the critical ground motion intensity resulting in the structural collapse is obtained and the collapse fragility curves of these two buildings are shown in Figure 3. It indicates that Building 2N has a slightly higher collapse resistance capacity than that of Building 2A.
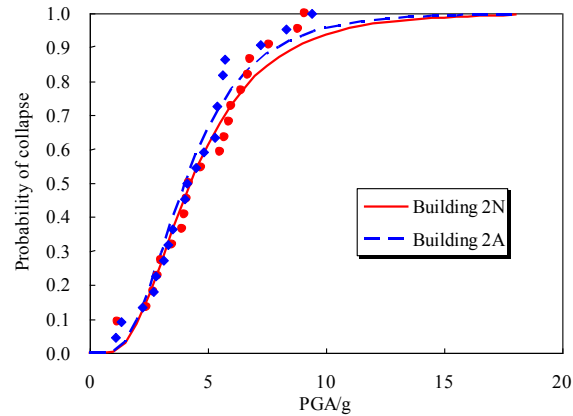

Figure 3 Collapse fragility functions of Buildings 2A and 2N

BUILDING PERFORMANCE MODELS OF PACT

The key step of the FEMA P-58 assessment procedure is to develop the building performance model. The performance model is a categorization of all those structural and nonstructural building components that are vulnerable to earthquake induced damage (Hamburger, 2011). In addition, the occupancy and the distribution of people in the building also need to be defined in the performance model. Vulnerable building components are categorized into fragility groups and performance groups. Fragility groups are sets of similar components that have the same potential damage characteristics in terms of vulnerability and consequences. Performance groups are subsets of a fragility group that will experience the same earthquake demands in response to earthquake shaking (FEMA, 2012a). Each fragility group has an associated series of damage states. Each damage state leads to a unique set of potential consequences, which may require some repair actions or threaten human life. The type and extent of damage that a component will experience is determined by using fragility functions. The component fragility functions are the distributions that indicate the conditional probability of incurring a damage state given a value of demand (FEMA, 2012a). Each damage state for each performance group also has an associated consequence function. Consequence functions are relationships that indicate the potential distribution of losses as a function of damage state. Consequence functions translate damage into potential repair and replacement costs, repair time, casualties, and other impacts (FEMA, 2012a).

Over 700 fragilities for common structural and nonstructural components and contents found in typical buildings and occupancies have been developed and provided as part of the FEMA P-58 methodology (FEMA, 2012b), called "Fragility Specification". Fragility specifications include information on component damage states, fragility functions, and consequence functions. For practical implementation of this methodology, an software, referred to as the Performance Assessment Calculation Tool (PACT) were developed, to help assemble the building performance model and implement a large number of building performance calculation.

This section briefly describes the PACT building performance models of Buildings 2A and 2N. Moehle et al. (2011) provides the total replacement cost of Building 2A ($149 million) and Davis Langdon Company (2010) presents the details of this cost. As Buildings 2A and 2N have the identical architectural layout and occupancy, this study assumes that all the other expenses for Building 2N are the same as Building 2A, except for the cost resulted from different amount of structural material consumptions (Lu et al., 2015). Accordingly, the total replacement cost of Building 2N is estimated as $151.3 million. The provided consequence functions in PACT represent the repair costs appropriate to Northern California in 2011. Users can address escalation and regional cost variation through the Region Cost Multiplier and Date Cost Multiplier to adjust the provided component repair cost consequence functions to appropriate present values (FEMA, 2012b). Since this study focuses on the differences of the seismic losses between the Chinese and the US designs, the inflation and region effect are ignored and the Region Cost Multiplier and Date Cost Multiplier are both assumed to be 1.0. The total loss threshold, which is the percentage of the construction cost where the structure would be taken as irreparable, is

assumed to be 1.0. A value of 1.0 insures that all the true consequence data is presented. If the value is less than 1.0, the consequences for realizations with higher costs would be replaced with the total repair costs and time, and the actual results would be lost (Jarrett et al., 2015). The replacement time for the two buildings is taken as 1500 days estimated in accordance with Comerio and Blecher (2010) and Comerio (2000). The occupancy type of the two buildings is retailing for the first floor and residence for all the other floors (Moehle et al., 2011). The corresponding population model provided by the PACT program is used.

Table 1 Structural component fragility specifications used in PACT models

Structural component name	Fragility specification	
	Building 2A	Building 2N
Moment frame joint	B1041.002a, B1041.003b	B1041.001a, B1041.001b B1041.002a, B1041.002b
Flat slab-column joint	B1049.031	-
Coupling beam	B1042.011a, B1042.011b	B1042.002b, B1042.012b
Shear wall	B1044.021, B1044.022, B1044.101	B1044.011, B1044.021 B1044.022, B1044.101

Table 2 Nonstructural component fragility specifications used in PACT models

Nonstructural component name		Fragility specification
Exterior enclosure	Exterior nonstructural wall	B2011.201a
	Exterior window system	B2022.001
Interiors	Partition	C1011.001a
	Stair	C2011.021b
	Wall finish	C3011.002a
	Ceilings and ceiling lighting	C3032.003b
Conveying	Elevator	D1014.011
Plumbing	Cold water piping	D2021.013a
	Hot water piping	D2022.013a
	Sanitary waste piping	D2031.013b
	Chilled water piping	D2051.013a
HVAC	Chiller	D3031.011c
	Cooling tower	D3031.021c
	HVAC ducting	D3041.021c
	HVAC drop	D3041.032c
	VAV box	D3041.041b
	Air handling unit	D3052.011c
Fire protection	Fire sprinkler drop	D4011.033a
Electrical service and distribution	Motor control center	D5012.013a
	Low voltage switchgear	D5012.021a
	Distribution panel	D5012.031a
Equipment and furnishings		Not considered

Following the definition of the general building characteristics, it is necessary to define the quantity, vulnerability, and distribution of damageable components and contents. PACT organizes this process into two parts: (1) identification of required fragility specifications for each floor, and (2) further division of fragility

specifications into different performance groups according to their demand parameters, and identification of the quantity of components in each performance group at each floor (FEMA, 2012b). The structural fragilities used in this study are shown in Table 1. The quantity of each structural performance group is calculated based on the actual number of structural components. The nonstructural component fragilities are chosen based on the building occupancy type with the aid of the provided Normative Quantity Spreadsheet, a supplemental file provided by the FEMA P-58 products. Table 2 shows the nonstructural fragilities used, which is identical for both the two buildings.

Depending on the selected fragility specifications, the peak story drift ratio, peak floor acceleration and peak coupling beam rotation are used as the structural demand parameters, which need to be entered into PACT. The previous THA of Buildings 2A and 2N has obtained these demand parameters for the selected 22 ground motions. The collapse fragility functions of these two buildings (Figure 3) are also established. When collapse occurs, the mean fatality rate, which is the fraction of occupants inhabiting the impacted collapse area who are fatally injured by the collapse, is assumed to be 10%; and the mean injury rate is assumed to be 90% according to FEMA (2008). One thousand Monte Carlo realizations are performed, which is generally sufficient to produce stable loss results (FEMA, 2012b).

SEISMIC LOSS RESULTS

With the parameters mentioned above, the seismic losses of Buildings 2A and 2N subjected to three different earthquake intensities are assessed using the FEMA P-58 procedure. No collapse has occurred in all 1000 realizations and the detailed assessment results are discussed as follows.

Repair cost

The median repair costs of Buildings 2A and 2N at three different earthquake intensities are compared in Figure 4. Since the comparison is conducted based on the median values, the median total repair cost does not equal to the sum of median structural and non-structural components repair costs. Figure 4 indicates that, in both designs, the repair cost of non-structural components accounts for the majority of the total repair cost at all the three earthquake intensities. At the SLE and DBE levels, the repair cost of structural components is very small; while this value increases at the MCE level.
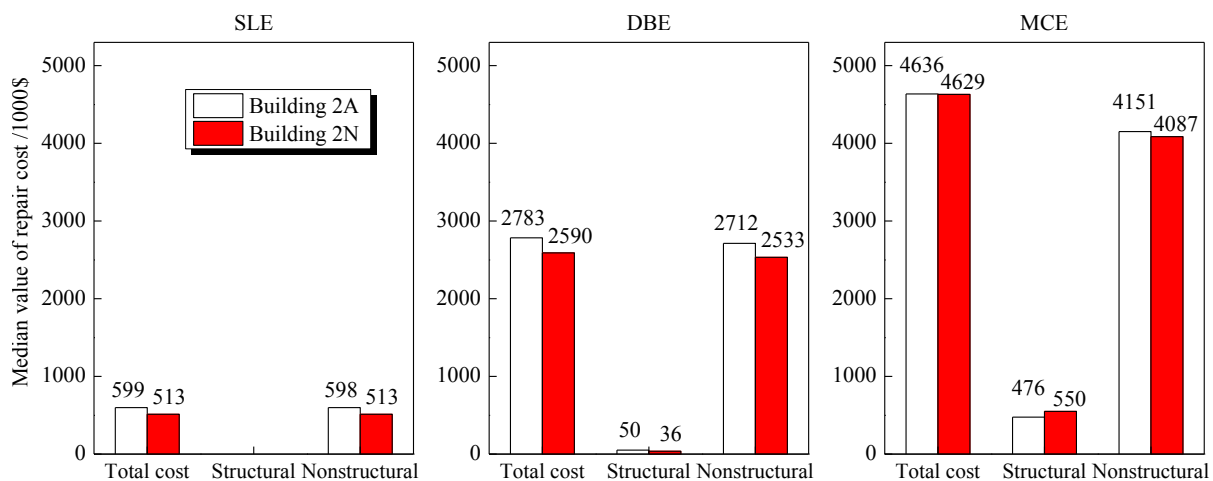

Figure 4 Median values of the repair cost for Buildings 2A and 2N

Comparison between the two buildings demonstrates that the median total repair cost of Building 2A is larger than that of Building 2N at all earthquake intensities. At the SLE and DBE levels, the total repair cost is dominated by the non-structural components, so the smaller non-structural component repair cost of Building 2N leads to an obviously smaller total cost. At the MCE level, the damage of structural components increase and Building 2N has more structural components (more internal walls, coupling beams and frame beams). These two reasons lead to a larger structural component repair cost in Building 2N than that that in Building 2A. Although Building 2N still has an obviously smaller repair cost of non-structural components, the gap between the total costs of Buildings 2A and 2N has been narrowed.

Note that the repair costs of Building 2A also has been assessed in the TBI program with the FEMA P-58 procedure (Moehle et al., 2011). Because the ground motions selected in the TBI program are different from

those used in this study, the corresponding response demand parameters and the estimated repair costs are consequently not consistent with each other. Additional verification indicates that if the structural responses used in the TBI program are also adopted in this assessment, the calculated repair costs will coincide with the values in the TBI program. It proves that the established building performance models and the operational approach used in this study are correct.

(a) Building 2A decomposition at the SLE level

(b) Building 2N decomposition at the SLE level

(c) Building 2A decomposition at the DBE level

(d) Building 2N decomposition at the DBE level

(e) Building 2A decomposition at the MCE level
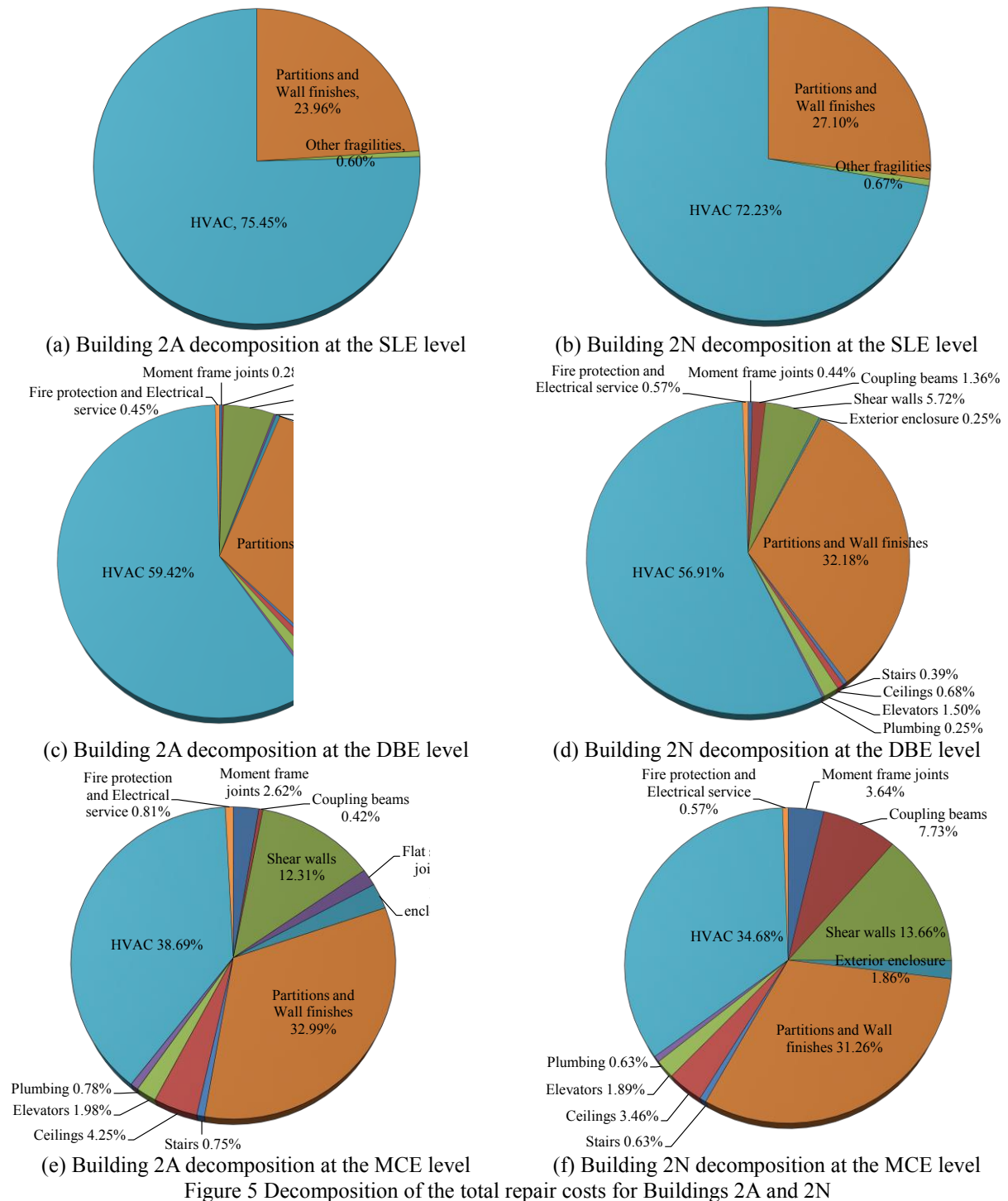
(f) Building 2N decomposition at the MCE level

Figure 5 Decomposition of the total repair costs for Buildings 2A and 2N

The decomposition of the total repair costs for Buildings 2A and 2N are shown in Figure 5. Figures 5(a) and 5(b) reveal that the repair costs are mainly caused by the damage of HVAC system, partitions and wall finishes at the SLE level. And at the DBE level (Figures 5(c) and 5(d)), the repair costs of HVAC system, partitions and wall finishes still account for the majority of the total cost. Meanwhile, the damage of shear walls, coupling beams, ceilings and elevators cannot be neglected either. At the MCE level (Figure 5(e) and 5(f)), the repair cost of structural components increases obviously, in which the repair cost of shear walls accounts for the majority.

Repair time

The repair time is characterized by the number of work days required for building restoration. A work day represents the labor quantity of a worker in one day. The comparison of the median values of the repair work days for Buildings 2A and 2N are shown in Figure 6. Similar to the repair cost, the median repair workload of nonstructural components accounts for the majority of the total repair workload for both designs at all the three earthquake intensities. The median repair workload of structural components is very small at the SLE and DBE level; while at the MCE level, this value becomes larger.

Comparison between the two buildings demonstrates that Building 2A has a larger amount of workload than that of Building 2N at the SLE and DBE levels. However, Building 2N requires a larger amount of workload at the MCE level. Three main reasons contribute to this conclusion: the damages of structural components increase at this level, Building 2N has more structural components, and the repair times for structural components are generally longer than those of non-structural components.
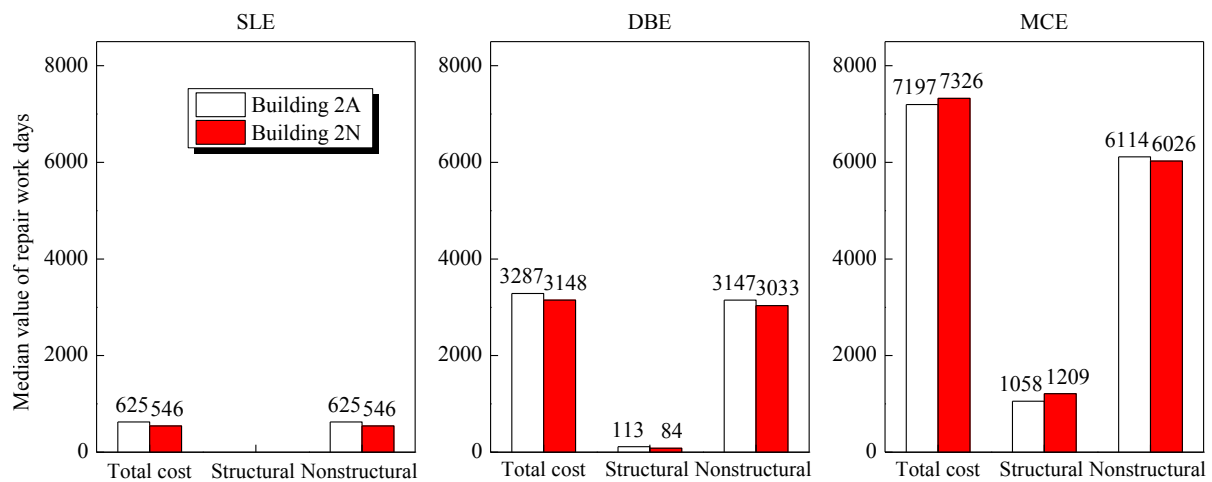

Figure 6 Median values of the repair work days for Buildings 2A and 2N

Casualties

No casualties occur for both buildings at the SLE and DBE levels. Table 3 shows the casualties of Buildings 2A and 2N at the MCE level. The overall casualties are quite low and Building 2N has a relatively smaller number of casualties. The ceilings and elevators are the major causes of casualties at the MCE level. Thus, credible measures ensuring the anchor of ceilings and other fragile non-structural components and the safety of the elevators can effectively reduce the direct casualties caused by earthquakes.

Table 3 Casualties of Buildings 2A and 2N at the MCE level

	Building 2A		Building 2N	
	Median value	Ninety percent value	Median value	Ninety percent value
Number of injuries	0.0577	0.447	0.0285	0.418
Number of fatalities	0.00157	0.0253	0.00166	0.0249

CONCLUSIONS

This study provides the comparative analyses of the seismic losses of two typical RC frame-core tube tall buildings designed following the Chinese and the US seismic design codes. The FEMA P-58 procedure is used to assess the seismic loss consequences. The comparison shows that the Chinese design exhibits better seismic performances in most cases with smaller total repair cost, shorter repair time and a smaller number of casualties, except slightly larger repair time at the MCE level. For both designs, the repair cost of nonstructural components accounts for the majority of the total loss. The ceilings and elevators are the major causes of casualties at the MCE level.

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