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A Spectral Intensity Index of Ground Motions for Input Selection with Consideration of Higher Modes Effect for Super High-Rise Buildings

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Abstract

The selection of ground motion is a critical problem which can influence the structural response in seismic design and analysis. Nevertheless, the proposed intensity indexes are unable to consider the influence of higher modes (HM) on seismic response of super tall building structures reasonably. The effect of HM is measured by sufficient number of modes that modal participating mass ratio (MPMR) more than 90% generally. In this study, some spectral intensity indexes are summarized. The MPMR of the first three order vibration mode are counted. Meanwhile, the base shear error of 5 different super high-rise buildings are analyzed when structures with different MPMR. And then, a ground motion spectral intensity index（S_{90}）which MPMR more than 90% for considering the HM is put forward. The correlation between it and super high-rise structural seismic response is studied, and the incremental dynamic analysis (IDA) method is adopted to evaluate its discreteness. The results show that the statistics value of MPMR of the first three orders generally amounted to less than 80%. Only considering the first three order vibration modes of the ground motion intensity indexes to reflect the structural response features is insufficient. There are more than 5% error for the structural base shear due to keeping down vibration modes, and it cannot meet the requirement of the structural base shear errors control according to the MPMR requirements of 90%.The correlation between the response of super high-rise building and S_{90} is good and the discreteness is also in a reasonable range. It can therefore be taken as a significant reference index when selecting the ground motions input for the analysis of super high rise buildings.

Keywords: super high-rise building structures; HM; MPMR; spectral intensity index of ground motions; correlation

1. Introduction

The ground motion intensity index is the key linkage between the input features of ground motion and the structural seismic response. Since the start of using ground motion intensity indexes as one of the parameters in seismic design and analysis to now, the correlation between the ground motion intensity indexes and the structural response has been always attracting attention [1-3]. Liu Huisien[1] proposed two directions for choosing ground motion indexes, i.e. the pattern of ground motion (peak parameter) and the maximum structural response (various spectral curves), with more traditional consideration on peak parameters (e.g. Peak Ground Acceleration = PGA) in choosing the input of ground motion. However, Housner G W[4] and Fajar P[5] think the peak indexes are not good enough. Particularly, this single ground motion intensity indexes based solely on peak parameters is not satisfactory in reflecting the response and damage under earthquake interaction. To avoid the insufficiency of single parameter, Nau J M[6] analysed and concluded that the complex indexes are not necessarily better than the single indexes in terms of fully reflecting the structural damage. Therefore, it is proposed that applying spectral intensity indexes can be a better choice. Shome N[7] suggested to use the elastic response spectrum ($S_a(T_1)$), which corresponds to the natural vibration period of the structure, as the new ground motion intensity index. Vamvatsikos D[8] compared the effectiveness of PGA and $S_a(T_1)$ and pointed out that for conventional and low structures the spectral intensity indexes $S_a(T_1)$ show better correlation. Although the spectral intensity indexes with the consideration of the first order vibration mode is more advanced than the peak indexes, it soon hits the bottleneck owing to rapid development of super tall building structures. The seismic response of super tall structure is strongly affected by HM. Only using $S_a(T_1)$ or PGA to measure the response of ground motion seems to be insufficient. Lately, a new kind of intensity indexes considering first three or more order vibration modes had been proposed for super high-rise structures [9-14]. These intensity indexes largely improved the correlations. However, these indexes are partly aimed for conventional structures, others are incompatible in current seismic design because of the orders of vibration mode to be used. Thus, it cannot be applied for reasonably estimating the response of super tall buildings. With respect to the condition that the MPMR must be more than 90%, we studied the shear error in cases where the conditions were not met, and put forward a spectral intensity index (S_{90})for super tall building structures. The S_{90} takes the HM into account and meets the MRMR 90% requirement. We analysed its correlation to the response of structure and the discreteness. Its usefulness is verified.

2. Ground motion spectral intensity indexes

2.1 Existing spectral intensity indexes

Based on Liu Huisien's peak parameters and structural maximum response one can classify all kinds of intensity indexes easily. Among peak indexes, PGA, PGV etc are commonly adopted. Later on, a series of indexes derived from the peak indexes. Among them, the effective peak, PGV/PGA etc, are most influential. On the other hand, spectral intensity indexes demonstrate clear advantages in reflecting the structure amplitude, frequency and in considering the effect of HM for super tall structures. Shome N[7,15] et al put forward an easier and applicable index $S_a(T_1)$ which considers only the first order vibration mode and satisfies mid-and short period purpose with damping spectrum accelerations as normalized difference ground motion intensity indexes. Yet one finds out that this index doesn't work very well for super high-rise structure, long-period structures as well as near-source ground motion. Those are more subject to impact from HM. Von Thun J L[16] laid out an intensity index using the area within 0.1s~0.5s of S_a (ξ=5%, T), a 5% damping ratio of pseudo-acceleration spectra (see Eq.1)**.** Housner G W[17] discovered a correlation (see Eq.2) between pseudo-velocity spectra **(**S_v**)** and the maximal strain energy of elastic response ($E_{e,\max}$), which indicates that the structural velocity response spectrum as a parameter reflects structural damage during energy input, and serves a reasonable choice for ground motion intensity indexes. Housner further defined the seismic intensity as a function of damping ratio ξ as shown in Eq.3. Von Thun J L[16] had a similar equation to Eq.3 with a 5% damping ratio of pseudo-acceleration spectra.

$$ASI = \int_{0.1}^{0.5} S_a(\xi = 5\%, T)dT \quad (1)$$

$$E_{e,\max} = mS_v^2 / 2 \quad (2)$$

$$HI = S_I(\xi) = \int_{0.1}^{2.5} S_v(\xi, T)dT \quad (3)$$

Cordova P P[14], taking into consideration of the fundamental natural period of vibration of structure (T_1) and non-linear equivalent extended period (T_f), suggested a ground motion intensity index with two-parameter (S^*) which can evaluate the collapse performance of frame structure (see Eq.4). Parameter μ is to be determined, it could be 0.5, and T_f could equal to $2T_1$. Vamvatsikos D and Cornell C A [18] had similar indexes as in Eq.5, in which T_a and T_b are any period available (generally, T_a is T_1 and T_b is T_f). ν is a parameter to be determined with a value less than 1. An obvious improvement of this intensity index is, due to that fact that T_a and T_b are any period available, it can be concluded preliminarily that the impact of HM has been considered, though in fact it still only refers to the impact of the first two order vibration modes. Intensity indexes S_{N1} and S_{N2} from Lin L and Naumoski N [12] achieve the same goal as S^* and S_{VC}, see Eqs.6 and 7, in which χ and ω are parameters to be determined. χ is 0.5, C is 1.5, and ω is 0.75. These two indexes introduce the elastic response spectrum corresponding to CT_1 and T_2, which is a similar concept of T_f in S^*. S_{N1} is mainly used in structures of shorter natural vibration period and the first order vibration mode, while S_{N2} is used in structures of longer period.

$$S^* = \left[S_a(T_1)\right]^{1-\mu} \cdot \left[S_a(T_f)\right]^{\mu} \quad (4)$$

$$S_{VC} = \left[S_a(T_a, 5\%)\right]^{1-\nu} \cdot \left[S_a(T_b, 5\%)\right]^{\nu} \quad (5)$$

$$S_{N1} = \left[S_a(T_1)\right]^{\chi} \times \left[S_a(CT_1)\right]^{1-\chi} \quad (6)$$

$$S_{N2} = \left[S_a(T_1)\right]^{\omega} \times \left[S_a(T_2)\right]^{1-\omega} \quad (7)$$

Bojórquez E and Iervolino I[13] suggested a more effective index ($S_{a,\mathrm{avg}}$)as shown in Eq.8, considering the number of vibration modes n=10. This reflects to a certain extent the impact of HM with preferable effectiveness, but lacks theoretical backing for choosing the simple value of n as 10. Otherwise, intensity indexes $\overline{S_a}$ and $S_{a,\mathrm{avg}}$ have a similar equation, also considering HM impact, and have made improvement on the selection of the number of vibration modes, see Eq.9, but the set period of 10s of the value of the number of vibration modes bears certain limitation.

$$S_{a,\mathrm{avg}} = \left[\prod_{i=1}^{n} S_a(T_i)\right]^{1/n} \quad (8)$$

$$n = \begin{cases} 1 & (T_1 \le 1s) \\ 0.39T_1 + 1.15 & (1s < T_1 \le 10s) \end{cases} \quad (9)$$

Luco N & Cornell C A[20] et.al put forward the ground motion intensity indexes $IM_{1E\&2E}$, see Eq.10, based on the first two order modals to detect the pulse effect of near-fault ground motions and the HM impact. These indexes take into account of the impact of story drift angle, with later improvement in including the impact of HM, using displacement spectrum as the parameter, which, nevertheless, is not only complicated, but also shows certain limitation because it works only for the pulse of near-fault ground motions.

$$IM_{1E\&2E} = \sqrt{[PF_1^{[2]}S_d(T_1,\xi_1)]^2 + [PF_2^{[2]}S_d(T_2,\xi_2)]^2} = \sqrt{1+R_{2E/1E}^2}\left|\frac{PF_1^{[2]}}{PF_1^{[1]}}\right|\left|PF_1^{[1]}\right|S_d(T_1,\xi_1) \tag{10}$$

where $R_{2E/1E} = \dfrac{PF_2^{[2]}S_d(T_2,\xi_2)}{PF_1^{[2]}S_d(T_1,\xi_1)}$; $S_d(T_1,\xi_1)$ and $S_d(T_2,\xi_2)$ are displacement spectrum values with damping ratio of ξ_1 and ξ_2 when the first two order modals are T_1 and T_2 respectively. $PF_1^{[2]}$ is the first order modal participating ratio corresponding to the maximum story drift angle which combining with square root of the sum of the squares (SRSS) method and considering the first two order modals of the structure; while $PF_1^{[1]}$ only considering the first order modal of the structure. According to the first order vibration modal period T_1, and T_m, which is MPMR over 95%, Adeli M M [21] suggested to use A_0, a new ground motion intensity index based on the area among 1.2 T_m to 1.5 T_1 period and elastic response spectrum.

2.2 Improved spectral intensity index

As we know, the influence of HM is significant for the super high-rise structures. However, the ground motion intensity indexes for such structures are not many. As remedial measures, other ground motion intensity indexes have been proposed, but opinions differ on the number of vibration modes and truncation methods during structural design and analysis. Some referred to MPMR reaching 80% during the the first three order vibration modes, which satisfies the HM requirement, that is to control the number of vibration modes via the minimum standard deviation of natural logarithm of maximum story drift angle. As to current mainstream codes and analytical methods, a 90%, not 80% of MPMR is needed considering the HM impact. In the meantime, MPMR always failed to reach 80% of the first three or more order vibration modes in super high-rise structures, see Table 1. Currently, structural base shear is an important controlling parameter in both the equivalent base shear method and the mode-superposition response spectrum method , and calculation has shown that when MPMR is 80%, the elastic base shear failed to meet the 5% control error requirement in five structures cases presented in this paper, while when MPMR reaches 90% and over, it serves 95% confidence level, the number of vibration modes chosen can cover most of the earthquake action of the structures, see Fig. 1. That means even 80% of MPMR cannot well reflect structural seismic response. And the period of proposed vibration mode numbers control method is within 10s, but current range in some super high-rise structures has exceeded this limit. With view to the base share satisfying 95% of confidence level, 90% MPMR can do a good job in controlling the HM impact. Based on this assumption, the number of vibration modes considering 90% MPMR and elastic base shear of 95% of confidence level contains more vibration mode information, thus improves the effectiveness of ground motion intensity index S_{90}, a ground motion intensity index by power function product form, is proposed in Eq.11. And because the conditional probabilities of ground motion intensity indexes and damage indexes meet the need of logarithmic normal distribution[23, 24], from Eqs.12, 13, an advantage in physics is discovered during the natural logarithsm, represented by logarithmic power function product form, of intensity indexes in mode summation.

$$S_{90} = \left[S_a\left(T_1,\xi\right)\right]^{\alpha} \cdot \left[S_a\left(T_2,\xi\right)\right]^{\beta} \cdots\cdots \left[S_a\left(T_n,\xi\right)\right]^{\theta} \tag{11}$$

$$\ln S_{90} = \ln\left\{\left[S_a\left(T_1,\xi\right)\right]^{\alpha} \cdot \left[S_a\left(T_2,\xi\right)\right]^{\beta} \cdots\cdots \left[S_a\left(T_n,\xi\right)\right]^{\theta}\right\} \tag{12}$$

$$\ln S_{90} = \alpha \cdot \ln S_a\left(T_1,\xi\right) + \beta \cdot \ln S_a\left(T_2,\xi\right) + \cdots + \theta \cdot \ln S_a\left(T_n,\xi\right) \tag{13}$$

Where $\alpha = \dfrac{m_1}{m_1+m_2+\cdots+m_n}$, $\beta = \dfrac{m_2}{m_1+m_2+\cdots+m_n}$, ... $\theta = \dfrac{m_n}{m_1+m_2+\cdots+m_n}$, m_1、$m_2 \ldots m_n$ are the modal participation ratio of the first n-order vibration modes with a period range of T1-Tn. The dominance principle is $\sum_{i=1}^{n} m_i \geq 90\%$, and the number of vibration modes with 95% confidence level of the elastic base shear.

Table 1-The MPMR sum of first three order vibration modes in some high-rise buildings

building name	height(m)	period T_1(s)	1st order PMPR(%)		2nd order PMPR(%)		3rd order PMPR(%)		sum of PMPR (%)
			X	Y	X	Y	X	Y	X/Y
Shenzhen pingan financial center	660	8.67	18.11	35.69	35.66	18．18	0.08	0.01	53.85/53.88
Guangzhou west tower	405	7.43	0.00	53.40	53.40	0.00	0.00	21.44	53.40/74.84
a residential	93	—	0.00	67.58	71.36	0.00	2.94	0.00	74.30/67.58
Tianjin silver 117 tower	597	9.07	50.06	0.00	0.00	49.49	0.00	0.00	50.06/49.49
wuhan international securities building	281	—	78.17	0.03	78.20	81.79	78.20	81.81	78.20/81.81
Z15 Tower	528	7.59	46.27	0.12	0.12	46.39	0.00	0.00	46.39/46.51
An irregular building	95	2.10	0.24	56.06	22.96	3.21	42.23	0.67	65.43/59.94

3. Analysis input of model

3.1 Analysis of model information

In control of the accuracy of structural seismic response, the key elements are structural and material models. In this study, 5 models are laid out for the super tall building RC frame-core wall structures in zones with seismic intensityⅦ. In PERFORM-3D analysis, beams and columns use the fiber element model, and the wall element uses shear wall element. Constitutive model of confined concrete uses the Scott-Kent-Park model, and that of the unconfined concrete refers to the "Code for Design of Concrete Structures", with the steel chosen under the ideal elastoplastic model. Parameters for seismic analysis refer to the 'Code for Seismic Design of Buildings' of the PR China, the designed damping ratio is 5%. The standard of load in the models is 5kN/m^2 of the roof dead load, 2.5kN/m^2 of the roof live load. Gravity representative value is a combination of 100% constant load, and 50% live load. Heights of the 5 models are 168.0, 247.5, 283.5, 315.0, and 340.0 meters respectively, and the corresponding structural periods are 4.601s, 5.447s, 6.617s, 7.735s and 8.876s. Table 2 contains basic information of the models, and Fig. 2 is the layout of structural plan.

Table 2-The information of models

model	stories	wall thickness (mm)	square pillars size(mm)	concrete grade		model	stories	wall thickness (mm)	square pillars size(mm)	concrete grade	
				wall/ column	beam/ plate					wall/ column	beam/ plate
1	1-15	500	1100	C60	C40	4	1-5	800	2100	C60	C40
	16-32	400	900	C60	C40		6-15	800	2000	C60	C40
	33-40	300	800	C50	C40		16-30	750	1800	C50	C40
2	1-15	800	1650	C60	C40		31-40	700	1600	C50	C40
	16-25	750	1400	C60	C40		41-50	500	1400	C50	C40
	26-35	700	1300	C60	C40		51-60	500	1200	C50	C40
	36-45	500	1200	C50	C40		61-70	500	1000	C50	C40

	46-55	500	1000	C50	C40	5	1-8	900,700	2200	C60	C40
3	1-15	800	1850	C60	C40		9-18	900,700	2100	C60	C40
	16-25	750	1600	C60	C40		19-33	800,600	1900	C60	C40
	26-35	700	1400	C60	C40		34-43	600,400	1800	C60	C40
	36-45	500	1200	C50	C40		44-57	600,400	1600	C50	C40
	46-56	500	1000	C50	C40		58-68	500,400	1400	C50	C40
	57-63	500	800	C50	C40		1-8	900,700	2200	C60	C40

3.2 Seismic wave input

The impacts on the response of structures of seismic wave input and structural model are of the same importance and remain the two basic elements in controlling structural response. This study has used forty seismic records, with magnitude greater than magnitude 6, PGA range from 80.53cm/s ²to 314.30cm/s ², PGV range from 0.867cm to 64.610cm, duration range from 0.72s to 104.82s. These records have covered all types of earthquakes, therefore they have a certain universality and representation. Using MATLAB software, ground motion parameters of these forty earthquake records have been retrieved for correlation analysis. The other thirteen records are selected by controlling two frequency stages which agree well with response spectrum**,** and discreteness analysis has been carried out based on IDA method.

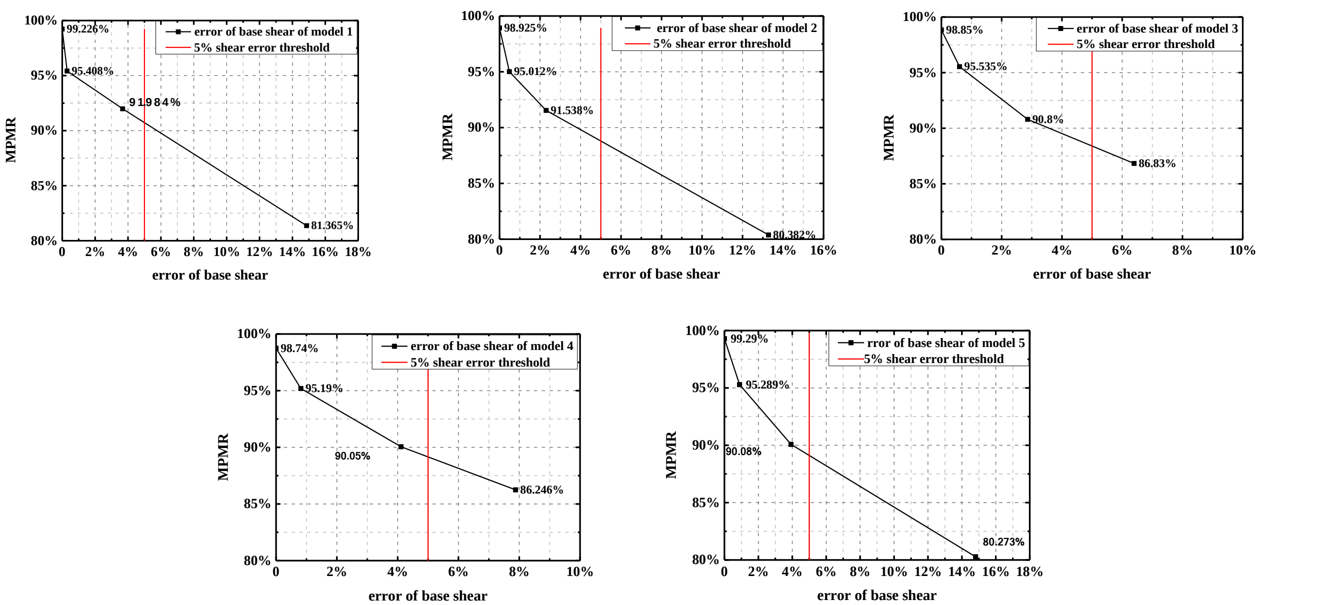
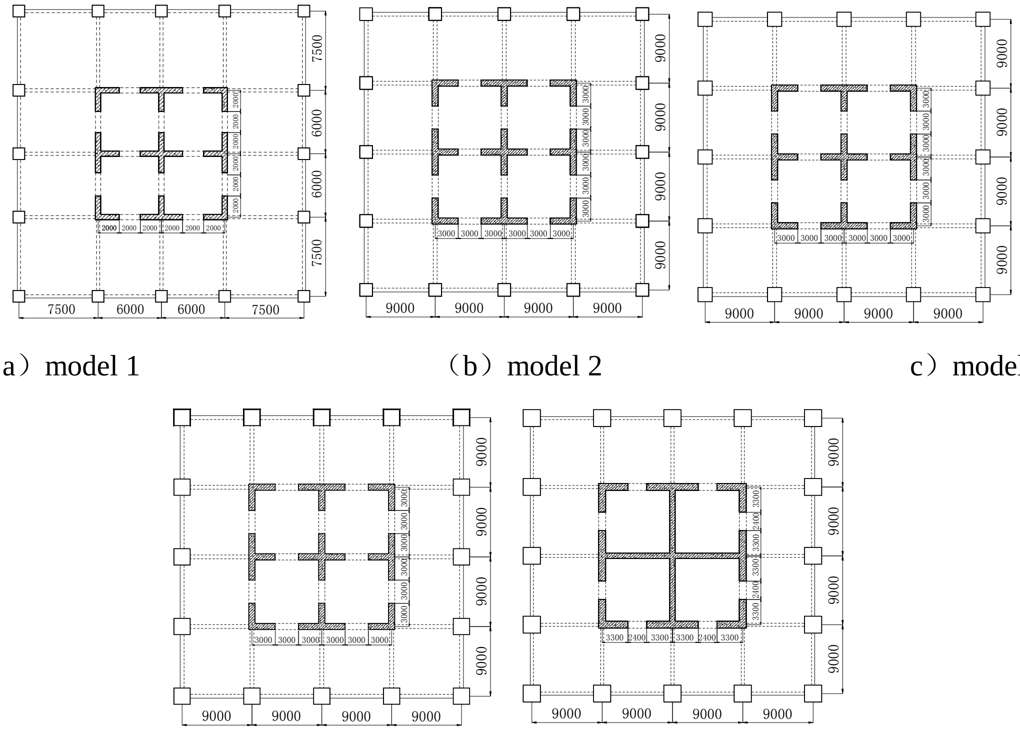


Fig.1 The base shear comparison among different MPMR



（a）model 1　　（b）model 2　　c）model 3

（d）model 4　　（e）model 5

Fig.2 The layout of structural plan

4. Evaluation of ground motion intensity indexes

5.1 Evaluation parameters

In previous researches, SDOF, MODF or simplified model were often used to analyze the effectiveness of intensity indexes, and those methods had certain limitation. With the more frequent use of 3-D analysis, correlation evaluation between structural nonlinear responses and ground motion parameters, using 3-D solid model, becomes a better solution to the effectiveness and applicability of intensity indexes, and more practical in predicting structural seismic response and damage. Correlation and discreteness analysis is the basic principle to evaluate effectiveness. Correlation coefficient is R, discrete coefficient is η. R is an effective parameter to evaluate the correlation between ground motion intensity indexes and structural response, see Eq.14, in which the value of R is between [-1, 1]. Normally, when $|R| \geq 0.8$, there is a satisfying correlation between the response indexes and intensity indexes. Discrete coefficient η provides solid evidence to the intensity indexes, as is shown in Eq.15, whose value reflects the degree of discreteness of structural damage measure (DM) under given intensity measure (IM). The smaller the value of η, the better the ground motion intensity indexes. According to the features of these parameters, analytical procedures in the reference[9] can be applied.

$$R = \frac{\mathrm{cov}(IM, DM)}{\sqrt{D(IM)} \cdot \sqrt{D(DM)}} = \frac{\sum_{i=1}^{n} \left(IM_i - \overline{IM}\right) \cdot \left(DM_i - \overline{DM}\right)}{\sqrt{\sum_{i=1}^{n} \left(IM_i - \overline{IM}\right)^2} \cdot \sqrt{\sum_{i=1}^{n} \left(DM_i - \overline{DM}\right)^2}} \tag{14}$$

$$\eta = \sqrt{\frac{\sum_{i=1}^{n} \left[\ln\left(DM_i\right) - \left(a + b \cdot \ln\left(IM_i\right)\right)\right]^2}{n-2}} \tag{15}$$

Where, $\mathrm{cov}(IM,DM)$ is covariance of sum of random variables. $D(IM)$, $D(DM)$ are mean square error, $\overline{IM}$ and $\overline{DM}$ are the average of variates, a and b are fitting parameters.

5.2 Correlation evaluation of ground motion intensity indexes

This study focuses on the correlation between the maximum seismic response indexes ($\theta_{\max}$ as story drift angle, $V_{\max}$ maximum base shear, $d_{\max}$ maximum top displacement, $v_{\max}$ maximum top velocity, $\alpha_{\max}$ maximum top acceleration, E_{input} structural input energy) and intensity indexes (PGA, HI, $S_a(T_1)$, $\bar{S}_a$, S_{N2}, A_0, S_{90}) of the structural models when input 40 records, using FEM, and the value of R as well as the correlation diagram between R and structural period T are used to evaluate the correlation between various intensity indexes and structural response. In Fig. 3, features of the correlation between intensity indexes and response indexes are discovered that, in long-period super high-rise structures, correlations between $\theta_{\max}$, $d_{\max}$, $v_{\max}$, E_{input} and ground motion indexes PGA are very low, in which the correlation between PGA and $\theta_{\max}$ is less than 0.40, with a minimum of only 0.271; the correlation with $d_{\max}$ fluctuates around 0.3, and the two correlations tend to decrease alongside the increase of structure period. The correlation between PGA and $v_{\max}$ increases alongside the structure period. It increases from 0.333 to 0.568, which indicates a growing correlation between PGA and the maximum velocity of structural top. The correlation with E_{input} also increases from 0.076 to 0.112, a similar tendency with $v_{\max}$, but varies significantly in terms of correlation. The correlations between PGA, $V_{\max}$ and $\alpha_{\max}$ remain relatively high, 0.654 and 0.698 respectively, but still fails to meet the satisfying threshold of | R | ≥0.8. The correlation between HI (velocity response spectrum based intensity indexes) and $\theta_{\max}$ is between 0.619 to 0.74, which is a smooth change and a good indicator of correlation and maximum structure story drift angle response to a certain extent. The correlation with E_{input} various from 0.727 to 0.830, a stable change, with a maximum greater than 0.8, and indicates well the damage index during structure energy input. But the correlations with $V_{\max}$, $d_{\max}$, $\alpha_{\max}$ are relatively poor, with a minimum of 0.064 and a maximum less than 0.553, and fail to represent the shear force, acceleration and displacement of the structure. The correlation interval with $v_{\max}$ is from 0.453 to 0.679, which changes significantly with an unstable correlation, and also reflects a tendency of decreasing correlation coefficient alongside increasing period. The existing spectra intensity indexes ($S_a(T_1)$, $\bar{S}_a$, S_{N2}, A_0) and S_{90}, and $\theta_{\max}$ from this study all show good correlation, with the coefficient ranging insignificantly from 0.828 to 0.947, which indicates the stability of response of spectra intensity indexes on maximum story drift angle, and the features of structural response is reflected. From Fig. 3(a), the maximum correlation coefficient average is S_{90}. A_0 shows higher correlation of the maximum structural base shear, with the maximum value of 0.861, while other parameters show less than 0.8, ranging from 0.548 to 0.748, with the maximum of 0.748 which corresponds to S_{90}, but the correlation coefficient has less fluctuation. Fig. 3(c) shows their correlation with $d_{\max}$: $S_a(T_1)$ and S_{N2} has better correlation with $d_{\max}$, with a same correlation coefficient, and maximum value of 0.983, which indicates a good correlation and consistence; S_{90} shows the second best performance, with the maximum correlation coefficient of 0.975, which also reflects well the maximum displacement response. $\bar{S}_a$ and $d_{\max}$ has a correlation coefficient over 0.852, which meets the requirement of the over 0.8 criterion, with only the coefficient of A_0 below 0.8, with a minimum value of 0.693. In Fig. 3(d), S_{90} has the best correlation coefficient, a maximum of 0.934, which reflects well the structural velocity response. Other intensity indexes also show good stability when periods change, with the correlation coefficient ranging from 0.70 to 0.92. In Fig. 3(e), spectra intensity indexes and S_{90} show poor correlation, with the value of the coefficient at only 0.2, while the correlation with PGA being high-lightened. The correlation coefficient between spectra intensity indexes and energy input is from 0.415 to 0.861, which changes significantly, and tends to increase alongside that of the period, but not significant. Only correlation coefficient of A_0 is over 0.8, which shows that among all indexes, only A_0 reflects well the impact of energy input, and relates to the physical property of the indexes. In Fig. 3(a-b), the R-T curves of $S_a(T_1)$ and S_{N2} are overlapped, and Fig 3 (e) and (f) also differ insignificantly, which indicates that these two intensity indexes have a consistent correlation with seismic response, and can reflect the same structural response. The correlation analysis of PGA and response indexes proved Housner's theory of "single peak intensity indexes being not able to reflect well the structural damage", and it also means that in super high-rise building structures, the dominant element is not the acceleration, but velocity and displacement. The higher correlation of spectra intensity indexes and super high-rise structure over PGA is strong evidence of the applicability of spectra intensity indexes. The correlation coefficient between HI and E_{input} is relatively high, with a maximum of 0.83, which falls into the | R | ≥0.8 criterion. This coincides with Housner's research that structural velocity response spectrum value is a parameter that reflects the structural damage during energy input. The poor correlation between HI and other response indexes also shows the limitation of Hi applied in super high-rise building structures. From the previous study, S_{90} has good correlation with various response indexes, meets the | R | ≥0.8 criterion, and reflects well structural seismic response, and has been proved a qualified intensity index for engineering practice.

5.3 Discreteness evaluation based on IDA

Discrete coefficient η is used to evaluate the effectiveness of intensity indexes, and can be achieved from analysis based on IDA. This study uses model 4 for the analysis. First of all, the IM and the corresponding structure DM relation IDA curve is achieved through nonlinear time-history analysis. Because story drift angle is the most common index to evaluate structural damage and collapse, DM analysis with $\theta_{\max}$ as IDA is a reasonable choice, and seismic IM chooses PGA, $S_a(T_1)$, $\bar{S}_a$, S_{N2}, A_0, S_{90}. Non-linear dynamic structural analysis is carried out for 13 records, and a series of data points are recovered, such as (DM, IM). These discrete points are then drawn to a 2-D coordinate with IM as the ordinate, and DM as the abscissa axis. Corresponding data points of every earthquake record are then connected to get 13 IDA curves for each index. On the one hand, PGA is used as the amplitude modulation of input record of IM time history analysis, so when other intensity indexes are used to draw the IDA , massive calculation is no longer necessary, and only the value of PGA intensity indexes with regard to each seismic wave. Fig. 4 shows the IDA curves of the six intensity indexes.

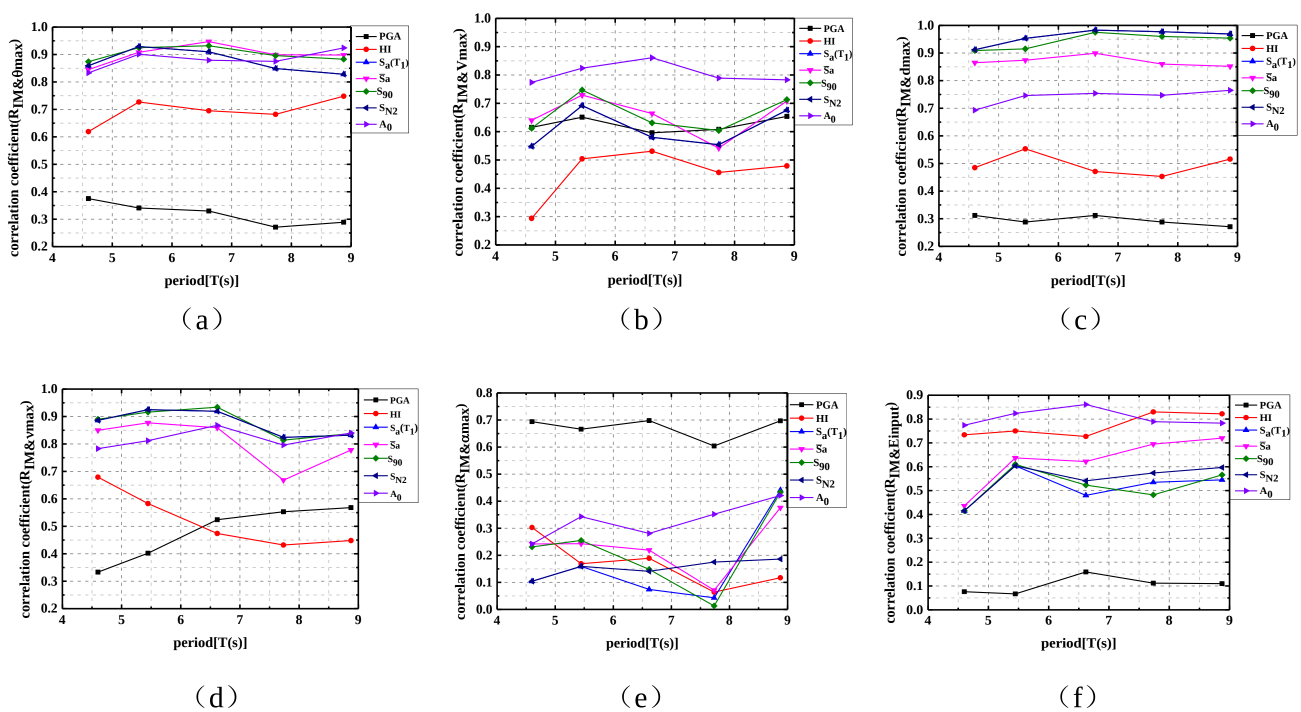


Fig.3 R-T relation curves of different structures with different parameters

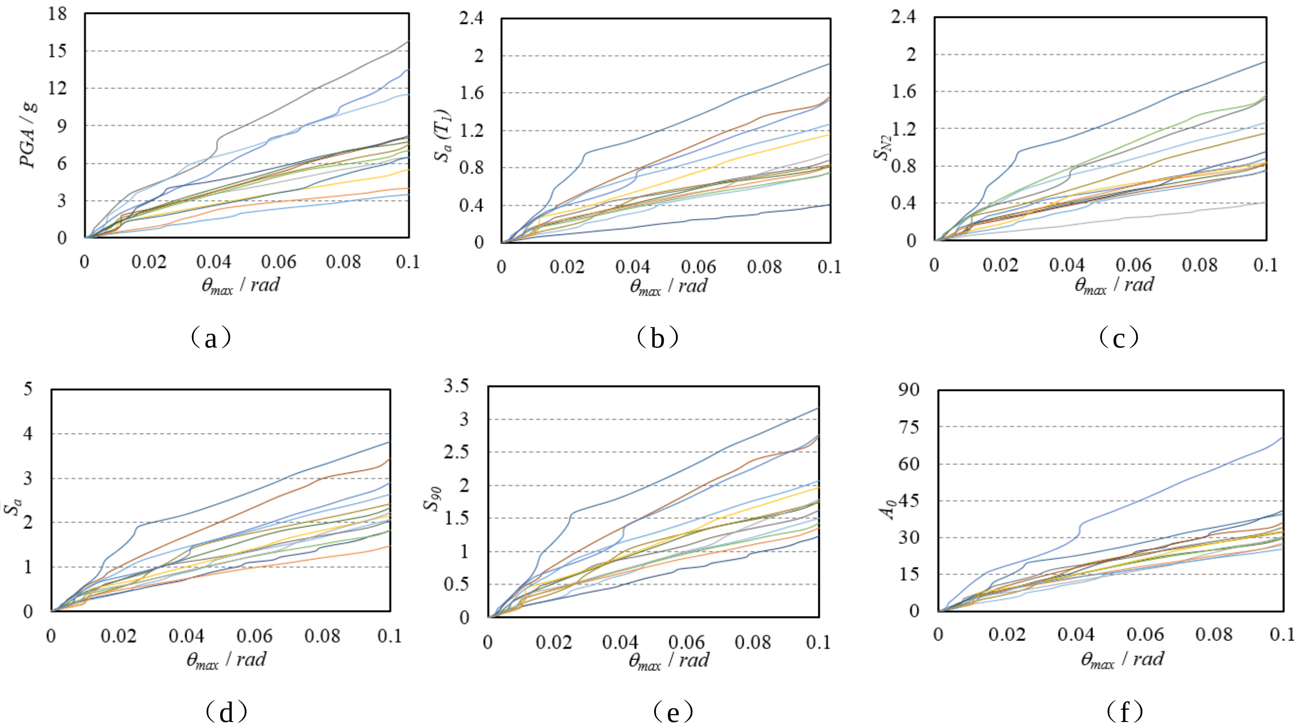
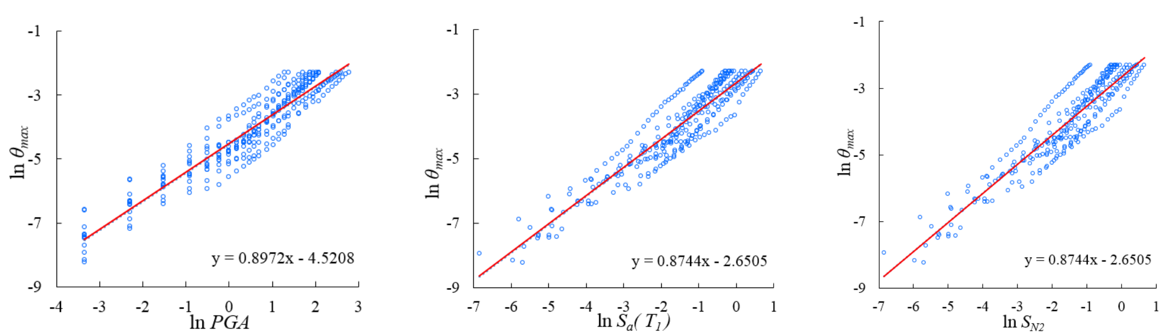


Fig.4 IDA curves of different intensity measures

The discreteness of IDA curve is an important parameter to evaluate intensity indexes, and can reflect confidence level of the statistics, and monitor the structural collapse process. In Fig. 4, when IDA is carried out for the same computational model, degrees of discreteness of IDA curves obtained from different ground motion intensity indexes vary significantly. The same case is found in IDA curves of the same intensity indexes based on different seismic records input. IDA curves using A_0 , $\bar{S}_a$, S_{90} as intensity indexes display a relatively concentrated distribution, with a lower discreteness; IDA curves using PGA, S_{N2} , $S_a(T_1)$ as intensity indexes display a more diffused distribution, with a higher discreteness, which is found in line with the results of the discrete coefficient. On the other hand, engineering parameters nearly fit normal distribution, natural logarithm of the corresponding discrete points (IM_i , DM_i) from dynamic elastic-plastic analysis generates a series of data points ($\ln(IM_i)$, $\ln(DM_i)$), and all data points collected are drawn on the 2-D coordinate with $\ln(IM)$ and $\ln(DM)$ as axes. According to $\ln DM = a + b \ln IM$, least square method is used for statistical regression of these discrete points to get parameters and of different intensity indexes. Regression calculation is shown in Fig.5.



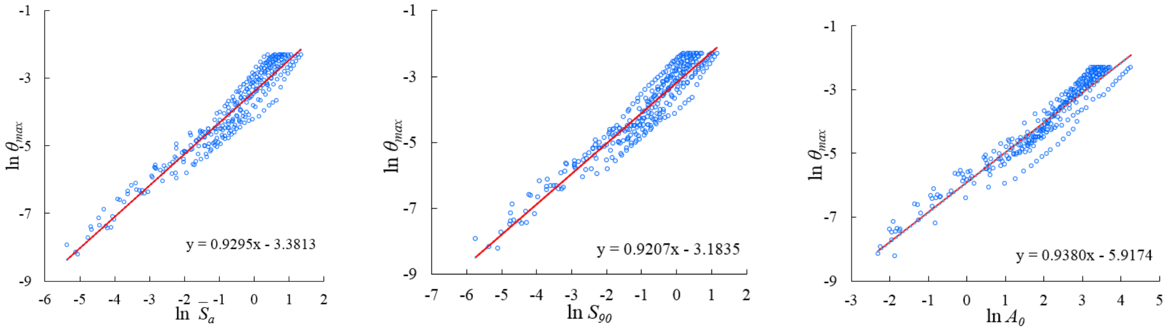


Fig.5 results of regression coefficients on logarithmic coordinates

And then, substitute the fitting parameters a, b into Eq.15 to get the discrete coefficient, and the value of which is used to determine the applicability of the intensity indexes, see Fig.6. Intensity indexes such as PGA, $S_a(T_1)$, S_{N2} which do not consider HM has a much higher discrete coefficient than indexes A_0, $\bar{S}_a$, S_{90} which take HM into consideration. This indicates that in super high-rise structures, the more vibration mode information included in ground motion indexes, the lower the discreteness found in nonlinear behavior of structural seismic action, and the better prediction of response as well as evaluation of structural performance on a more accurate level. Comparing discrete coefficients of $S_a(T_1)$ and PGA, effectiveness of analysis using $S_a(T_1)$ index is lower than that of PGA, which indicates, different from lower and short period, super high-rise building structures significantly influenced by HM adopt only the ground motion intensity indexes containing the first order vibration mode information, and cannot effectively reduce discreteness, that reaches similar conclusion with M. Mahdavi Adeli et al [11]. Compared with ground motion indexes used in super high-rise building structures, discrete coefficient of A_0 is small at only 0.375, mainly because it considers MPMR of 95%. While the corresponding logarithmic standard deviation of $S_a(T_1)$ and S_{N2} is highest at 0.509, which also indicates a reasonable choice of S_{90} as an intensity index from this study.

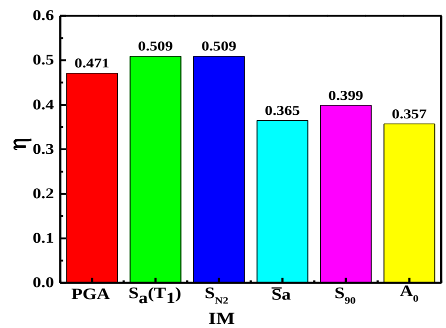


Fig.6 the standard deviation of different IM

5. Conclusions

HM has a significant impact on super high-rise structures. Therefore, this study gathers MPMR data of the first three vibration modes, and considers the control method of the number of vibration modes. Using the mode-superposition response spectrum method to analyze shear force response error of different MPMR, a power function product form, S_{90}, has been proposed to indicate spectral intensity indexes. The analysis of elastic-plastic time-history of 3-D structural model, by using ground motion input, explores the correlation between ground motion spectral intensity indexes and structural response. And based on IDA the discreteness of structural spectral intensity indexes has been discussed. The following conclusions can be drawn:

(1) HM impact should not be overlooked because of the complexity of height and structure of super high-rise buildings. The MPMR of the first three vibration modes normally fails to reach 80%, which is inconsistent with the requirement of 90% in structural design. 95% confidence level of structural base shear error when MPMR reaches 90%, and when MPMR is 80%, the structural base shear error would exceed the 5% threshold.

(2) In super high-rise building structures, correlation between PGA and other structural response indexes is barely satisfactory. When | R | <0.8, acceleration has a relatively less impact on super high-rise building structures response, and velocity and displacement play a more critical role. The correlation between HI and other structural response indexes is also rather low, showing the limitation in the application in super high-rise building structures. However, correlation coefficient between HI and E_{input} is over 0.8, which is a good indicator of response energy input. Indexes such as $S_a(T_1)$, $\bar{S}_a$, S_{N2}, A_0, and S_{90}, proposed in this study, have a fairly preferable general correlation, other than a poor one with maximum acceleration response. S_{90} maintains good correlation with all intensity indexes, and can reflect proper ground motion intensity indexes of seismic response in super high-rise building structures.

(3) IDA is a good indicator to analyze discreteness of ground motion intensity indexes. The more vibration modes information the indexes contain, the less discreteness ground motion intensity indexes have, with better application. has an obvious advantage in the discreteness of ground motion intensity indexes with HM impact, and S_{90} is a reasonable intensity index to consider HM impact in super high-rise building structures.

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