Development and application of a simplified model for the design of a super-tall mega-braced frame-core tube building

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Abstract: Resilience-based earthquake design for next-generation super-tall buildings has become an important trend in earthquake engineering. Due to the complex structural system in super-tall buildings and the extreme computational workload produced when using refined finite element (FE) models to design such buildings, it is rather difficult to efficiently perform a comparison of different design schemes of super-tall buildings and to investigate the advantages and disadvantages of different designs. Here, a simplified nonlinear model is developed and applied to compare two design schemes (the fully braced scheme and half-braced scheme) of a super-tall mega-braced frame-core tube building, which is an actual engineering project with a total height of approximately 540 m. The accuracy of the simplified model is validated through a comparison of the results of modal analyses, static analyses, and dynamic time history analyses using the refined FE models. Subsequently, the plastic energy dissipation of different components and the distribution of the total plastic energy dissipation over the height of the two design schemes are compared using the proposed simplified model. The analyses indicate that the fully braced scheme is superior because of its more uniform energy distribution along the building height and the large amount of energy dissipated in the replaceable coupling beams, which enables rapid repair and re-occupancy after an earthquake. In contrast, the potential damage in the half-braced scheme is more concentrated and more severe, and the damage in the core tubes is difficult to repair after an earthquake.

Keywords: super-tall building; simplified model; resilience; plastic energy dissipation; plastic energy distribution.

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1. INTRODUCTION

In recent years, studies on the resilience-based seismic design of super-tall buildings have become increasingly popular [1-5]. To design an earthquake-resilient super-tall building, the performance of super-tall buildings subjected to various earthquake intensities should be accurately simulated; such simulations are used to predict the seismic energy dissipated in replaceable and repairable components as well as the structural damage in key components that are difficult to repair.

Numerous studies have been conducted using three-dimensional (3D) refined FE models to investigate the nonlinear seismic performance and predict the potential collapse modes of super-tall buildings. Nonlinear time-history analyses of various super-tall buildings, including the Taipei Financial Center (H = 508 m) [6, 7], the Shanghai Tower (H = 632 m) [8-12], the Republic Plaza (H = 280 m) [13], and the Shanghai World Financial Tower (H = 420.5 m) [14], were conducted using refined FE models, which were established using various general-purpose FE software packages (e.g., ANSYS [15], Perform 3D [16], LS-DYNA [17] and ABAQUS [18-23]) and open-source software packages (e.g., OpenSees [24]). The seismic performances of these super-tall buildings subjected to various seismic intensities were predicted to optimize the seismic designs. More recently, collapse simulations of super-tall buildings subjected to extreme earthquakes were successfully performed by Lu et al. [25, 26] using MSC.Marc [27] and by numerous researchers using ABAQUS [19-21]. The potential collapse modes of these super-tall buildings were predicted, and the critical zones that might induce collapse were identified, which could serve as a reference for future improved designs.

As described above, the refined FE model has been widely applied to investigate the seismic performance and reveal the potential collapse modes of tall and super-tall buildings with various structural systems [6-14, 19-23, 25, 26, 28]. However, such simulations have several drawbacks: first, the refined FE model cannot be accurately established without specific structural design details, which are typically unavailable at the preliminary design stage, thus restricting the applications of this type of model at this stage. Moreover, super-tall buildings are typically composed of many different components, thereby leading to an extremely large computational workload and low efficiency when using the refined FE models. Such models restrict the implementation of parametric analyses or incremental dynamic analyses (IDAs). In particular, several different design schemes are typically proposed at the preliminary design stage. Due to the lack of specific design details and the large computational workload, the comparison between various design schemes, which is essential for the design of super-tall buildings, cannot be easily performed using the refined FE models.

In contrast, a simplified model that represents the key nonlinear and dynamic characteristics of super-tall buildings and that effectively reduces the computational effort has the potential to facilitate the comparison of different preliminary design schemes. Moreover, if the engineering demand parameters are available through the analysis of the simplified model during the preliminary design stage, such a model can also be used to guide and optimize the preliminary design.

Although limited research has been reported on establishing a simplified model of super-tall buildings, many researchers have conducted studies on establishing simplified models for conventional tall buildings. For example, a simplified model for the framed-tube structure proposed by Connor and Pouangare [29] was applied to analyze its elastic response subjected to static lateral loads and subsequently used to guide the preliminary design. Luco and Cornell [30] developed a simplified model involving the interconnection of two shear beams to predict the seismic performance of tall buildings. Maftah et al. [31] presented a simplified approach for the seismic analysis of a tall building braced by shear walls and thin-walled open section structures, and a simplified formulation for the vibrational frequencies and internal forces subjected to earthquakes was obtained based on D'Alembert's principle. An important achievement in the simplified modeling of super-tall buildings was accomplished by Lu et al. [32]; specifically, a two-dimensional (2D) simplified model encompassing nonlinear beam-column elements and nonlinear spring elements for the Shanghai Tower (H = 632 m) was proposed. The reliability of this model was validated by comparing the results of the simplified model with those of the refined FE model. The analyses of the plastic energy dissipation indicated that the outrigger was the primary plastic energy dissipation component, and the total plastic energy distribution along the height of the building subjected to three seismic intensities was identified. Despite these efforts, the simplified model has only been used for the Shanghai Tower (which is a mega-column/core-tube/outrigger system), in a study by Lu et al. [32]. Additional validation of the reliability of this model is required for other types of super-tall buildings. In addition, further studies should also be performed on the application of the simplified model at the preliminary design stage and the comparison of different design schemes.

Therefore, based on the simplified model and associated parameter determination approaches proposed by Lu et al. [32], a simplified model is developed for the seismic analysis of an actual super-tall mega-braced frame-core tube building. In addition, this simplified model is used to perform the comparison of two preliminary design schemes for this building in terms of its resilient performance. The studies indicate that this simplified model is also capable of efficiently and reliably predicting the key seismic characteristics of this building, thereby laying a foundation for the further comparison of different design schemes. Subsequently, the energy dissipation characteristics of these two structural schemes are investigated and discussed through nonlinear time-history analyses using the simplified models. The plastic energy dissipation contribution of each component as well as the total plastic energy distribution along the height of the building are compared for both schemes, thereby conclusively providing a reference for the selection of a better option among the various considered schemes. The analytical results indicate that the fully braced scheme induces a more uniform plastic energy dissipation distribution than the half-braced scheme. Furthermore, the fully braced scheme effectively enables the energy dissipation to be located in the readily replaceable components (e.g., coupling beams and perimetric trusses) instead of the key components that are difficult to repair (e.g., mega columns, core tubes and mega braces). As a result, the fully braced scheme provides a better seismic resilient performance than the half-braced scheme. The outcome of this study serves as a guideline for a method to reliably and efficiently understand the seismic performance of different preliminary design schemes of super-tall buildings, which can provide guidance and serve as a reference for the performance-based and resilience-based earthquake design of super-tall buildings.

2. INTRODUCTION OF TWO DESIGN SCHEMES AND THE ASSOCIATED REFINED FE MODELS

The project studied in this research is a multi-functional, super-tall office building with a total height of approximately 540 m. The building adopts a hybrid lateral-force resisting system named the "mega-braced frame-core tube" [26]. Two design schemes are proposed at the preliminary design stage, which are referred to as the "fully braced scheme" and "half-braced scheme". The fully braced scheme involves the use of mega columns, mega braces within the full height of the structure (i.e., Zones 1 to 8), perimetric trusses, concrete core tubes, and secondary frames, as shown in Fig. 1. In contrast, the half-braced scheme involves the use of mega columns, mega braces in the lower four zones of the structure (i.e., Zones 1 to 4), outer frame tubes in the higher four zones (i.e., Zones 5 to 8), perimetric trusses, outriggers, concrete core tubes, and secondary frames, as shown in Fig. 2. Further details of the half-braced scheme are presented in Lu et al. [26]. The differences between these two schemes are listed in Table 1.

This super-tall building is located in Beijing, a relatively high seismic region in China [33] (with a maximum spectrum acceleration of 0.9 g for the Maximum Considered Earthquake (MCE) level, where g is the acceleration of gravity); both the wind and seismic loads play important roles in the structural design. An elastic analysis of the building indicates that the maximum drift ratios, when subjected to the designed seismic load (i.e., 63% probabilities of exceedance in 50 years) and the designed wind load, are approximately 1/940 and 1/570, respectively, both of which meet the maximum allowable story drift ratio of 1/500 at the design level. Nevertheless, if this building is incapable of being functionally recovered immediately after an earthquake due to severe damage that could occur in key components, great economic losses will occur, and there will be negative social impacts. Hence, for this super-tall building, the controls of the degree and location of damage are critical issues; ideally, the damage should be uniformly distributed in the replaceable components along the height of the building, and severe damage inside the key components and damage concentration along the height of the building should be avoided.

The 3D refined FE model of the half-braced scheme was established by Lu et al. [26] using MSC.Marc. In this work, the 3D refined FE model of the fully braced scheme is also established using identical methodology proposed by Lu et al. [26]. The refined FE models of two structural schemes are illustrated in Fig. 3, including 78,099 elements for the fully braced scheme and 64,875 elements for the half-braced scheme.

3. ESTABLISHMENT OF THE SIMPLIFIED MODEL

3.1 General provisions of the simplified model

Simplified models of both schemes are established based on the simplification approaches proposed by Lu et al. [32]. Because the major objective of establishing these simplified models was to compare the dynamic properties and characterize the plastic energy dissipation of the two schemes, the following three principles are followed when developing the simplified models.

(1) The models of the 3D building are simplified as planar models. The fundamental periods in the x and y directions of the 3D refined FE model of the fully braced scheme are approximately 7.38 and 7.33 s, respectively. Similarly, the fundamental periods in the x and y directions of the half-braced scheme are 7.44 and 7.69 s, respectively. Therefore, the lateral stiffness values in the two orthogonal directions are highly similar for both schemes. The first torsion period is 2.77 s for the fully braced scheme and 3.42 s for the half-braced scheme. Both torsion-translational period ratios (0.38 and 0.44) are considerably smaller than the upper limitation of 0.85 specified in the Chinese design code (i.e., the Technical Specification for Concrete Structures for Tall Buildings, JGJ 3-2010 [34]). Thus, the torsion effects in both schemes are not significant. As a result, it is feasible to simplify the models using planar models instead of 3D models. Based on the above discussion, the models for the fully braced scheme and half-braced scheme are simplified into planar models in the fundamental transitional vibration plane.

(2) The simplified models only consider the mega columns, core tubes (including the shear walls and coupling beams), mega braces, and trusses (including the outriggers and perimetric trusses). Other components, such as secondary frames, are not included in these simplified

models. To validate the rationality of such simplifications, the stiffness of each type of component is reduced by 50% in the refined FE models, whereas the total mass and the properties of other components remain unchanged, to investigate the stiffness contribution of various components [32]. The first 9 vibrational periods of the models with reduced stiffness and those of the original models are compared in Table 2. A 50% reduction in the stiffness of the mega columns results in a clear change in the fundamental translational periods (approximately 15% for the fully braced scheme and 20% for the half-braced scheme) but a relatively small change in torsional periods, indicating that the mega columns make substantial contributions to the lateral stiffness of the building. When the stiffness of the core tubes is reduced, significant changes in the higher-order vibration periods are observed, with a maximum change of 21.15% for the fully braced scheme and 21.65% for the half-braced scheme. Moreover, the reduction in the core tube stiffness also has a great impact on the torsional periods: 15.47% and 18.86% increases for the fully braced scheme and half-braced scheme, respectively. These results indicate that the core tube stiffness significantly contributes to both the lateral and torsional stiffness of the building. When the stiffness of the mega braces and trusses is reduced, the change in period is a moderate change of less than or equal to 7.42%. A 50% reduction in the stiffness of the secondary frame results in a small change of less than or equal to 2.07%. Thus, the stiffness contribution of the secondary frame is negligible and can be ignored in the simplified model. Given the above comparisons, the mega columns, core tubes, mega braces, and trusses are regarded as primary components that must be properly considered in the simplified model.

(3) The nonlinear beam-column elements and nonlinear spring elements proposed by Lu et al. [32] are adopted to simulate the above primary components in the simplified model. The hysteretic model and corresponding parameter calibration method proposed by Lu et al. [32] are also adopted. The details of these models will be discussed in the following section.

Given the above-mentioned assumptions, the final simplified models of both schemes are established. Comparisons between the refined FE models and corresponding simplified models are presented in Figs. 4 and 5. Note that the number of elements in the simplified model decreases to approximately 1.68% for the fully braced scheme (i.e., 1,309 elements) and 2.80% for the half-braced scheme (i.e., 1,817 elements); thus, the computational workload will be significantly reduced.

3.2 Simplified models for the primary components

A nonlinear beam-column element is adopted to simulate the mega columns, mega braces, outriggers, and perimetric trusses. A typical layout of the core tubes with coupling beams and shear walls is shown in Fig. 6. The distributions of the coupling beams of both schemes are regular and mainly distributed symmetrically along the two orthogonal axes of the core tubes. Hence, the core tubes are divided into two identical sub-tubes by the coupling beams. As proposed by Lu et al. [32], the coupling beams (i.e., the coupling beams A and B in Fig. 6(a) and the coupling beams A to F in Fig. 6(b)) are simulated as one shear spring element. The two identical sub-tubes are simulated as two nonlinear beam-column elements. The rigid arms are used to connect the coupling beams and sub-tubes to consider the actual size of the core tube.

The hysteretic model proposed by Lu et al. [32, 35] (shown in Fig. 7) is adopted to simulate different components in the simplified model. The parameters in this model can be classified into two groups: (1) parameters for the backbone curve and (2) parameters for the hysteretic rule. The first group of parameters includes K0 (the initial stiffness), Fy (the generalized yield strength, e.g. axial force, shear force or bending moment), ⍰⍰(the hardening ratio), ⍰soft (the softening ratio), ⍰⍰(the ratio of peak strength and yield strength), and ⍰⍰(the ratio of reversed yield strength and yield strength), as shown in Fig.7. The second group of parameters includes ⍰⍰(representing the pinching effect), ⍰⍰(representing the position of the ending point of slip), C (dimensionless accumulated hysteretic energy dissipation parameter, which reflects the capacity of resisting strength degradation caused by the cyclic loading) and

The corresponding parameters are also determined using the methods proposed by Lu et al. [32]. For the steel components, such as
the mega braces and outriggers, all of the parameters in Fig. 7 are calibrated through the analysis of the corresponding refined FE models. In regard to other components, the backbone curve parameters are also obtained through the analysis of the corresponding refined FE

models. The hysteretic parameters are calibrated through the experimental data from similar specimens tested under cyclic loads, as proposed by Lu et al. [32], by considering the difficulties in simulating the hysteretic behavior of concrete components. The typical values of the hysteretic parameters for different components are summarized in Table 3.

4. VALIDATION OF THE SIMPLIFIED MODEL

Based on the analytical results of the refined FE MSC.Marc models for both schemes, modal analyses, static analyses, and nonlinear time history analyses are performed to validate the reliability of the simplified models. In addition, analyses using ETABS [36], a commercial software widely used for the design of structures, are also conducted here to further validate the reliability of the proposed simplified models.

The first six translational periods of the refined FE MSC.Marc models compared to those of the simplified models exhibit close agreement for both schemes as shown in Table 4, with a maximum relative error of approximately 5.57%. In addition, the first three translational periods of the refined FE ETABS models, which are usually used for the design of structures, also agreed well with those of the simplified models. Specifically, the relative errors for the fully braced model are -2.15%, -4.72% and 2.85%, respectively, and the relative errors for the half-braced model are 0.85%, 4.12% and 4.53%, respectively. The above comparisons indicate that the simplified model is capable of capturing the basic dynamic characteristics.

Subjected to gravity load only, the total axial forces in the mega columns and core tubes at the bottom of each zone are calculated and presented in Fig. 8. The axial forces in the mega columns and core tubes in the simplified models are shown to be in good agreement with those in the refined FE models for both schemes, thus validating the consistency of the mass distribution between these two types of models.

To further validate the reliability and rationality of the simplified models, time history analyses are performed for the simplified and refined FE models. The widely used El Centro EW 1940 record is selected as the typical ground motion input. Because ETABS is incapable of conducting nonlinear analysis for super-tall buildings, linear time history analyses are first conducted here. The peak ground acceleration (PGA) is scaled to 70 gal, which is the intensity of service level earthquake (SLE) for the site of this building. The comparisons of story drift ratio predicted by the simplified model and refined FE models for the fully braced scheme are compared in Fig. 9(a). Good agreement is observed, which validates the reliability of the proposed simplified model at the elastic stage. To validate the reliability of the proposed simplified model at the nonlinear stage, nonlinear time history analyses using simplified model and refined FE MSC.Marc model are conducted. The PGA is scaled to 400 gal, which is the intensity of the MCE for the site of this building. The comparisons of the time history analysis results are illustrated in Fig. 9(b-e). Fig. 9(b) shows the roof displacement histories of the fully braced scheme, which are nearly identical. The story displacement envelopes of the fully braced scheme shown in Fig. 9(c) are also in good agreement, with some negligible difference in the central zones. The story drift ratio envelopes of the fully braced scheme are also found to be in good agreement with the refined FE MSC.Marc model, as shown in Fig. 9(d). As a result, the time history analysis results predicted by the 2D simplified model are shown to be in good agreement with those of refined FE MSC.Marc models for the fully braced scheme. Similar findings are also obtained for the half-braced scheme. The story drift ratio envelopes of the half-braced scheme shown in Fig. 9(e) are found to be in good agreement. In particular, the simplified model of the half-braced scheme is capable of representing the inter-story drift sudden change due to the outriggers. The slight discrepancy between the predicted results of the simplified model and refined model found at the top zone of the building is considered to be acceptable, which conclusively validate the reliability of the proposed simplified model even at the nonlinear stage. Such a time history analysis using the simplified model takes only 18 min, whereas the analysis using the refined FE MSC.Marc model requires more than 36 h on the same computer, which is a 2.20 GHz Intel(R) Xeon(R) CPU E5-2430 and 48 GB of memory. The computational time is reduced by more than 100 times.

The above-described validations confirm that the simplified models of both schemes are capable of predicting the nonlinear story displacements and story drift ratio as well as the critical mechanical characteristics. This capability of the simplified models will reliably and effectively assist researchers and designers in understanding the seismic behaviors of different design schemes.

Note that the above-mentioned validation of the simplified models based on a comparison with the refined FE models is mainly for research purposes validating the accuracy of the simplified model. At the preliminary design stage, it is rather difficult and time consuming to create a refined FE model. Therefore, it is almost impossible to use the refined FE model to assist in designing at the preliminary design stage. In contrast, the simplified models developed in this study can be readily established based on the preliminary design information of the structural system and the primary components. As a result, the seismic behavior of the preliminary design schemes can be efficiently predicted through the linear and nonlinear analysis of this simplified model, thus facilitating the development of an improved design.

5. COMPARISON OF THE PLASTIC ENERGY DISSIPATION

To satisfy the demands for resilience, the earthquake-induced damage should be uniformly distributed in the replaceable components along the height of the building. Severe damage inside the key components and damage concentration along the height of the building should be avoided. The cumulative hysteretic energy dissipated by various structural components is commonly considered as a good indicator of earthquake-induced damage. Hence, the hysteretic energy dissipated by different components and the hysteretic energy distribution along the height of the two schemes are compared using the simplified models validated above.

To investigate the influence of the seismic intensity on the plastic energy dissipation distribution, three different seismic intensities are adopted: PGA = 220 gal (i.e., the MCE in the Intensity 7 Region), PGA = 310 gal (i.e., the MCE in the Intensity 7.5 Region) and PGA = 400 gal (i.e., the MCE in the Intensity 8 Region, which is the design intensity of this super-tall building) [33]. Moreover, due to the significant randomness in the input ground motion records, the seismic response obtained from a single specified ground motion record may result in a large deviation. Hence, 22 far-field ground motion records recommended by FEMA P695 [37] are adopted as the basic ground motion set. The plastic energy dissipation behaviors are discussed based on the mean value of the response parameters obtained from the 22 ground motions. Note that the simplified models offer a notable advantage in terms of computational efficiency. Although both the simplified models and refined FE models are established and used in this work, the nonlinear dynamic analyses of 22 ground motions under three different seismic intensities using the refined FE models will require more than 5,000 h, which is unacceptable for the design of a super-tall building. In contrast, using the simplified models analysis requires only approximately 40 h; such a reduced computational burden for the simulations will be beneficial for the comparison of different design schemes.

5.1 Plastic energy dissipation analysis of different components

Nonlinear time history analyses of the three above-mentioned seismic intensities and 22 ground motion records are performed for both schemes. The mean percentages of the plastic energy dissipation contribution of different structural components are presented in Table 5.

The coupling beams dissipate most of the plastic energy in both schemes (no less than 93.80% in the fully braced scheme and no less than 63.25% in the half-braced scheme), although the percentages slightly decrease with increased seismic intensity. Thus, the coupling beams are the primary plastic energy dissipation component in both schemes. Because the coupling beams are replaceable components, they can be designed to be readily replaced so that the super-tall building can be easily repaired after an earthquake [38], which satisfies the requirement for resiliency.

The perimetric trusses in the fully braced scheme dissipate approximately 5.07% of the total plastic energy when PGA = 400 gal, which means that moderate damage occurs in these elements. Note that the perimetric trusses bear a limited load when the building is subjected to the service load; as a result, they can be conveniently repaired or replaced after an earthquake [39, 40]. The plastic energy dissipation contribution of the outriggers and perimetric trusses in the half-braced scheme is considerably higher, with a value of up to 28.39%, because the outriggers bare significant shear forces due to the different patterns of deformation between the mega columns and core tubes. Both the outriggers and perimetric trusses can be designed to be repairable or replaceable after an earthquake to satisfy the requirement for resiliency.

The braces are normally regarded as replaceable components in conventional buildings [41]. However, the mega braces in this super-tall building are difficult to be replaced due to its extremely large weight (> 150 tons). The mega braces in the fully braced scheme basically remain elastic and dissipates little plastic energy (≤ 0.55%) when subjected to all three considered intensities. In contrast, the mega braces in the half-braced scheme dissipate 5.44% of the plastic energy when PGA = 400 gal. This result indicates that the mega braces in the half-braced scheme suffer moderate damage and should be repaired or replaced after an earthquake, which is rather difficult to implement.

The mega columns and core tubes are key components of the building that are difficult to repair or replace. The plastic energy dissipation contribution of these components in both schemes increase with increasing seismic intensities. The plastic energy dissipation percentage of the mega columns and core tubes are 0.02% and 0.56%, respectively, for the fully braced scheme when PGA = 400 gal. This result indicates that these key components in this design scheme remain elastic and dissipate little plastic energy, thereby resulting in minor damage and enabling immediate re-occupancy of this building without any repair after earthquakes. In contrast, the plastic energy dissipation ratio of the core tubes is up to 2.92% for the half-braced scheme when PGA = 400 gal, which means that a certain extent of damage occurs inside these core tubes, thus requiring repair, which will delay the functional recovery of the building.

As described above, the fully braced scheme is proven to be more effective in focusing the damage to occur at the readily replaceable components (i.e., coupling beams and perimetric trusses) instead of the key components that are more difficult to repair (i.e., mega columns, core tubes and mega braces), thus ensuring the functional resilience of super-tall buildings after strong earthquakes. Therefore, the fully braced scheme is a better choice than the half-braced scheme for the design of this super-tall building.

5.2 Total plastic energy dissipation distribution along the building height

The total plastic hysteretic energy dissipation distribution along the height of the building based on the half-braced scheme is illustrated in Fig. 10 and is named the "Total" curve. Because the lateral-force resisting system significantly changes at Zone 5 (i.e., the mega braces installed in the lower four zones no longer exist in the upper four zones), large deformations and higher plastic energy dissipation are present in the upper 4 zones of the building as opposed to the lower 4 zones. Furthermore, the dimensions of the cross section of each floor gradually decrease from Zone 5 to Zone 6 and subsequently gradually increase from Zone 6 to Zone 7, thus leading to more plastic energy dissipation in these zones compared to the lower 4 zones. As a result, these zones have the potential to suffer relatively severe damage and are rather difficult to repair after an earthquake.

The total plastic hysteretic energy dissipation distribution along the height of the fully braced scheme is illustrated in Fig. 11 and is also called the "Total" curve. Because there are only 6 stories in Zone 8, the plastic hysteretic energy dissipated in Zone 8 is combined with that in Zone 7 as a single zone (i.e., Zone 7). Compared to the half-braced scheme, a relatively uniform distribution of the total plastic energy is observed for the fully braced scheme. The following two characteristics of fully braced scheme account for this difference: (1) The mega braces are installed along the height of the entire building, which leads to a more uniform distribution of the structural stiffness compared to the half-braced scheme; (2) the strength-based design of the fully braced scheme is more reasonable; as a result, the degree of nonlinearity in each zone is approximately identical. Moreover, the maximum dissipated plastic energy in a single zone of the fully braced is only 50.84% of that of the half-braced scheme when PGA = 400 gal.

The above discussions illustrate that the fully braced scheme induces a more uniform plastic energy dissipation distribution compared to the half-braced scheme. In addition, the fully braced scheme avoids severe damage concentration in a single zone. All of these plastic energy dissipation characteristics improve the performance of the building in terms of resiliency and enable rapid recovery in the aftermath of a strong earthquake.

5.3 Plastic energy dissipation contribution of each component along the building height

The average plastic energy dissipations in different components along the height of the half-braced scheme is presented in Fig. 10 and are referred to as “Mega column”, “Core tube”, “Mega brace”, “Trusses”, and “Coupling beam” curves. Because the mega braces are only installed in the lower 4 zones of the building, the plastic energy dissipated in the mega braces is located in the lower 4 zones. In contrast, most of the plastic energy is dissipated by the core tube, trusses and coupling beams located in the upper 4 zones. Significant damage concentration in the coupling beams is observed in Zones 4, 5 and 7, which means that the half-braced scheme does not make full use of the energy dissipation capacity of all of the coupling beams along the height of the building.

The average plastic energy dissipation contribution in different components along the height of the fully braced scheme building is presented in Fig. 11. The trusses participated in the energy dissipation at the upper 3 zones of the building. The results clearly indicate that the primary plastic energy dissipation component is the coupling beams. In addition, the plastic energy dissipated by coupling beams is uniformly distributed along the height of the building, which avoids the concentration of damage.

As described above, the fully braced scheme induces a more uniform plastic energy dissipation in the coupling beams along the building height compared to the plastic energy dissipation of the half-braced scheme. The fully braced scheme makes full use of the energy dissipation capacity of the coupling beams in each zone of the building.

6. CONCLUSIONS

A 2D simplified model is developed for the seismic analysis of a super-tall mega-braced frame-core tube building and applied to compare two design schemes of this building in terms of resilience. The following conclusions can be drawn:

(1) Compared to the refined FE model, the proposed simplified model reduces the computational time by a factor of 100 while still providing reliable accuracy in predicting the seismic performances of the two considered design schemes.

(2) The plastic energy dissipation characteristics of both schemes are predicted and compared using the proposed simplified model. The fully braced scheme is found to induce a uniform

plastic energy dissipation distribution and effectively enables the energy dissipation to be located in readily replaceable components. In contrast, significant plastic energy dissipation concentration is observed at the upper 4 zones of the half-braced scheme, and the core tubes are found to suffer significant damage; as a result, the functional recovery of the building will be delayed. Overall, the fully braced scheme provides a better seismic resilient performance compared to the half-braced scheme.

The outcome of this study will provide guidance and act as a reference for the resilience-based earthquake design of super-tall buildings. The proposed simplified model can be used to compare various design schemes at the preliminary design stage.

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