Comparison and Selection of Ground Motion Intensity Measures for Seismic Design of Super High-Rise Buildings

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Abstract: Ground motion intensity measures (*IMs*) are an important basis of structural seismic design. Extensive research has been conducted on the selection of *IMs* for building structures with fundamental periods shorter than 6 s. However, minimal work has been performed for super high-rise buildings whose fundamental periods are much longer than 6 s. To fill the gap in this research area, this paper aims to develop a simplified analytical model for super high-rise buildings based on the flexural-shear coupled beam model. The variation in correlation and dispersion between different *IMs* is evaluated and the structural seismic response demand measures (*DMs*) are analyzed for different structural fundamental periods. Subsequently, rational *IMs* for the seismic design of super high-rise buildings are suggested. In addition, the influence of different flexural and shear stiffness ratios, α_0 , on the selection of an *IM* for super high-rise buildings is also discussed. Finally, a series of incremental dynamic analyses (IDA) of the Shanghai Tower, with a total height of approximately 632 m, is performed to verify the rationality of using different *IMs* for super high-rise buildings. The numerical results indicate that with a minimum dispersion, the peak ground velocity (PGV) has a better correlation to the story drift ratio than any other *IMs*. While considering structural nonlinearity, PGV still yields the minimum coefficient of variation in the collapse analysis of the actual super high-rise building. It is therefore recommended that PGV be used as an *IM* for the seismic design of super high-rise buildings.

Key words: super high-rise building, collapse analysis, ground motion intensity measure, simplified model.

1. INTRODUCTION

With the advancement of structural materials and construction technology, super high-rise building construction has entered into a new period of vigorous development since the completion of the first super high-rise building taller than 500 m-Taipei 101 in 2004. The 2012 statistical data from the Council on Tall Buildings and Urban Habitats (CTBUH) (<http://buildingdb.ctbuh.org/>) indicates that by the end of 2012, a total of thirteen super high-rise buildings of

500 m or taller have been constructed or under construction worldwide. This necessitates further studies to understand the seismic performance of such super high-rise buildings.

One of the most important issues in seismic design is the ground motion intensity measure, which relates the seismic hazards to the structural responses. Selection of a sufficient and efficient *IM* can effectively reduce the deviations of the structural responses. In view of this, extensive research has been performed worldwide

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through which a number of *IMs* have been proposed (Tothong and Luco 2007; Cordova *et al.* 2001; Luco and Cornell 2007; Baker and Cornell 2005; Vamvatsikos and Cornell 2005; Ye *et al.* 2013). However, the application of these proposed *IMs* in seismic analysis and design has largely been based on normal structures whose fundamental periods are shorter than 6 s and are mostly in the range of 0–4 s. On the other hand, little work has been reported with regard to *IMs* for super high-rise buildings. One of the most distinguished features of super high-rise buildings is that their fundamental periods are much longer than those of the low- to medium-rise frame or shear wall buildings. For example, the fundamental periods of Shanghai Tower and Ping-An Finance Center, two super high-rise buildings still under construction in China, will be larger than 9.0 s (Lu *et al.* 2011; Yang *et al.* 2011). More importantly, the structural displacement responses of super high-rise buildings are not governed by the fundamental vibration mode. Instead, the higher-order vibration modes contribute more significantly to the displacement responses. As such, a systematic evaluation of the rationality of the existing *IMs* for seismic design of super high-rise buildings is both necessary and significant for engineering practice.

In this study, a simplified model combining a flexural and a shear beam is developed based on the mechanical properties of an actual super high-rise building—the Shanghai Tower. Time-history analysis is also implemented to discuss the correlations between some existing *IMs* and structural seismic response demand measures (*DMs*), as well as their corresponding dispersions. A total of 22 pairs of far-field ground motion records suggested by FEMA P695 (FEMA 2009) are adopted for the time-history analysis. Subsequently, rational *IMs* for the seismic design of super high-rise buildings are suggested. Finally, the rationality of the recommended *IMs* when considering structural nonlinearity is verified via a series of incremental dynamic collapse analyses of the Shanghai Tower.

2. EXISTING INTENSITY MEASURES

Published literature (Tothong and Luco 2007; Cordova *et al.* 2001; Luco and Cornell 2007; Baker and Cornell 2005; Vamvatsikos and Cornell 2005; Ye *et al.* 2013) indicates that the existing *IMs* can be classified into two types: scalar- and vector-valued. Scalar *IMs* can be further categorized into single-parameter and multi-parameter *IMs* according to the number of parameters required.

The traditional peak values of ground motion records such as PGA, PGV and PGD are typical single-

parameter scalar *IMs*. These *IMs* are still widely used in many national design codes because of their simplicity (MOC 2010; The Building Centre of Japan 2000). Some recent literature indicates that PGA or PGV *IMs*, which only consider the characteristics of ground motions, do not perform well in predicting structural responses within short to medium periods. Therefore, the 5%-damped elastic spectral acceleration $S_a(T_1)$ at fundamental period T_1 (Shome *et al.* 1998) has been widely used for these periods. $S_a(T_1)$ covers not only the characteristics of ground motions but also the dynamic features of the structures. Compared with PGA, the dispersion of the predicted structural responses based on $S_a(T_1)$ is remarkably reduced (Ye *et al.* 2013), particularly for first-mode-dominated, moderate period structures. Despite these advancements, $S_a(T_1)$ only deals with the characteristics of the elastic fundamental period of the structure. When progressing into the nonlinear stage, the fundamental period of a structure elongates gradually, leading to changes in the structural dynamic characteristics. Furthermore, for high-rise buildings, higher-order vibration modes greatly influence the structural responses. All these factors however were not considered in the $S_a(T_1)$ formulation (Lucchini *et al.* 2011).

To overcome the aforementioned limitations, various attempts have been made to improve $S_a(T_1)$ by considering the effect of period elongation and the contribution of the higher modes (Tothong and Luco 2007; Cordova *et al.* 2001; Vamvatsikos and Cornell 2005). For example, Cordova *et al.* (2001) proposed an *IM* called S_a^* , which is based on the fundamental period T_1 and the longer period T_f representing the inelastic (damaged) structure S_a^* , also taking into consideration the period elongation in the nonlinear stage, as expressed by Eqn 1 below:

$$S_a^* = S_a(T_1) \left(\frac{S_a(T_f)}{S_a(T_1)} \right)^\alpha, \quad T_f = c \cdot T_1, \quad (1)$$

where α and c are two parameters to be calibrated.

Note that the dynamic behavior of buildings upon entering into the nonlinear stage is not fully understood, particularly at the early stage of structural design because of the difficulties in accurately predicting the nonlinear behavior of the building. Consequently, *IMs* that consider period elongation are limited in scope and practical application.

With respect to the higher vibration modes, the *IM* proposed by Vamvatsikos and Cornell (2005) also considers the participation of higher-order vibration modes. In the expression $IM = S_a(\tau_a, 5\%)^{1-\beta} S_a(\tau_b, 5\%)^\beta$,

the values of τ_a and τ_b represent the higher modal periods of concern.

Similarly, Luco and Cornell (2007) proposed an IM called $IM_{1E\&2E}$ based on the first two vibration modes and the square-root-of-sum-of-squares (SRSS) rule of modal combination. This IM considers the effect of the contribution of higher modes caused by the near-fault pulse, as expressed in Eqn 2.

$$IM_{1E\&2E} = \sqrt{\left[PF_1^{[2]}S_d(T_1, \zeta_1)\right]^2 + \left[PF_2^{[2]}S_d(T_2, \zeta_2)\right]^2} \\ = \sqrt{1 + R_{2E/1E}^2} \left| \frac{PF_1^{[2]}}{PF_1^{[1]}} \right| PF_1^{[1]} S_d(T_1, \zeta_1), \quad (2)$$

where $R_{2E/1E} = \frac{PF_2^{[2]}S_d(T_2, \zeta_2)}{PF_1^{[2]}S_d(T_1, \zeta_1)}$ and $S_d(T_1, \zeta_1)$ and

$S_d(T_2, \zeta_2)$ are the elastic spectral displacements, respectively, at damping ratios ζ_1 and ζ_2 with corresponding periods T_1 and T_2 . Eqn 2 also defines the j^{th} – mode participation factor for the story drift ratio of the i^{th} story, θ_i , as $PF_j(\theta_i) = \Gamma_j \frac{\phi_{j,i} - \phi_{j,i-1}}{h_i}$. Note that

$PF_1^{[2]}$ denotes the first mode participation factor for the maximum peak inter-story drift angle, $\theta_{\max}$, obtained using the SRSS of the first two modes. Similarly, $PF_1^{[1]}$ represents the first-mode participation factor for $\theta_{\max}$ when only the first mode is considered. The coefficient $\sqrt{1 + R_{2E/1E}^2}$ reflects the effect of the second mode on $\theta_{\max}$ and the relevant spectral shape. The coefficient $\left| PF_1^{[2]} / PF_1^{[1]} \right|$ mainly indicates the influence of the first-mode and first-two-modes estimations on $\theta_{\max}$, which may occur on different stories. The earlier proposed $IM_{1E\&2E}$ has also been improved by the same researchers (Luco and Cornell 2007), where the inelastic spectral displacement $S_d'(T_1, \zeta_1, d_y)$ was adopted as a replacement for the elastic spectral displacement $S_d(T_1, \zeta_1)$. The new IM is called $IM_{1I\&2E}$. Despite the advancement of $IM_{1E\&2E}$ and $IM_{1I\&2E}$, their complex expressions render the various parameters difficult to be determined, especially at the early design stage, thus limiting practical application of these IM s.

In addition to the aforementioned scalar IM s, some vector-valued IM s have also been proposed in recent years. For example, Baker and Cornell (2005) proposed a two-parameter vector-valued IM called $\langle S_d(T_1), \varepsilon \rangle$. The definition of parameter ε is originated from seismology engineering. This parameter reflects not only the distinction of the spectral shapes but also, to some extent, the effect of higher-mode contribution and period elongation. However, due to the absence of the

basic data required to obtain ε in many countries, more work must be performed before the vector-valued $IM \langle S_d(T_1), \varepsilon \rangle$ can be applied worldwide.

Ye *et al.* (2013) compared the applicability of 18 commonly used IM s in an elasto-plastic analysis of different buildings. The findings indicated that $S_d(T_1)$ has the best correlation with the structural demand measures (DM s) for short- and moderate-period structures. In addition, the correlation between PGV and the DM s is also acceptable. Furthermore, PGV performs better as an IM when the fundamental period of the structure increases. Based on their study, $S_d(T_1)$ and PGV have been recommended for use as the IM s in actual structural design.

Existing research on IM s as outlined above has mainly targeted structures whose fundamental periods are shorter than 6 s and mostly in the range of 0 to 4 s. It is evident that the rationality of the existing IM s requires further validation for super high-rise buildings with fundamental periods as long as approximately 10 s. This validation is the focus of the present study, with an emphasis on the actual requirements of structural design. A total of six different IM s (PGA, PGV, PGD, $S_d(T_1)$, $S_a(T_1)$, and $IM_{1E\&2E}$) are evaluated and their rationalities are discussed. PGV and $S_d(T_1)$ were selected based on the recommendations of Ye *et al.* (2013). To consider the contribution of the higher-order vibration modes of super high-rise buildings, $IM_{1E\&2E}$ was also selected. In addition, PGA, PGD and $S_d(T_1)$ were considered because they are widely used in actual design. The correlation between the IM s and DM s with different vibration periods is analyzed, and rational IM s for super high-rise building design are recommended.

3. ANALYTICAL MODEL AND GROUND MOTION SELECTION

3.1. Simplified Analytical Model

This study mainly focuses on super high-rise buildings with very long fundamental vibration periods. In general, the commonly used lateral seismic-resistant super high-rise systems include framed tube, bundled tube, tube-in-tube, diagonalized (trussed tubes, diagrids or braced frames), core and outrigger (central lateral system linking to the perimeter system through outriggers) and hybrid systems (Lu *et al.* 2011). The deformation modes of these structural systems are typically a combination of flexure and shear, which differs greatly from the traditional frame or shear wall buildings. This implies that the traditional multistoried shear model and cantilever flexural beam model are not suitable for super high-rise buildings. Consequently, the dynamic properties and seismic

responses of super high-rise buildings are approximated herein by using an equivalent continuum model consisting of a shear and a flexural cantilever beam, which is proposed by Miranda and Taghavi (2005) (Figure 1). The shear and flexural cantilever beams are connected by an infinite number of rigid links to achieve horizontal deformation compatibility at the same height. The response of this model subjected to ground acceleration can be expressed in the following partial differential equation:

$$\begin{aligned} \frac{m(x)}{EI_0} \frac{\partial^2 u(x,t)}{\partial t^2} + \frac{c(x)}{EI_0} \frac{\partial u(x,t)}{\partial t} + \frac{1}{H^4} \frac{\partial^2}{\partial x^2} \left(S(x) \frac{\partial^2 u(x,t)}{\partial x^2} \right) - \frac{\alpha_0^2}{H^4} \frac{\partial}{\partial x} \left(S(x) \frac{\partial u(x,t)}{\partial x} \right) \\ = - \frac{m(x)}{EI_0} \frac{\partial^2 u_g(t)}{\partial t^2}, \end{aligned} \quad (5)$$

where $u(x,t)$ is the horizontal displacement at non-dimensional height x at time t ; $m(x)$ is the mass per unit length of the model; x varies between 0 at the base of the building and 1 at the roof level; $u_g(t)$ represents ground displacement at time t ; H denotes the total height of the model; and $EI(x)$ and $GA(x)$ are, respectively, the flexural stiffness of the flexural beam and the shear stiffness of the shear beam at non-dimensional height x . Miranda and Taghavi (2005) assumed that the variations in the shear and flexural stiffness along the height are identical, i.e., $S(x)$ or

$$EI(x) = EI_0 S(x) \quad (6)$$

$$GA(x) = GA_0 S(x) \quad (7)$$

in which, $S(x)$ is the dimensionless function that defines the variation of stiffness along the height of the simplified model, and EI_0 and GA_0 are, respectively, the flexural and shear stiffness at the base of the model. The dimensionless parameter $\alpha_0 = H(GA_0/EI_0)^{1/2}$ controls the proportions of the flexural and shear deformations in the simplified model and thus the lateral deformation shape of the building. Theoretically, when $\alpha_0 = 0$, the model degenerates into a pure bending model, and when $\alpha_0 = \infty$ it degenerates into a pure shear model. Miranda and Taghavi (2005) indicated that α_0 is normally between 0 and 1.5 for shear wall buildings; between 1.5 to 5 for buildings with dual structural systems consisting of a combination of moment-resisting frames and shear walls; and between 5 and 20 for moment-resisting frame buildings. In addition, when $\alpha_0 \geq 30$, the deformation mode is dominated by shear deformations. A number of different buildings were analyzed by Miranda and Taghavi (2005) to verify the appropriateness of the model in simulating the elastic vibration of different buildings.

To ensure the appropriateness of the simplified model, the 632 m super high-rise building Shanghai Tower (see Figure 2) is selected herein to calibrate the key parameters used in the model. The Shanghai Tower

Figure 1. The simplified model consisting a flexural and a shear beam

Figure 2. The FE model of Shanghai Tower

has a hybrid lateral seismic-resistant system called the “mega-column/core-tube/outrigger-truss”. According to architecture and functional requirements, eight reinforced stories were built for every 13 to 16 stories, which divide the entire structure into eight zones. The plane layouts of these eight stories are also briefly illustrated in Figure 2. The fundamental period of the Shanghai Tower is nearly 9.83 s (Lu *et al.* 2011), which is far beyond the range of 6 s specified in the design response spectrum of the Chinese Code for the Seismic Design of Buildings (MOC 2010).

The parameters of the simplified model are calibrated such that the fundamental dynamic properties of the simplified model approach those of the Shanghai Tower in the XZ plane. The total gravitational load of the simplified model, which is determined from a combination of $1.0 D + 0.5 L$, is identical to that of the Shanghai Tower, in which D is the dead loads, including the effects of self weight and permanently attached equipment and fixtures; L is the live loads. Note that the dead and live loads of the Shanghai Tower have little variation along its height; the simplified model thus has a constant mass distribution along the height. According to the stiffness distribution of the Shanghai Tower, a quadratic curve, i.e., $S(x) = 1 - (1 - 0.55)x^2$, is adopted to model the structural stiffness along the height (Eqns 6 and 7). The flexural stiffness at the base, EI_0 , and the dimensionless flexural and shear ratio, α_0 , are calibrated according to the dynamic properties of the Shanghai Tower.

The values of the main parameters used in the simplified model are summarized in Table 1. The first five free translational vibration periods and the relative deviation Δ between the simplified model and the fine FE model of Shanghai Tower are compared in Table 2. The relative deviation Δ , defined by Eqn 8, is adopted to describe the distinction between the two models. In the equation, T_{simplify} and T_{fine} represent, respectively, the free translational vibration periods of the

simplified model and the fine FE model of Shanghai Tower.

$$\Delta = (T_{\text{simplify}} - T_{\text{fine}}) / T_{\text{fine}} \quad (8)$$

It is evident that some discrepancies are present in the comparison. Note that a substantial reduction in computational costs is achieved by employing the simplified model compared with a very fine FE model. Note also that the focus of this study is to offer a general rule for super high-rise buildings instead of investigating the characteristics of a particular structure. Therefore, the resulting discrepancies are considered acceptable.

3.2. Ground Motion Selection

Due to significant randomness in the ground motion records, the structural responses may vary significantly in a time-history analysis. To minimize such a variation while maintaining inherent randomness of the ground motions, 22 pairs of far-field ground motion records recommended by the FEMA P695 (2009) are adopted herein as the basis ground motion record set. Note that two horizontal components of each ground motion are considered, thereby resulting in 44 natural records. Their corresponding elastic acceleration spectrums at a 5% damping ratio are presented in Figure 3. This ground motion record set not only maintains the randomness of the ground motions themselves but also leads to a certain degree of consistency in the structural responses.

4. EVALUATION OF THE INTENSITY MEASURES

4.1. Evaluation Method

The existing literature (Baker and Cornell 2005, 2008; Padgett *et al.* 2008) indicates that the relationship between the structural seismic response demand measures (DMs) and the ground motion intensity

Table 1. Main parameters used in the simplified model

Gravity load (ton)	EI_0 (N·m ²)	α_0	Damping ratio ζ
6.88×10^8	2.4×10^{14}	4.0	5%

Table 2. Comparison of the first five periods between the simplified model and the fine FE model

	T_1	T_2	T_3	T_4	T_5
The fine FE model (s)	9.83	3.57	1.67	1.02	0.72
The simplified model (s)	9.85	3.25	1.71	1.05	0.70
Relative deviation Δ	-0.23%	-8.9%	2.87%	3.18%	-2.60%

Figure 3. The elastic acceleration spectrum of the 44 far-field ground motions at a 5% damping ratio

measures (IMs) can be estimated using the power-function model presented in Eqn 9:

$$DM = a \cdot IM^b, \quad (9)$$

in which a and b are the regression coefficients. The equation can be rearranged in a linear regression form of logarithms of DM and IM ; i.e.,

$$\ln(DM) = \ln a + b \ln(IM) \quad (10)$$

Eqn 10 clearly satisfies the classical linear regression model. The correlation coefficient ρ between $\ln(DM)$ and $\ln(IM)$ and the corresponding dispersion β can then be obtained by regression analysis of n discrete data points (DM_i, IM_i) . This analysis can be performed using the least square method in time-history analysis. The value of dispersion β is calculated from Eqn 11 below:

$$\beta = \sqrt{\frac{\sum_{i=1}^n (\ln(DM_i) - \ln(aIM_i^b))^2}{n-2}} \quad (11)$$

The correlation coefficient ρ ranges between -1 and 1 . The closer to 1 $|\rho|$ is, the better the correlation between the calculated DMs and IMs . In general, $|\rho| \geq 0.8$ implies a good correlation. In addition, the smaller the dispersion β , the better the IMs are.

Note that all modern-day super high-rise buildings are rigorously designed with sufficient safety margins to meet the seismic requirement under the Maximum Considered Earthquake (MCE) specified in the design codes. Thus, the major lateral force resisting components remain elastic or slight yielding, instead of exhibiting obvious nonlinear behavior, when subjected to MCE. Therefore, elastic time history analysis is considered appropriate in evaluating the rationality of the existing IMs for super high-rise buildings.

The specific research procedures are as follows:

- (1) For a simplified model with a given fundamental period and corresponding parameters, calculate the maximum structural response DM_i subjected to the i^{th} ground motion record using the time-history analysis method;
- (2) Calculate the IM_i of the corresponding i^{th} ground motion record;
- (3) Calculate n ground motion records and obtain n discrete data points (IM_i, DM_i) and plot them in the $\ln(IM)$ - $\ln(DM)$ coordinate system. The correlation coefficient ρ between $\ln(IM)$ and $\ln(DM)$ for the given fundamental period and the corresponding dispersion β are obtained by

Figure 4. The schematic diagram of correlation between the IM and DM

linear regression. The schematic diagram of correlation between the IM and DM is shown in Figure 4;

- (4) Adjust the model parameters (EI_0 or $\rho(x)$) to obtain a series of structures with different fundamental periods. Next, repeat steps 1 to 3 to calculate the variations in the correlation coefficient between DM_i and IM_i and in the corresponding dispersion for different structural fundamental periods. Note that although only EI_0 or $\rho(x)$ is modified in this step, the influence of adjusting other parameters will be discussed in later sections of this work.

While many DMs are available, such as the maximum top displacement ($d_{\max}$), maximum acceleration ($a_{\max}$), maximum story drift ratio ($\theta_{\max}$), maximum base shear force ($F_{\max}$), and total input energy (E_{input}), the most commonly used DMs in structural seismic design and seismic response analysis are $\theta_{\max}$, $d_{\max}$ and $a_{\max}$. Therefore, this paper aims to discuss the variations in the correlation coefficients between these DM_i ($\theta_{\max}$, $d_{\max}$, $a_{\max}$) and IM_i and their corresponding dispersion with different structural fundamental periods T_1 .

4.2. Result Analysis

Based on the simplified model with $\alpha_0 = 4$ proposed in the above section in Table 1, five different simplified models with the fundamental periods of 1, 3, 6, 9 and 10 s respectively are established by adjusting the values of EI_0 and $\rho(x)$. In these models, classical Rayleigh damping is adopted with a value of 5%. Figures 5 to 10 demonstrate the variations in correlation between the DMs of interest ($\theta_{\max}$, $d_{\max}$, $a_{\max}$) and the IMs (PGA, PGV, PGD, $S_a(T_1)$, $S_d(T_1)$, and $IM_{1E\&2E}$) and their corresponding dispersion for different fundamental periods.

Figure 5. The variations of correlation between $\theta_{\max}$ and IMs at different fundamental periods

Figure 5 demonstrates that the correlation between PGA, $S_a(T_1)$, $S_d(T_1)$, and $IM_{1E \& 2E}$ and $\theta_{\max}$ decrease gradually with an increase at the fundamental period, particularly for PGA. When the fundamental period approaches 10 s, the correlation coefficient of PGA is less than 0.4, which indicates that PGA has little correlation with $\theta_{\max}$ for the structures with super long periods. In addition, the correlation coefficient for $IM_{1E \& 2E}$ is approximately 0.7, which indicates that $IM_{1E \& 2E}$ has some correlation with $\theta_{\max}$; however, it does not accurately represent the characteristic of the seismic response of super high-rise buildings because only the first two modes are considered. The correlation between PGV and PGD and $\theta_{\max}$ increase gradually with increasing fundamental period, and after 6 s, the correlation between PGV and PGD and $\theta_{\max}$ decreases slightly. In addition, for a fundamental period of 9 s, the correlation coefficient ρ of PGD increases to 0.6, while the correlation coefficient ρ of PGV approaches 0.84. Therefore, PGV has the best correlation with $\theta_{\max}$, and PGA has the least correlation with $\theta_{\max}$ at a period of 10 s. Similarly, Figure 6 indicates that in the range of 10 s, the dispersion of PGV is the smallest, while the dispersion of PGA is the largest. These results indicate that for super high-rise buildings, PGV has the best correlation with $\theta_{\max}$ and the minimum dispersion. The existing literature also suggests that $\theta_{\max}$ has a significant correlation with structural damage (Bozorgnia and Bertero 2001; Miranda and Akkar 2006). This further confirmed that PGV can be suggested as an IM for structural seismic design and collapse analysis for super high-rise buildings.

Figure 7 indicates that $S_a(T_1)$, $S_d(T_1)$ and $IM_{1E \& 2E}$ have the greatest correlation with $d_{\max}$, and the correlation coefficients always remain at approximately 0.94, even when the fundamental period increases from

Figure 6. The variations of dispersion between $\theta_{\max}$ and IMs at different fundamental periods

Figure 7. The variations of correlation between $d_{\max}$ and IMs at different fundamental periods

1 to 10 s. The correlation between PGD and $d_{\max}$ increases gradually from less than 0.25 (at a period of 1 s) to 0.94 (at a period of 10 s). The correlation between PGV and $d_{\max}$ decreases gradually, and the correlation coefficient ρ is approximately 0.48 at a period of 10 s. Finally, the correlation between PGA and $d_{\max}$ decreases rapidly with increasing period and even becomes negative when the fundamental period is equal to 10 s. Figure 8 also indicates similar results, i.e., $S_a(T_1)$, $S_d(T_1)$ and $IM_{1E \& 2E}$ have the minimum dispersion among these IMs.

Figures 9 and 10 demonstrate that PGA has the greatest correlation with the structural maximum acceleration ($a_{\max}$) and the smallest dispersion β ; and the value of the correlation coefficient ρ remains at approximately 0.85 when the fundamental period increases from 1 to 9 s. PGV also shows some correlation with $a_{\max}$, and the correlation coefficient ρ remains near

Figure 8. The variations of dispersion between $d_{\max}$ and IMs at different fundamental periods

Figure 9. The variations of correlation between $a_{\max}$ and IMs at different fundamental periods

Figure 10. The variations of dispersion between $a_{\max}$ and IMs at different fundamental periods

0.6 and increases slightly with an increasing period. In contrast, $S_a(T_1)$, $S_d(T_1)$ and $IM_{1E\&2E}$ display poor correlations with $a_{\max}$ with large dispersions β . The

correlation coefficients decrease rapidly with an increase in the fundamental period. When the fundamental period becomes very long, the correlation coefficients become negative. This implies that the two variations have very little correlation, in other words, the more the IMs ($S_a(T_1)$, $S_d(T_1)$ and $IM_{1E\&2E}$) are, the smaller the $a_{\max}$ is. In addition, PGD also shows little or no correlation with $a_{\max}$ and the correlation coefficients is close to 0 in the period range concerned. In performance-based design, acceleration is usually associated with comfort and property loss; therefore, PGA can be selected as the IM for structural acceleration prediction.

4.3. Influence of α_0

To make the aforementioned results universal and representative, the influence of α_0 on the simplified model is discussed. Miranda and Taghavi (2005) reported that when $\alpha_0 = 0$, the simplified model degenerates into a pure bending model, whereas when $\alpha_0 = \infty$, this model can be approximately considered as a pure shear model. Although pure bending or pure shear behavior have little possibility of occurrence in super high-rise buildings with a vibration period as long as 9 s, for investigation purpose, a series of simplified models with $\alpha_0 = 0$ and 30 and $T_1 = 1, 3, 6, 9$, and 10 s (totally 10 simplified models) are created to examine the relationship between α_0 and the correlation coefficient between DMs and IMs. The variation in the correlation coefficients and the corresponding dispersion between $\theta_{\max}$ and each IM when $\alpha_0 = 0$ are illustrated in Figures 11 and 12, respectively. In addition, similar variations in the correlation and dispersion when $\alpha_0 = 30$ are illustrated in Figures 13 and 14, respectively. For different values of α_0 , Figures 15 and 16 demonstrate, respectively, the correlation coefficient ρ and the corresponding dispersion β between PGV and $\theta_{\max}$.

Figure 11. The variations of correlation between $\theta_{\max}$ and IMs at different fundamental periods when $\alpha_0 = 0$

Figure 12. The variations of dispersion between $\theta_{\max}$ and IMs at different fundamental periods when $\alpha_0 = 0$

Figure 13. The variations of correlation between $\theta_{\max}$ and IMs at different fundamental periods when $\alpha_0 = 30$

Figure 14. The variations of dispersion between $\theta_{\max}$ and IMs at different fundamental periods when $\alpha_0 = 30$

When $\alpha_0 = 0$, the simplified model represents a pure bending structure, whose higher-order period decreases rapidly. The second and third vibration periods (i.e., T_2

Figure 15. The variation of correlation coefficients between PGV and $\theta_{\max}$ with different value of α_0

Figure 16. The variation of dispersion between PGV and $\theta_{\max}$ with different value of α_0

and T_3) are approximately $1/6$ and $1/16$ of the fundamental period T_1 , respectively. For the flexural/shear structures with $\alpha_0 = 4$, on the other hand, T_2 and T_3 are approximately $1/3$ and $1/6$ of T_1 , respectively. It is evident that the influence of higher-order modes is smaller for pure bending structures. However, $IM_{1E \& 2E}$, which considers the first two vibration modes, has a significant correlation with $\theta_{\max}$. Figures 11 and 12 indicate that the correlation coefficients decrease slightly with increasing T_1 , and the values of the correlation coefficients remain above 0.9 for different fundamental periods. The dispersion of $IM_{1E \& 2E}$ is also the smallest. The correlation coefficients between PGV and $\theta_{\max}$ at different fundamental periods when $\alpha_0 = 0$ are always above 0.7. The correlation between $S_a(T_1)$ (or $S_d(T_1)$) and $\alpha_{\max}$ when $\alpha_0 = 0$ decreases gradually when T_1 increases. When T_1 is shorter than 6 s, the correlation between $S_a(T_1)$ (or $S_d(T_1)$) and $\theta_{\max}$ is slightly greater than that of PGV; however, when T_1 is longer than 6 s, the correlations are slightly worse than those of PGV. The correlation between PGD and $\theta_{\max}$ increases rapidly with increasing

T_1 , and when T_1 is longer than 6 s, the correlation decreases slightly, which is similar to the variation in PGV. PGA exhibits the least correlation with $\theta_{\max}$, and the correlation coefficients decrease rapidly with increasing T_1 . When T_1 approaches 10 s, the correlation coefficient increases slightly, but the value is still very low at approximately 0.27. When $\alpha_0 = 30$, the variations in the correlation coefficient ρ and the corresponding dispersion β between $\theta_{\max}$ and each IM at different fundamental periods approximate the values when $\alpha_0 = 4$. That is, by increasing the structural fundamental period, the correlations between the IM s (PGA, $S_a(T_1)$, $S_d(T_1)$, and $IM_{1E \& 2E}$) and $\theta_{\max}$ decrease gradually, while the correlations between PGV (or PGD) and $\theta_{\max}$ increase steadily. For a super long-period structure with a fundamental period longer than 9 s, PGV exhibits the best correlation with $\theta_{\max}$ and minimum dispersion. In contrast, PGA exhibits the least correlation with $\theta_{\max}$ and maximum dispersion.

Furthermore, Figures 15 and 16 demonstrate that for a pure bending structure ($\alpha_0 = 0$), the correlation coefficients between PGV and $\theta_{\max}$ at different fundamental periods are smaller than the values of a flexural-shear deformation combined structure ($\alpha_0 = 4$) or a shear deformation dominant structure ($\alpha_0 = 30$). For the bending-shear deformation combined structure ($\alpha_0 = 4$), the correlation coefficients are slightly smaller than the values for the shear deformation dominant structure ($\alpha_0 = 30$) if the fundamental period T_1 is shorter than 5 s. However, when T_1 is longer than 5 s, the correlation coefficients are slightly larger than the values for the shear deformation dominant structure ($\alpha_0 = 30$). Moreover, when T_1 approaches 10 s, these values are very similar. Overall, for different values of α_0 , PGV always shows a good correlation with $\theta_{\max}$. Considering that in practical application, most super high-rise buildings are flexural-shear deformation combined systems, it is still feasible to select PGV as the IM for the structural seismic design and collapse analysis of super high-rise buildings.

Similarly, for different values of α_0 , Figures 17 and 18 demonstrate, respectively, the variations of the correlation coefficient ρ and the corresponding dispersion β between $S_a(T_1)$ and $d_{\max}$; and such variations between PGA and $a_{\max}$ are presented in Figures 19 and 20, respectively. Figure 17 illustrates that α_0 has little influence on ρ between $S_a(T_1)$ and $d_{\max}$, where ρ is constantly above 0.95 for different values of α_0 in the period range concerned. The corresponding β remains below 0.2 (see Figure 18). Figure 19 illustrates that best correlations are achieved between PGA and $a_{\max}$ for shear deformation dominant structures ($\alpha_0 = 30$), and this is followed by flexural-shear deformation combined structures ($\alpha_0 = 4$). Whereas smallest correlation is found for pure bending structures ($\alpha_0 = 0$).

Figure 17. The variation of correlation coefficients between $S_a(T_1)$ and $d_{\max}$ with different value of α_0

Figure 18. The variation of dispersion between $S_a(T_1)$ and $d_{\max}$ with different value of α_0

Figure 19. The variation of correlation coefficients between PGA and $a_{\max}$ with different value of α_0

Such findings suggest that the larger the value of α_0 is, the better the correlation is between PGA and $a_{\max}$. It is necessary to note that the smallest correlation coefficient for pure bending structures ($\alpha_0 = 0$) is still larger than 0.78. This implies that PGA correlates very well with

Figure 20. The variation of dispersion between PGA and $a_{\max}$ with different value of α_0

$a_{\max}$ when α_0 varies between 0 and 30; and the influence of α_0 on the correlation coefficients is insignificant.

In general, α_0 has little influence on the correlation between IMs and DMs . Although the discussions given in Section 4.2 are based on the analysis of the Shanghai Tower, the findings remains universal and are applicable to other similar super high-rise buildings. In summary, for different values of α_0 , PGV, $S_d(T_1)$ and PGA always demonstrate good correlations with $\theta_{\max}$, $d_{\max}$ and $a_{\max}$, respectively.

5. VERIFICATION THROUGH COLLAPSE ANALYSIS OF THE SHANGHAI TOWER

It should be noted that the maximum story drift ratio $\theta_{\max}$ is always the controlling factor for seismic safety measure of building structures. Due to a very good correlation between PGV and $\theta_{\max}$, as indicated in the above analysis, PGV could be selected as an IM for seismic design and response prediction of super high-rise buildings. This conclusion is reached based on the simplified model illustrated in Figure 1 with elastic dynamic analysis; however, a super high-rise building may enter the nonlinear stage or even collapse under extreme earthquake conditions. Hence, the rationality of this conclusion must be further validated when the super high-rise building enters the nonlinear region. For this reason, the following section presents a verification of the above conclusion using an incremental dynamic collapse analysis of the Shanghai Tower presented in Figure 2.

Lu *et al.* (2011, 2013a, 2013b) proposed a nonlinear finite element model based on the general purpose finite element code of MSC.Marc to simulate the collapse of the high-rise and super high-rise buildings, including the Shanghai Tower. In the FE model of Shanghai Tower, the fiber model is adopted to simulate the columns and beams in the external frame and the components of the outriggers. The multi-layer shell model is used to simulate the shear walls, the coupling beams and the

mega columns; and elemental deactivation technology is adopted to simulate the failure of the components upon reaching their ultimate deformation stage. The complete FE model of the Shanghai Tower is illustrated in Figure 2. Further details of the structural dimensions, the material constitutive relationships and the collapse simulation procedures can be found in Lu *et al.* (2011).

Incremental dynamic analysis (IDA) is a powerful method in earthquake engineering for performing a comprehensive assessment of the structural behavior under seismic loads. During IDA, several ground motions are considered with increased levels of seismic intensity thereby making the structure undergo an entire range of behavior, from elastic to inelastic and finally to collapse. In this paper, the larger components of each of the 22 pairs of far-field ground motion records suggested by FEMA P695 (2009) are adopted as the ground motion set. In addition, data from the El-Centro EW 1940 ground motion record and the Shanghai artificial ground motion record are added into this ground motion set. This makes a total of 23 natural records and 1 artificial record. During the analysis, these ground motion records are input into the fine FE model along the X direction and their intensities are gradually increased up to collapse of the building. One of the typical potential collapse modes is shown in Figure 21. The values of different IMs (PGA, PGV, PGD, $S_d(T_1)$, $S_d(T_1)$, and $IM_{1E \& 2E}$) that cause the building to collapse are presented in Table 3. Since this building is very

Figure 21. The potential collapse mode of Shanghai Tower (The failed elements are removed from the model)

Table 3. The values of different IM s at the critical collapse status of the Shanghai Tower

	PGA (g)	PGV (cm/s)	PGD (cm)	$S_a(T_1)$ (g)	$S_d(T_1)$ (cm)	$IM_{1E\&2E}$ (g)
CAPEMEND_RIO270	2.2	249.92	122.34	0.0766	179.48	0.6024
CHICHI_CHY101-E	1.0	200.03	128.25	0.0830	193.56	1.3065
CHICHI_TCU045-E	2.8	216.48	298.92	0.2494	591.10	1.9549
DUZCE_BOL000	3.7	287.04	117.25	0.0988	226.69	0.9899
FRIULI_A-TMZ000	4.9	307.11	57.45	0.0327	66.57	0.4374
HECTOR_HEC000	2.2	236.55	186.39	0.1568	369.28	1.2333
IMPVALL_H-DLT262	1.9	207.61	95.63	0.1243	293.62	1.6218
IMPVALL_H-E11140	2.6	245.97	113.06	0.0762	179.00	1.2303
KOBE_NIS000	3.8	278.04	71.02	0.0473	104.36	0.5727
KOBE_SHI000	1.6	248.89	56.17	0.0315	67.93	0.6726
KOCAELI_ARC000	2.8	226.24	174.49	0.2093	495.39	1.6524
KOCAELI_DZC180	0.8	150.69	112.93	0.1070	251.70	0.8599
LANDERS_CLW-LN	2.4	217.43	116.41	0.1224	287.30	0.9601
LANDERS_YER270	0.7	146.99	125.11	0.1153	272.45	0.9064
LOMAP_CAP000	3.4	225.12	58.94	0.0362	70.57	0.4070
LOMAP_G03000	4.6	295.63	68.18	0.0518	110.49	0.6404
MANJIL_ABBAR—L	2.6	218.35	76.15	0.0563	123.70	0.5656
NORTHR_LOS000	2.0	209.65	56.91	0.0330	63.70	0.5564
NORTHR_MUL009	2.0	283.33	63.17	0.0323	65.47	0.5090
SFERN_PEL090	2.2	198.35	130.36	0.0808	187.77	1.1285
SUPERST_B-ICC000	2.0	258.85	98.17	0.1247	293.54	0.9823
SUPERST_B-POE270	2.8	224.43	55.26	0.0286	57.11	0.7038
El-Centro	1.8	201.98	48.07	0.0359	79.42	0.6422
Shanghai	1.4	182.75	92.94	0.0311	63.61	0.4805
COVs	0.443	0.182	0.530	0.690	0.729	0.468

important, its safety margin against earthquake-induced collapse is very large. The coefficient of variation (COV) of each IM is presented in Figure 22. It can be seen that when the building begins to collapse under the 24 ground motion records, the COVs of $S_d(T_1)$ and $S_a(T_1)$ are maximum, reaching nearly 0.729 and 0.690. This is followed by the PGA, PGD and $IM_{1E\&2E}$. In addition, PGV has the minimum COV, i.e. 0.182, which is approximately 2/5 of the COV of PGA and 1/4 of the COV of $S_a(T_1)$. Even when the super high-rise building

enters the nonlinear stage, PGV still shows a very good correlation with the structural response and collapse possibility. PGV can therefore be selected as the IM for seismic response prediction and collapse analysis.

6. CONCLUSIONS

This paper briefly reviewed some existing IM s and constructed a simplified flexural-shear beam-coupling model based on an actual super high-rise building, the Shanghai Tower. The correlations between the fundamental DM s and some existing IM s of this super high-rise building are analyzed, and the appropriate scopes for different IM s are also discussed. Consequently, the following preliminary conclusions are obtained:

- (1) With increasing structural fundamental period, the correlations between PGA, $S_a(T_1)$, $S_d(T_1)$ and the maximum story drift ratio (θ_{max}) decrease rapidly, and their corresponding dispersions increase gradually. Hence, PGA, $S_a(T_1)$ and $S_d(T_1)$ are not suitable IM s for super high-rise buildings for seismic design and time history analysis.
- (2) In the range of super long periods, the correlation coefficient between PGV and θ_{max} is more than 0.8 with a smallest dispersion. These results indicate that PGV has a significant

Figure 22. The distribution of COV for different IM s at the stage of collapse

correlation with $\theta_{\max}$. Therefore, PGV can be adopted as the *IM* for super high-rise buildings with super long periods.

- (3) Over the range of short to super long periods, $S_a(T_1)$ and $S_d(T_1)$ always exhibit significant correlations with the structural maximum top displacement ($d_{\max}$); therefore, $S_a(T_1)$ and $S_d(T_1)$ can be selected as the *IMs* for structural global displacement prediction.
- (4) PGA exhibits a significant correlation with the structural acceleration response; therefore, PGA can be selected as the *IM* for structural acceleration prediction and comfort design.
- (5) From the collapse analysis of the Shanghai Tower, it can be concluded that under strong or extreme earthquakes, even when the super high-rise building enters the nonlinear region, PGV can still be selected as the *IM* for seismic response prediction and collapse analysis.

It is worth mentioning that the above findings are reached based on the investigation of a single building - the Shanghai Tower. To ascertain these research findings to be used for seismic design guidelines of super high-rise buildings, more case studies on other tall buildings are desirable in further work.

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NOTATION

The following symbols are used in this paper:

IM ground motion intensity measure
DM seismic response demand measure
 PGA the peak ground motion acceleration
 PGV the peak ground motion velocity
 PGD the peak ground motion displacement
 $S_a(T_1)$ the spectral acceleration at fundamental period

$S_d(T_1)$ the spectral displacement at fundamental period
 $d_{\max}$ the maximum top displacement
 $a_{\max}$ the maximum acceleration
 $\theta_{\max}$ the maximum story drift ratio
 IDA incremental dynamic analyses
 THA time-history analysis
 α_0 the dimensionless parameter controls the proportions of the flexural and shears deformations
 ρ the correlation coefficient between *DM* and *IM*
 β the dispersion coefficient between *DM* and *IM*
 COV the coefficient of variation