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A simplified model for analysis of high-rise buildings equipped with hysteresis damped outriggers

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SUMMARY

A simplified model is developed to estimate the seismic response of high-rise buildings equipped with hysteresis damped outriggers. In the simplified model, the core tube is considered as a cantilever beam, and the effects of outriggers on the core tube are considered as concentrated moments. Modal decomposition method is adopted to obtain the seismic response of the simplified model. To investigate the accuracy and effectiveness of the simplified model, a high-rise building with a height of 160 m was adopted as the example structure, and its response subjected to a ground motion was analyzed using the simplified model. A corresponding finite element model was built and analyzed by a finite element program called SAP2000 (Computers and Structures, Inc. Berkeley, California, United States). The analysis results obtained from the two models were compared. To consider the randomness of the ground motion, comparisons between the two models were further conducted using another 22 ground motions. It is found that the analysis results obtained from the simplified model agree well with those obtained from the finite element model, and the computation time used for the simplified model is almost negligible compared to that used for the finite element model. Such observations demonstrate that the simplified model is accurate and effective.

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KEY WORDS: high-rise building; outrigger; hysteresis damper; simplified model; distribute parameter system

1. INTRODUCTION

Vibration control of high-rise buildings when subjected to large earthquakes is of great importance for the structure safety (Housner *et al*., 1997). Three control methodologies, passive control, semi-active control and active control, are currently available (Soong & Spencer, 2002). Among them, passive control is by far widely used mainly because of its stability and economical advantages. When applied to high-rise buildings, passive control is mainly to increase the damping ratio of the structures. It is reported that the damping ratio of the high-rise buildings becomes smaller with the increase of the height (Jeary, 1997). Therefore, energy dissipation devices, e.g. viscoelastic and viscous dampers, are commonly installed to increase the damping ratio of the high-rise buildings. For instance, viscous dampers are used in San Bernadino County Medical Center in California and Woodland Hotel (Woodland, CA) (Kareem & Kijewski, 1999), and more than 10 000 viscoelastic dampers were installed in the World Trade Center in New York (Smith & Willford, 2007). Both viscoelastic and viscous dampers are quite effective to provide damping effects to high-rise buildings. However, the high damping solid polymer in viscoelastic damper is frequency and temperature sensitive, and the

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damping fluid in viscous damper is easily leaked, leading to many concerns in the engineering community. Hysteresis damper, which is commonly made of metals, e.g. low-yield steel and lead, can also provide significant damping effects to the structures when sustaining large enough deformations (Skinner *et al*., 1974). In addition, compared to viscous and viscoelastic dampers, hysteresis dampers are commonly cheaper and more reliable in terms of durability (Eryaşar & Topkaya, 2010; Zhang *et al*., 2012; Deng *et al*., 2013; Marino 2013). Previous studies showed that the hysteretic damper could effectively reduce damage of structures during earthquakes (Kam *et al*., 2010; Oviedo *et al*., 2011). Therefore, the control of the seismic response of high-rise buildings with hysteresis dampers is expected to be more reliable, economical and effective. However, applications of hysteresis dampers in high-rise buildings are rarely reported.

A parametric study is important to obtain the optimum design of high-rise buildings, and especially useful to make the best use of dampers. Since parametric study commonly involves many rounds of analysis, and each round takes a lot of computation efforts if using sophisticated finite element models, simplified models are of great interest to engineers and researchers. Some simplified models have been developed for high-rise buildings equipped with viscous dampers. In these models, the high-rise buildings were commonly simplified as cantilever beams. Singh *et al*. proposed a cantilever beam with an axial viscous damper attached at its free end (Singh *et al*., 1989). Oliveto *et al*. conducted a case study of a high-rise building with two rotational viscous dampers installed at the ends of a simply supported beam. By considering the dampers as boundary conditions, a dynamic stiffness matrix was derived (Oliveto *et al*., 1997). Engelen *et al*. considered the first three modes of a beam model with a damper attached. By using a numerical method, the optimal damper size for a maximum system damping ratio was obtained (Engelen *et al*., 2007). Lee proposed an analytical model to analyse the deformations of high-rise wall-frame structure under lateral force. In this model, the shear and flexible deformations could be considered. A numerical model was built in MIDAS to check the accuracy of the simplified method (Lee *et al*., 2008). Chen *et al*. developed a simplified model with two viscous dampers attached at the ends of outriggers. A characteristic equation governing complex eigenvalues is derived for the model. By maximizing the damping ratio, the optimal position and damping coefficient of the damper were calculated (Chen *et al*., 2010). Rahgozar *et al*. provided a solution though solving partial differential equations (PDEs) of the cantilever beam model with two rotation springs (Rahgozar & Malekinejad, 2011). The natural frequencies and modes obtained from the cantilever beam model and another model built in SAP2000 were compared. The results obtained were close. Although a few simplified models were developed, a simplified model of high-rise buildings with hysteresis damper has not been investigated so far.

To this end, a simplified cantilever beam model was developed for high-rise buildings with hysteresis dampers installed at outrigger ends. In this model, the core tube was considered as a cantilever beam and the outriggers as rigid bodies. To obtain the mass and stiffness matrices of the system, the classic solutions of distribute parameter systems were adopted. The moment from outrigger was considered as a concentrated moment attached at the height of the outrigger. The simplified model was solved by a homemade MATLAB program (MATLAB, The MathWorks, Inc. Natick, Massachusetts, United States). To investigate the accuracy of the simplified model, a sophisticated model was built using a general finite element program called SAP2000 (SAP2000 Advanced 14.1.0,). Based on the comparison of the analysis results between the simplified and sophisticated models, the accuracy of the simplified model was calibrated. In this paper, the formulation of the simplified model is presented, and the accuracy and efficiency of the simplified model are reported.

2. HIGH-RISE BUILDING WITH HYSTERESIS DAMPED OUTRIGGER

The detailing of the high-rise building equipped with hysteresis damped outriggers is shown in Figure 1. The outrigger truss is connected to the exterior column through a slipping connection, which restricts the horizontal displacement but allows sliding in the vertical direction. The hysteresis damper is installed between the outrigger and column. During large earthquakes, there is a relative displacement between the outrigger trusses and columns, and the hysteresis damper would sustain a large deformation and dissipate energy.

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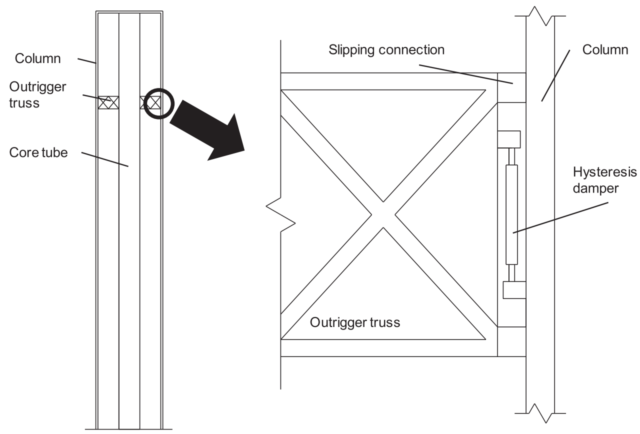

Figure 1. Structure of outrigger truss with hysteresis dampers.

3. SIMPLIFIED MODEL

In this part, the simplified model of the studied structure will be illustrated. As shown in Figure 2, the uniform cantilever beam with a flexible stiffness of EI has a length of H. E represents the young modulus of the material, and I is the moment inertia of the cross-section. The mass per unit length of the beam is m. In this paper, only one outrigger truss is considered. This truss with a length of r is installed at a height of a. The hysteresis dampers are installed at the ends of the outrigger. Considering that the stiffnesses of outriggers and exterior columns are commonly much larger than those of hysteresis dampers, they are assumed to be infinite. The initial stiffness of the damper is k, and its yield force is F_y. When the cantilever beam rotates at the height of a, the outrigger will transform it into a deformation of the hysteresis damper.

To calculate the displacement of the cantilever, the following coordinate system is adopted. The original point is at the fixed end of the cantilever beam. X coordinate is along the cantilever beam, and Y coordinate is perpendicular to the X coordinate. The cantilever beam is a Bernoulli–Euler beam, which cannot take the shear deformation into consideration. This beam is much simpler for solving the dynamic response and the error between Bernoulli–Euler beam, and Timoshenko beam is taken to be very small for high-rise buildings (Han *et al.*, 1999). The displacement of the cantilever beam can

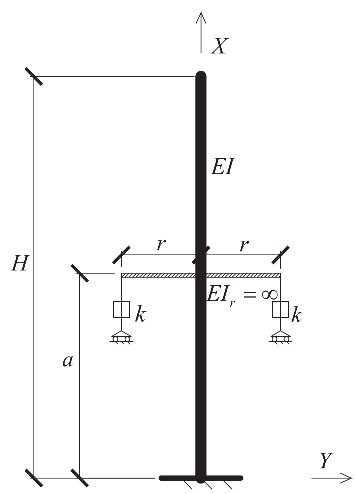

Figure 2. Cantilever beam model with hysteresis damper.

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be considered as a function of height and time shown in Equation (1), where x indicates the height, and t is the time.

$$y = y(x, t) \tag{1}$$

A cantilever beam without outrigger is first considered to simplify the problem. The classic equation of distribute parameter systems is shown in Equation (2). $f(x,t)$ indicates the external loading.

$$m\frac{\partial^2 y(x,t)}{\partial t^2} + EI\frac{\partial^4 y(x,t)}{\partial x^4} = f(x,t) \tag{2}$$

Through the mode decomposition method, the displacement of cantilever beam can be written as Equation (3).

$$y(x,t) = \sum_i \phi_i(x) q_i(t) \tag{3}$$

The natural frequencies and modes are given as Equations (4) to (6) (Clough & Penzien, 1995). ω_i indicates the i^th^ natural frequency, ϕ_i designates the i^th^ mode of the beam and λ_i is the coefficient. Here, $\lambda_1 = 1.875, \lambda_2 = 4.694, \lambda_3 = 7.855$ and when $i \geq 4$, $\lambda_i \approx (i - 1/2)\pi$.

$$\omega_i{}^2 = \lambda_i{}^2 \sqrt{\frac{EI}{mH^4}} \tag{4}$$

$$\phi_i(x) = \sin\lambda_i x - \sinh\lambda_i x + \widetilde{B}_i(\cosh\lambda_i x - \cos\lambda_i x) \tag{5}$$

$$\widetilde{B}_i = (\sin\lambda_i + \sinh\lambda_i)/(\cos\lambda_i + \cosh\lambda_i) \tag{6}$$

The mass and stiffness of the vibration mode can be calculated as Equation (7). For the convenience of formulation, ϕ_i is normalized to make M_i equals mH.

$$M_i = \int_0^H m\phi_i^2(x)dx, K_i = \omega_i^2 M_i \tag{7}$$

In what follows, the outrigger with a damper installed at the end is considered. The restoring force of the damper can be written as Equation (8). ηk indicates the post-yield stiffness of the damper.

$$F_r(t) = \begin{cases} kr\frac{\partial y(x,t)}{\partial x}|_{x=a} & kr\frac{\partial y(x,t)}{\partial x}|_{x=a}\langle F_y \\ F_y + \eta kr\left(\frac{\partial y(x,t)}{\partial x} - \frac{\partial y_y(x,t)}{\partial x}\right)|_{x=a} & kr\frac{\partial y(x,t)}{\partial x}|_{x=a} \geq F_y \end{cases} \tag{8}$$

The moment from the outrigger is $2rF_r(t)|_{x=a}$, and in the coordinate system, its loading can be written as Equation (9). $\delta(x-a)$ is the Dirac-delta function, which is defined as Equation (10).

$$M_d(x,t) = \delta(x-a)2rF_r(t) \tag{9}$$

$$\delta(x-a) = \begin{cases} \infty & x = a \\ 0 & x \neq a \end{cases} \tag{10}$$

Substituting Equations (3) and (8) into Equation (9), the moment from the outrigger is described as Equation (11) for the unyielding case. Furthermore, a yield coefficient η, which represents the ratio of post-yield stiffness to initial stiffness, is needed when considering yield cases.

$$M(x,t) = \delta(x-a)2kr^2 \sum_i \frac{\partial \phi_i(x)}{\partial x} q_i(t) \tag{11}$$

The modal force is expressed as Equation (12).

$$P_i(t) = \int_0^H \phi_i(x) f(x,t) dx \tag{12}$$

Substituting Equation (11) into Equation (12), the modal force from outrigger can be obtained as Equation (13).

$$F_{oi}(t) = \int_0^H \phi_i(x) \frac{\partial M}{\partial x} dx = \int_0^H \phi_i(x) \frac{2kr^2 \partial \delta(x-a) \sum_i \frac{\partial \phi_i(x)}{\partial x} q_i(t)}{\partial x} dx \tag{13}$$

In which, $\phi_i(x)$ indicates a modal vector, and the first derivative of the moment from the outrigger truss $\frac{\partial M}{\partial x}$ is the equivalent force corresponding to the modal vector. Taking advantage of the Dirac-delta function, Equation (13) can be written as Equation (14).

$$F_{oi} = 2kr^2 \frac{\partial \phi_i(x)}{\partial x} \sum_i \frac{\partial \phi_i(x)}{\partial x} q_i(t)|_{x=a} \tag{14}$$

The external loading and the mode force can be written as Equations (15) and (16), respectively, for an earthquake load, which is a uniform external load. s_i is the load coefficient of i$^{\text{th}}$ mode.

$$f(x,t) = -m\ddot{x}_g \tag{15}$$

$$f_i(t) = -\int_0^H \phi_i(x) m \ddot{x}_g(t) dx = -s_i m H \ddot{x}_g \tag{16}$$

The dynamic equation of each mode can be obtained as Equation(17). In the equation, the coefficient of each mode has been adjusted so that the modal mass of all the vibration modes equals mH.

$$\ddot{q}_i(t) + \omega_i{}^2 q_i(t) = -s_i \ddot{x}_g - \frac{2kr^2}{mH} \frac{\partial \phi_i(x)}{\partial x} \sum_i \frac{\partial \phi_i(x)}{\partial x} q_i(t) \tag{17}$$

Taking the first n modes into consideration, the system can be described as a third-order matrix equation shown in Equation (18). Note that a yield coefficient is needed for the stiffness if the damper yields.

$$[M][\ddot{q}(t)] + [K][q(t)] = -\ddot{x}_g[s] - \frac{2kr^2}{mH}\phi^{'}\phi^{'T}[q(t)] \tag{18}$$

Where ϕ', $[M]$ and $[K]$ are described as Equations (19), (20) and (21), respectively.

$$\phi^{'} = \left[\frac{\partial\phi_1(x)}{\partial x}, \frac{\partial\phi_2(x)}{\partial x}, \cdots, \frac{\partial\phi_n(x)}{\partial x}\right]^T |_{x=a} \tag{19}$$

$$[M] = \begin{bmatrix} 1 & & \\ & \ddots & \\ & & 1 \end{bmatrix} \tag{20}$$

$$[K] = \begin{bmatrix} \omega_1^2 & & \\ & \ddots & \\ & & \omega_n^2 \end{bmatrix} \tag{21}$$

In Equation (18), $\frac{2kr^2}{mH}\phi^{'}\phi^{'T}$ represents the contribution to the stiffness matrix of the damped outrigger. The outrigger significantly increases the stiffness of the system. Finally, a damping matrix is added to the equation as shown in Equation (22).

$$[M][\ddot{q}(t)] + [C][\dot{q}(t)] + [K][q(t)] = -\ddot{x}_g[s] - \frac{2kr^2}{mH}\phi^{'}\phi^{'T}[q(t)] \tag{22}$$

$[C]$ is described as follows. ζ_i indicates the damping ratio of the i^{th} mode. In the following analysis, ζ_i is taken to be 1%.

$$[C] = \begin{bmatrix} 2\zeta_1\omega_1 & & \\ & \ddots & \\ & & 2\zeta_n\omega_n \end{bmatrix}[M] \tag{23}$$

Note that analyses of a distribute parameter system commonly need to solve PDEs (Chen *et al.*, 2010), whereas Equation (22) is an algebraic equation. Compared to PDEs, algebraic equations are much easier to solve. Therefore, the simplified model significantly reduces the difficulty in solving the system. A homemade program is developed with MATLAB. Equation (22) is solved using Newmark-β integration algorithm, and the dynamic response of the system is easily obtained.

4. MODEL VERIFICATION

To investigate the accuracy and efficiency of the simplified model, a typical high-rise building is considered as an example. The height of the structure is 160 m. The thickness t of the shear wall is 400 mm, and its width b is 8 m. The material's Young's modulus is 30 GPa, Poisson's ratio is 0.2 and the density ρ is 2400 kg/m^3. An outrigger with a length of 8 m is equipped to the structure at the height of 130 m. In accordance with the assumption of rigid outrigger truss, the cross-sections of the truss members are relatively large. The cross-section area of both the upper and lower chords is 0.1 m^2 and that of the brace is 0.03 m^2. The outrigger truss has a height of 4 m. Two hysteresis dampers are installed at the two ends of the outriggers, respectively. The initial stiffness, yield force and hardening ratio of the dampers are 640 kN/mm, 1280 kN and 0.01 kN/mm, respectively.

As shown in Figure 3, the example structure is first modelled accurately by a general finite element program called SAP2000. The shear walls are modelled by shell elements, and the hysteresis dampers are modelled by link elements. The link element is configured to have a typical kinematical hardening behaviour, which is suitable for simulating the hysteresis dampers. In order to obtain precise enough results, a relatively fine mesh is adopted. The model built in SAP2000 has 5489 nodes and 5186 elements for the example structure.

The structure is then modelled by the simplified model developed in this study. The main parameters of the simplified model are calculated as follows.

$$EI = \frac{Eb^3t}{12} = \frac{3\times10^{10}\times8^3\times0.4}{12} = 5.376N\times m^2 \tag{24}$$

$$m = \rho bt = 7680kg/m \tag{25}$$

Modal analyses were conducted to investigate the accuracy of the simplified model, and the first three modes of models were calculated. The natural period of the first three modes of the sophisticated finite element model and that obtained from the proposed simplified model are compared in Table 1. The difference between simplified and SAP2000 models is very small. It is notable that the natural periods obtained from the sophisticated model are slightly larger than those obtained from the simplified model. The little difference is mainly because the shear deformation can be considered in the finite element model, whereas it is neglected in the simplified model. Figure 4 compares the first three modal shapes obtained from the two models. The modal shapes obtained from the two models also agree very well.

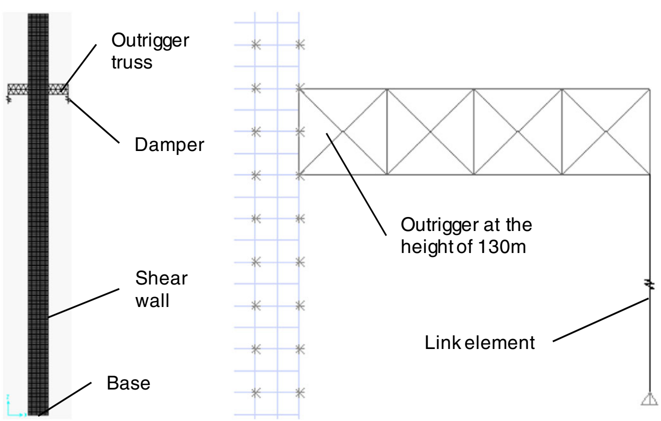

Figure 3. SAP2000 model.

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Table 1. Comparison on natural periods.

	SAP2000 (s)	Simplified model (s)	Error (%)
1st period	3.96	3.89	1.8
2nd period	0.77	0.75	2.7
3rd period	0.31	0.30	3.3

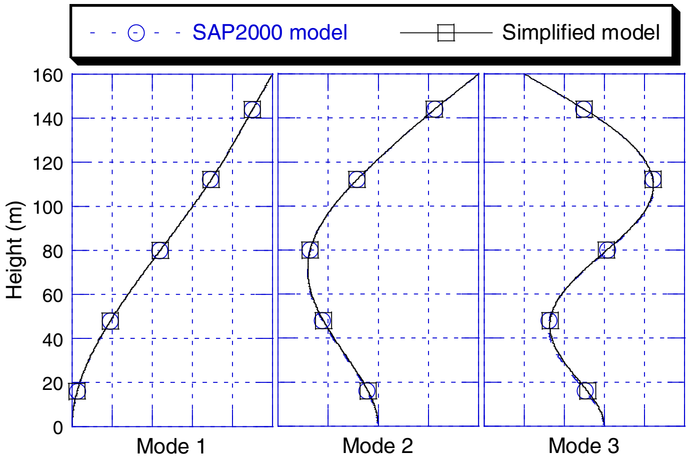


Figure 4. Modal comparison.

The equivalent stiffness of the yield damper changes with the time, thus the system is not a linear time-invariant system. The dynamic response cannot be calculated directly from the frequencies and modes because they change when the hysteresis dampers get into plasticity. To further investigate the effectiveness of the simplified model, nonlinear time history analyses were carried out. Newmark-β method was adopted for the analyses, and the NS component of the ground motion recorded in 1940 El Centro earthquake was used as input excitation. The acceleration history of the input ground motion is shown in Figure 5. The peak ground acceleration (PGA) of the record was adjusted to 220 Gal, which is for the maximum considered earthquakes of Intensity 7 according to the Chinese seismic design code. Furthermore, according to the code, the story drift of high-rise buildings is limited to 1/100 during the maximum considered earthquake, while in engineering practice, it is often less than 1/200 to ensure adequate safety margin (China Building Industry Publishing House, 2011). Recent research shows that high performance shear wall could remain elastic under such a deformation angle (Ji *et al*., 2013). Therefore, the cantilever beam was assumed to remain elastic in the simplified model.

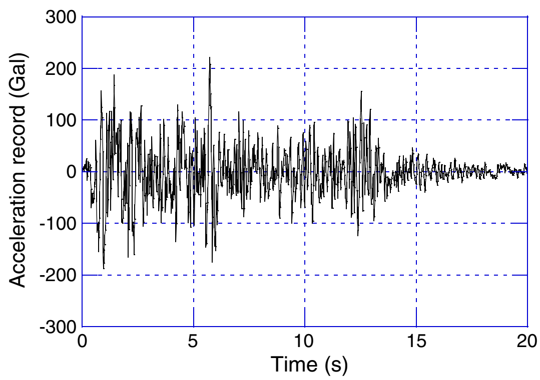


Figure 5. Acceleration history of El Centro.

Figure 6(a) compares the displacement of top floor between the two models. It can be observed that the results of the two models agree well. The maximum displacements are 0.2028 m and 0.1856 m for the simplified model and SAP2000 model, respectively. The relative error for top floor's displacement is 9.2%. Such a good agreement demonstrates that the simplified model has a satisfactory accuracy.

In the simplified model, the displacement of all floors can be calculated. For instance, the displacement of the 20th story at a height of 80 m can be obtained by using Equation (26).

$$u(80,t) = \sum_{i=1}^{n} \phi_i(80)q_i(t) \tag{26}$$

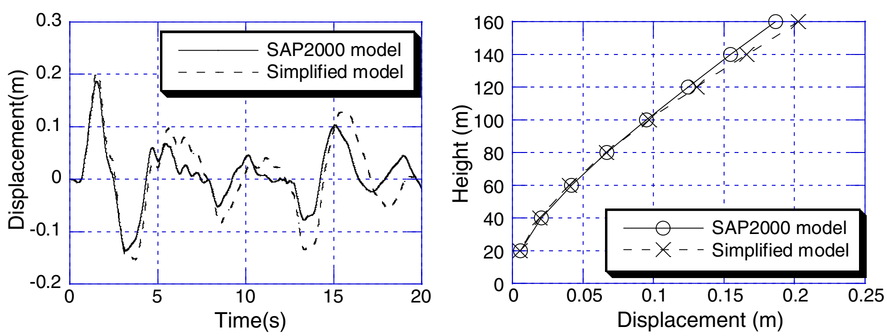
The comparison of the displacements of all stories is shown in Figure 6(b). The results from these two models are very close. Furthermore, once displacements of all floors are obtained, the story drift, and story shear of each story can also be calculated.

The hysteresis curve of the damper is shown in Figure 7. It can be observed that much energy is dissipated by the dampers.

SAP2000 took about 90 minutes to accomplish a 20-s time history analysis by using the sophisticated finite element model, while MATLAB program only took 0.156 s for the same time history by using the simplified model. Therefore, the simplified model is much more effective in term of computation efficiency.

In order to further verify the accuracy and efficiency of the simplified model, another 22 earthquake records from ATC63, which were designated as Eqr.1 to Eqr.22, were adopted (FEMA, 2008). The PGAs of these records were adjusted to 220 Gal, which is for the maximum considered earthquake of Intensity 7. The pseudo acceleration response spectrums of the 22 records are shown in Figure 8. The response spectrum for maximum considered earthquake of Intensity 7 prescribed in China seismic design code is also given in the figure. The duration and time interval of the records are 40 s and 0.01 s, respectively. The 22 records were applied to the same example structure. SAP2000 took about 66 hours to accomplish the time history analysis for the 22 earthquake records, while the simplified model just took 6.3 seconds for the same job. The efficiency of the simplified model is obvious.

The maximum displacements of all floors are obtained both from the simplified model and the SAP2000 model for 22 earthquake records. The displacements of all floors were estimated statistically as a 'median' and a 'standard deviation' (SD). Here, median refers to the mean of all data. SD refers to the standard deviation of all data and approximately equals the coefficient of variation (Marino & Nakashima, 2006). Figure 9(a) shows the median and SD of maximum story displacements for the 22 earthquake records. Both the median and SD of each floor obtained from the simplified model are very close to those from SAP2000 model. To further compare the simplified model SAP2000 models, the ratios of maximum displacements obtained from the sophisticated finite element model



(a) Top floor displacement (b) Maximum floor displacement

Figure 6. Comparisons on displacement between two models: (a) top floor displacement; (b) maximum floor displacement.

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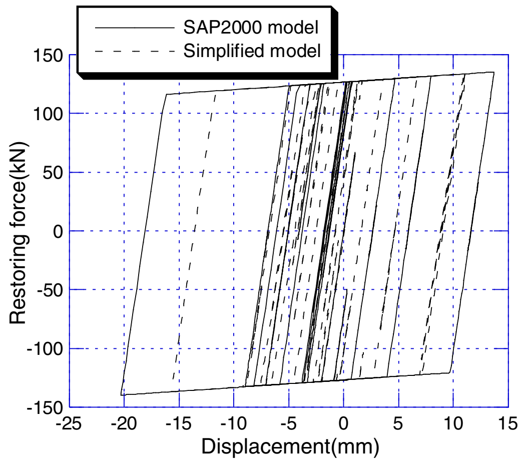

Figure 7. Hysteresis curves of hysteresis dampers.

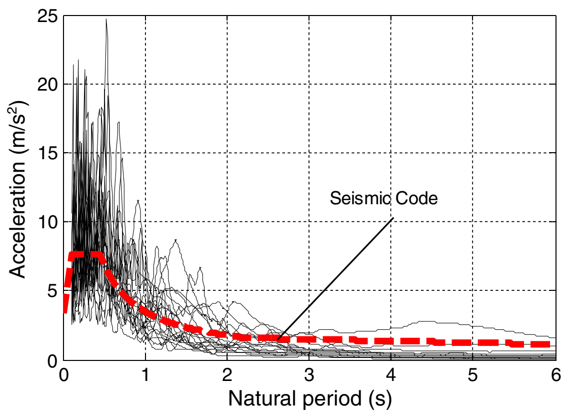

Figure 8. Acceleration response spectrums of 22 records.

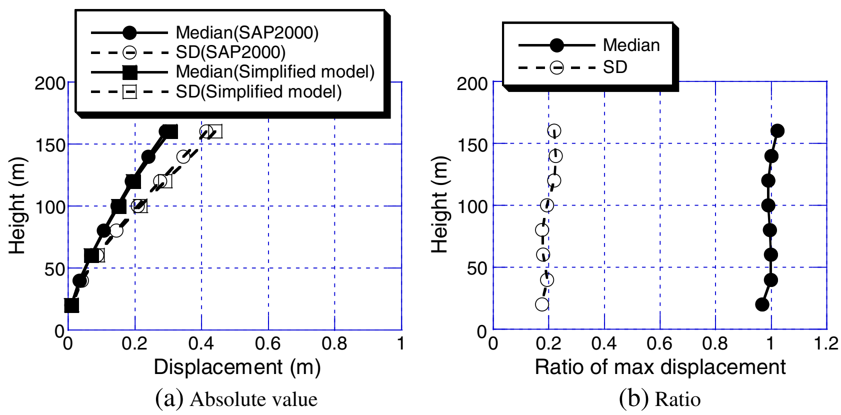

Figure 9. Comparisons of maximum floor displacements (220 Gal): (a) absolute value; (b) ratio.

to those obtained from the simplified model are calculated. The medians and SDs of the ratios are shown in Figure 9(b). It is found that the medians of the ratios are very close to 1.0 for all the stories, while the SDs are about 20%.

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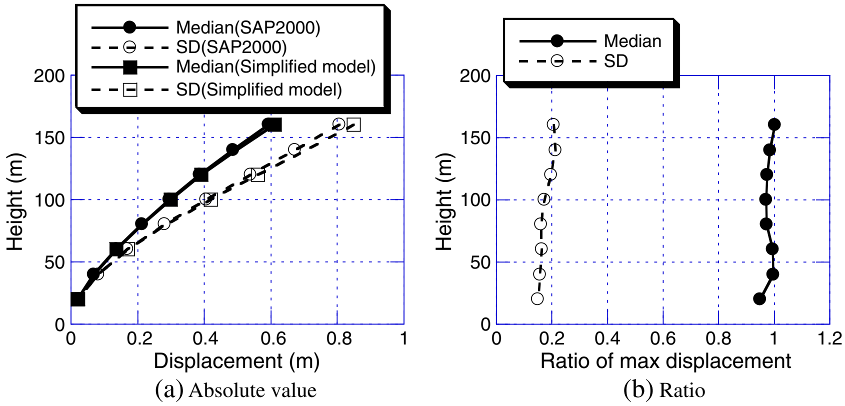

Figure 10. Comparisons of maximum floor displacements (400 Gal): (a) absolute value; (b) ratio.

The dampers are expected to exhibit stronger nonlinearity when subjected to larger ground motions. Thus, ground motions with higher intensity were adopted to investigate the applicability of the simplified model to structures whose dampers have higher level of plasticity. According to the Chinese code for seismic design of building, the PGAs of 22 earthquake records were adjusted 400 Gal, which is for the maximum considered earthquake of Intensity 8. Analyses were repeated for the 22 earthquake records by using both the SAP2000 model and the simplified model. The results obtained from the SAP2000 model and from the simplified model are compared in Figure 10. Similarly, Figure 10(a, b) is the absolute values and ratios for the maximum floor displacement.

In both Figures 9(b) and 10(b), the SDs are much less than the medians for all the stories, indicating that the results for the two models are consistent within the variation of the earthquake records. The good agreement between the two models shows that the simplified model can be applied to the seismic analysis of earthquakes having different intensity levels. These observations demonstrate that the accuracy of the simplified model is acceptable.

5. CONCLUSIONS

In this paper, a simplified model of high-rise buildings with hysteresis damped outrigger is proposed. The main feature of the system is the outrigger truss equipped with hysteresis dampers at their ends. The calculation of the structure response is based on the classic solutions of distribute parameter systems, which can avoid solving the PDEs. A sophisticated finite element model is built in SAP2000 to check the accuracy of the simplified model. The comparison on the responses between the simplified model and SAP2000 model shows that the simplified model has a high accuracy. Major conclusions obtained in this study are as follows:

1) A simplified model is proposed to obtain the response of high-rise building with damped outrigger truss when subjected to earthquakes. In the model, the contribution of outrigger equipped with hysteresis damper can be considered through a Dirac-delta function. The model can be easily solved based on the classic solution of distribute parameter system, in which only algebraic equations need to be solved, and solving PDEs can be avoided.
2) Through the simplified model, the natural frequencies and modes as well as the displacements response under an earthquake can be easily obtained. The results obtained from the simplified model agree well with those obtained from SAP2000 model, demonstrating a satisfactory accuracy of the simplified model.
3) For a time history analysis, the computation time needed for the simplified model is negligible compared to that needed for the sophisticated finite element model, demonstrating that the simplified model is very effective in term of computation time.

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4) The simplified model provides an effective and accurate solution for time history analysis on high-rise building with hysteresis damped outriggers. However, because the simplified model is based on Euler–Bernoulli beam, it is not necessarily applicable to the high-rise buildings whose shear deformation is not negligible.

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