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Dynamic Analyses on the Floor Acceleration Response Spectra of Steel Building Structures with Different Heights

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Abstract

The percentage of non-structural systems in the total cost of the building is continuously increasing in the last decades. The seismic capability of non-structural components is the key to the building function. Finding suitable floor acceleration response spectra in order to realize feasible design of non-structural members under earthquake is very important. Nowadays steel structure is widely used in the construction industry. This paper aims to study the floor acceleration response spectra of steel buildings designed in compliance with Chinese building code. Chinese version architectural design software PKPM is used in this study. The ground motions are selected from Pacific Earthquake Engineering Research Center(PEER) and PKPM ground motion database. Based on the required characteristic period, the appropriate ground motions are selected in group. The objects of this study are 23 steel structures with different basic period and heights, including steel frame structure, concentrically braced frames structures and eccentrically braced frames structures. Site category and seismic grouping are taken into account when the building models are selected. In the elastic time history analysis, the characteristic period corresponding to each structure is determined in compatible with the site category of each structure and the designed seismic grouping. Also in compatible with the first three modal periods of each structure model, 100 ground motions are selected as the acceleration input for the elastic time history analysis. All elastic time history analyses are carried out with the mode superposition method. The bi-directional horizontal time history analyses are carried out in PKPM for model structures with seismic precautionary intensity of 6, 7 and 8. While 3-dimentional including the vertical direction time history analyses are performed for the model structures with seismic precautionary intensity of 9. With the floor response time-history results obtained from the elastic analyses, the acceleration response spectra in principal direction, secondary direction and vertical direction of each floor at different heights are calculated, and the upper and lower envelope spectra, average spectra, and the spectra corresponding to ±20% standard deviation of these acceleration response spectra are obtained. Amplification factors of the peak floor acceleration response spectral with the height are analyzed. The results could provide useful reference for the seismic design of non-structural components in steel structural buildings.

Key words: floor acceleration response spectra; elastic time history analysis; steel structures

1. Introduction

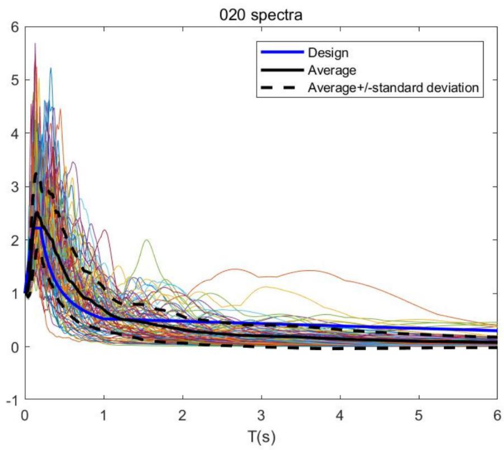
Non-structural components is an important part of the buildings. The loss of non-structural components caused by earthquakes is huge. The study on capacity of non-structural system is necessary. The study on floor acceleration response spectra is the base of the study on the seismic capability of non-structural system. This study is aimed at the floor acceleration response spectra of steel structures.

This paper contains 3 parts, according to the research process. The first part is the selection of ground motions. The second part is the selection of models. The third part is elastic time history analysis.

2. Study on floor acceleration response spectra of steel structure buildings

2.1 Selection of ground motions

The ground motions used in this study are selected from Pacific Earthquake Engineering Research Center (PEER) and PKPM ground motion database. Corresponding to the site characteristic period in seismic grouping of GB 50011, these selected ground motions are divided into 10 groups. The characteristic period of each group is 0.2 second, 0.25 second, 0.3 second, 0.35 second, 0.4 second, 0.45 second, 0.55 second, 0.65 second, 0.75 second, and 0.90 second respectively. There are 100 sets of ground motions selected for each group, including 96 sets of three-dimensional natural ground motions and 4 sets of three-dimensional artificial ground motions. The artificial ground motions are selected from PKPM ground motion database. The normalized spectra of selected ground motions in each group are shown in Fig-1.



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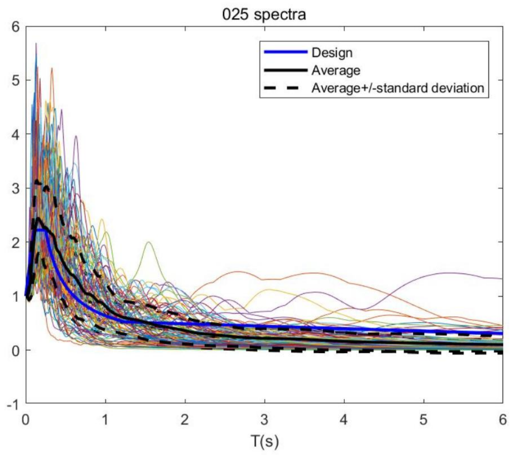

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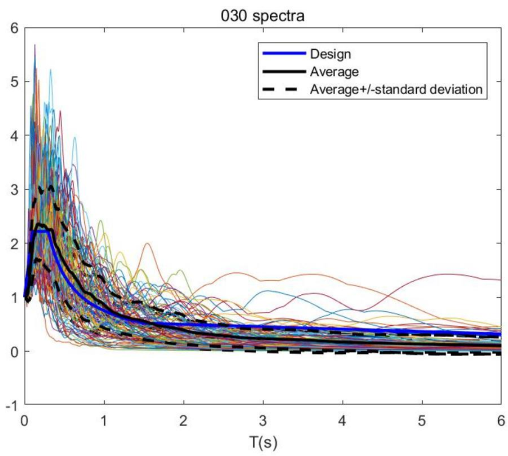

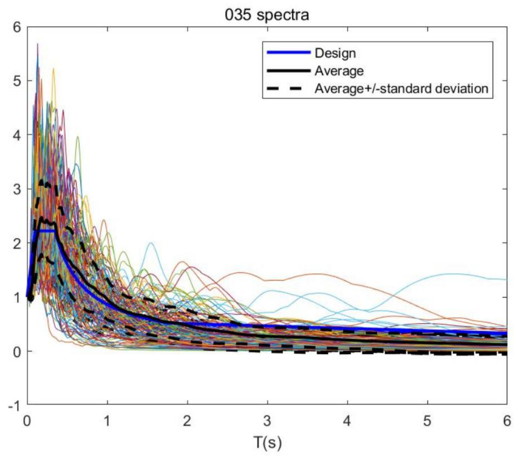

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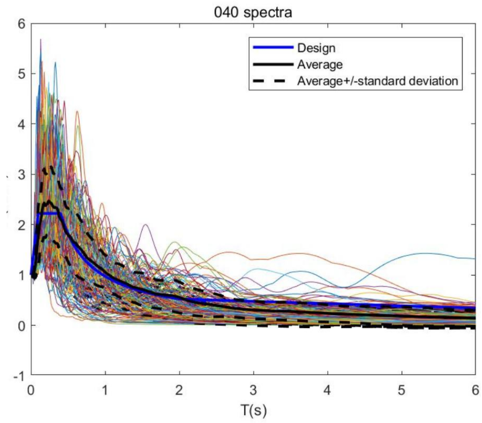

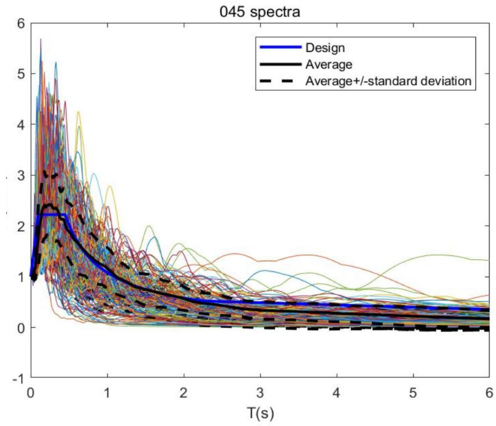

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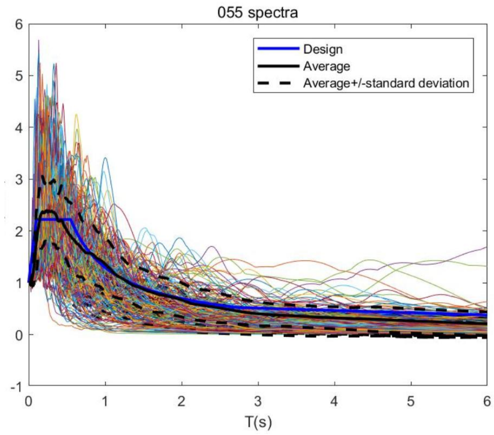
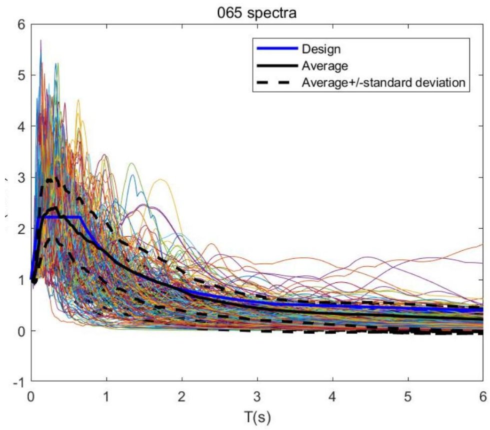
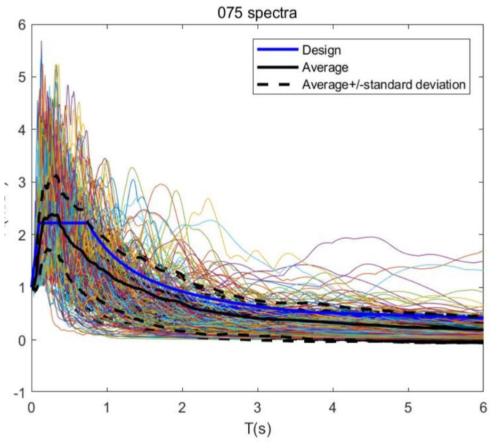
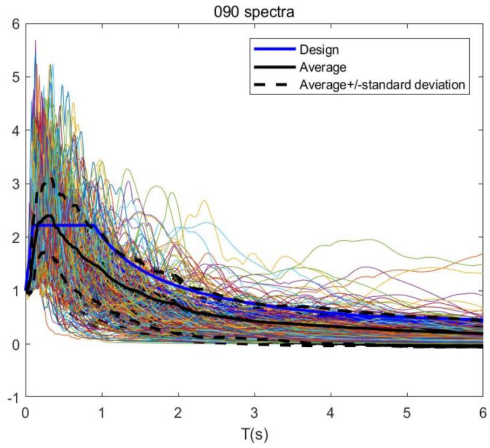




(g) (h)





(i) (j)

Fig 1- the normalized spectra of selected ground acceleration records in each group

2.2 Selection of steel structural models

There are 23 models in this study, including steel frame structure, concentrically braced frame structure and eccentrically braced frame structure. All the structural models are derived from existing buildings in China. These models have different heights and natural period. The models are divided into three major groups based on basic period according to Zhai C. and Xie L. (2005) as listed in the reference. For each groups, they are divided into three sub groups.

i. models with short period (0s$<$T$\leq$0.5s)

(0s$<$T$\leq$0.1s), (0.1s$<$T$\leq$0.3s), (0.3s$<$T$\leq$0.5s)

ii. models with medium period (0.5s$<$T$\leq$1.5s)

(0.5s$<$T$\leq$0.8s), (0.8s$<$T$\leq$1.1s), (1.1s$<$T$\leq$1.5s)

iii. models with long period (1.5s$<$T$\leq$5.5s)

(1.5s$<$T$\leq$2.5s), (2.5s$<$T$\leq$4.0s), (4.0s$<$T)

The damping ratio of 4% is assumed for all models in elastic analyses. The existence of in-filled walls is not considered in the models, anyway a periodic reduction coefficient 0.9 is adopted in analyses. In these models, the assumption of rigid floor is compulsory. The steel density is 78 kN/m^3. The reduction coefficient of the stiffness of the connecting beam is 0.6. The effective modal mass coefficient in the first seismic direction EX and in the second seismic direction EY are respectively more than 90%. According to *Code for seismic design of buildings* (GB50011-2010)(2016 edition), the effective mass coefficient is sufficient.

Table 1-model information

Model			**T_1(s)**	**T_2 (s)**	**T_3 (s)**	**Tg(s)**	**Height (m)**	**Floor**	**No.**
Model type	**Seismic precautionary intensity**	**Designed basic seismic acceleration (*g*)**							
Steel frame	6	0.05	1.995	1.872	1.634	0.25	21.9	6	1
	6	0.05	2.639	2.632	2.218	0.25	86.8	28	2
	7	0.1	1.776	1.745	1.544	0.9	37.35	7	3
	7	0.1	2.677	2.583	2.194	0.55	53.65	16	4
	8	0.2	1.532	1.125	1.028	0.25	74.2	22	5
	8	0.2	2.609	2.593	2.24	0.4	56.4	15	6
	9	0.4	1.74	1.64	1.36	0.25	88.7	25	7
	9	0.4	1.103	1.068	0.886	0.25	63.5	18	8
Steel eccentrically braced frame	6	0.05	2.32	2.139	1.855	0.45	51.92	12	9
	6	0.05	3.222	3.077	2.533	0.45	64.4	16	10
	6	0.05	4.407	4.276	3.49	0.45	81	19	11
	7	0.1	2.33	2.004	1.421	0.25	71.7	21	12

	7	0.1	2.706	2.69	2.267	0.55	47.05	16	13
	8	0.2	2.242	1.623	1.492	0.4	56.4	15	14
	8	0.2	2.769	2.666	2.464	0.25	51.3	12	15
Steel concentrically braced frame	6	0.05	1.693	1.37	1.347	0.25	115.2	28	16
	6	0.05	2.821	2.597	2.149	0.25	194.4	47	17
	7	0.1	1.462	1.394	1.256	0.9	33	7	18
	7	0.1	3.527	3.491	3.143	0.9	98.4	23	19
	8	0.2	2.069	1.961	1.536	0.4	68.6	17	20
	8	0.2	2.901	2.835	2.287	0.4	85	22	21
	8	0.2	4.057	3.964	3.04	0.35	284.8	62	22
	9	0.4	2.018	1.756	1.431	0.25	145.8	34	23

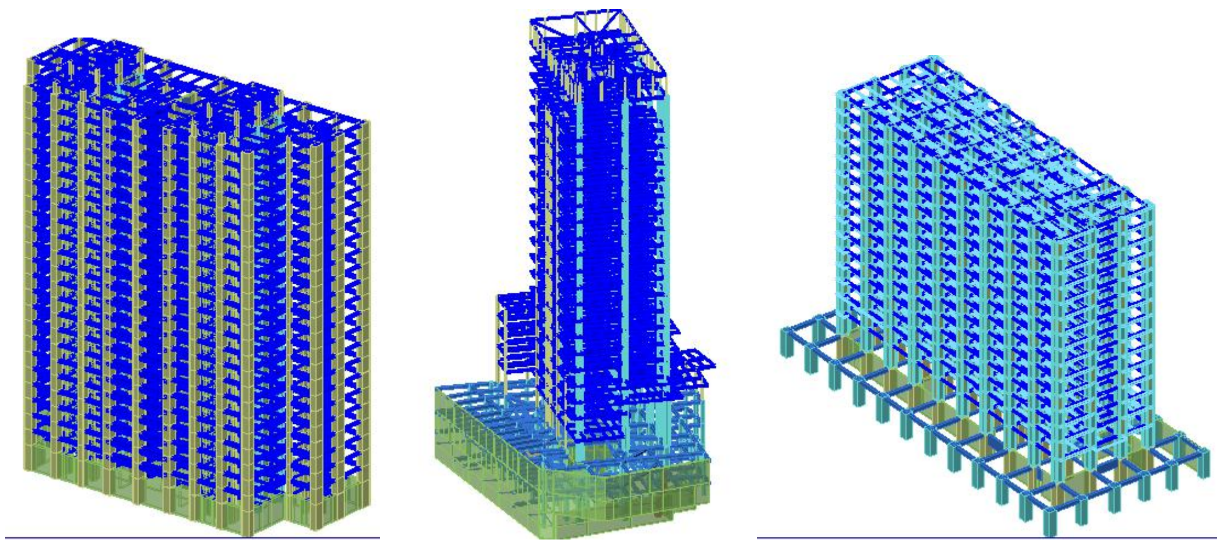


Fig 2- finite element model examples of 23 steel structures studied

2.3 Elastic time history analysis

Chinese version structural design software PKPM is used in the elastic time history analysis. The relationship between the required peak ground acceleration in the principal direction and the seismic precautionary intensity of the region the model located is shown in table-2.

Table-2 peak ground acceleration in principal direction of ground motion with different seismic precautionary intensity

Seismic precautionary intensity	**intensity 6**	**intensity 7**	**intensity 8**	**intensity 9**
peak acceleration in principal direction of ground motion(cm/s^2)	18	35	70	140

In the time history analysis, the input PGA ratio in principal, secondary and vertical direction is 1:0.85:0.65. According to *Code for seismic design of buildings* (GB50011-2010)(2016 edition) , vertical seismic action is required for large span or long cantilever structures with seismic precautionary intensity 8 or 9 and high-rise buildings with seismic precautionary intensity 9. In this study, the models are all high-rise buildings. The vertical seismic action are only calculated for models with seismic precautionary intensity 9.

Based on the height of each structure, the models are divided into two groups. The models in the first group have more than 10 stories. In this group, 5 floors acceleration responses are chosen to be studied deliberately. These 5 floors include the top floor, 3 quarters of the total number of floor, half of the total number of floor, one quarter of the total number of floor and the first floor. The rest of the models belong to the second group. In this group, 3 floors acceleration responses are chosen to be studied. These 3 floors include the top floor, half of the total number of floor and the first floor.

For studying the floor acceleration response spectra, the acceleration responses at the geometric center point of each selected floor is taken as the representative of the floor response calculated from the elastic time history analyses. In the study, the torsional effects caused by both of the random asymmetric or the asymmetric distribution between the stiffness and the floor mass are not taken into account for simple reason. For models with seismic precautionary intensity 9, the floor acceleration response time-history in principal direction, secondary direction and vertical direction will be output. For models in the region with seismic precautionary level 6, level 7, level 8, the floor acceleration time-history only in principal direction and secondary direction will be output. Floor acceleration response spectra are conducted based on the floor acceleration response time-history. Each graph shows the floor acceleration response spectra in a certain direction at a specific floor, under the action of 100 ground motions. There are 105 curves in each graph, including 100 floor acceleration response spectra curves, the upper and lower envelope spectra curves, average spectra curves, and the spectra corresponding to ±20% standard deviation of these acceleration response spectra curves.

2.4 Result and discussion

The elastic time history analysis results in the principal direction of a 22-floor, steel frame model with seismic precautionary intensity 8 are shown here. The first period is 1.53 second, and the second period is 1.13 second. The site characteristic period is 0.25 second. The 1st floor, 7th floor, 12th floor, 17th floor and 22nd floor are chosen to output the result. In the figure, the abscissa is period and the ordinate is the acceleration.Here are results of elastic time history analysis in the fig-3 to fig-7.

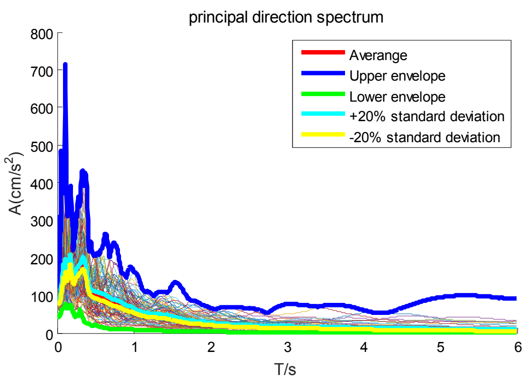


Fig -3 floor acceleration response spectra of the first floor in principal direction

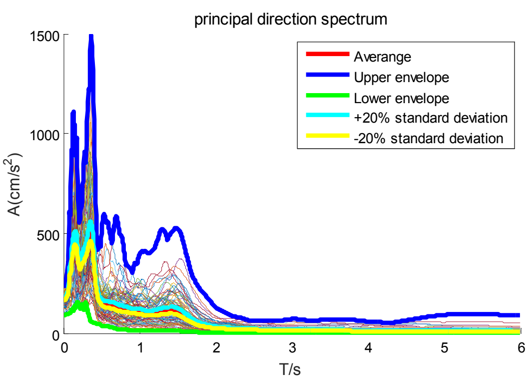


Fig-4 floor acceleration response spectra of the 7th floor in principal direction

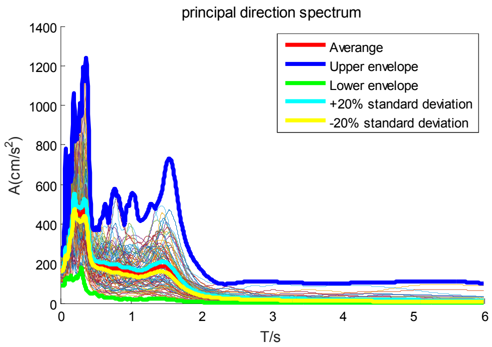


Fig-5 floor acceleration response spectra of the 12th floor in principal direction

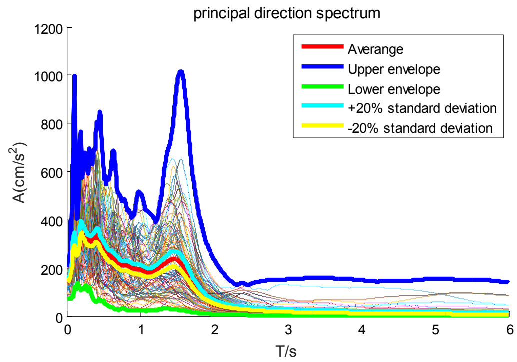


Fig-6 floor acceleration response spectra of the 17th floor in principal direction

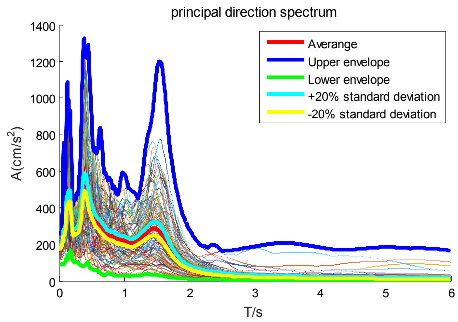
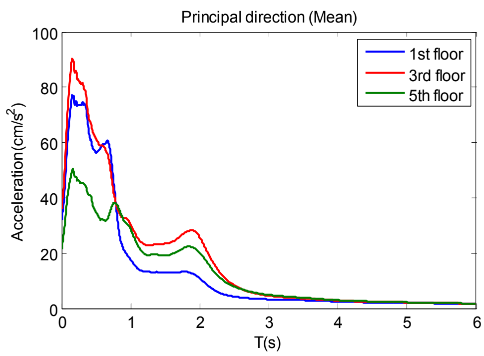


Fig-7 floor acceleration response spectra of the 22nd floor in principal direction

Due to the ground motion and model resonance, the response spectrum curve will increase sharply near the basic period of the building, resulting in a peak value.

The elastic time history analysis results of 23 steel structures models are shown in the figure-8. The spectrum curves in the fig-8 are the average spectrum curves of 100 floor acceleration response spectra under the action of 100 ground motions on each floor.



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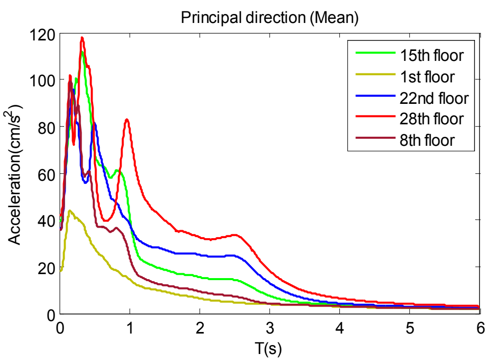

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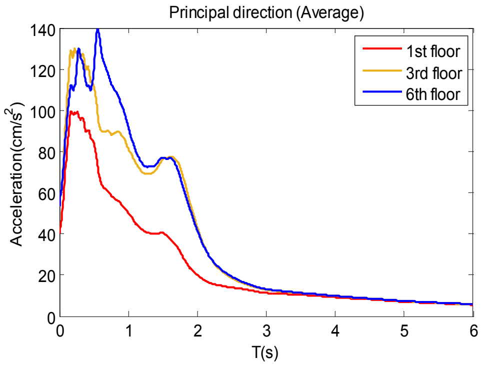

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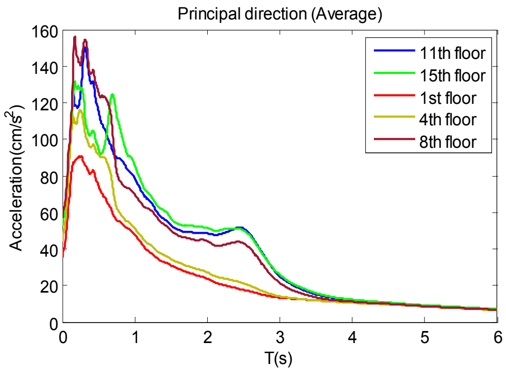

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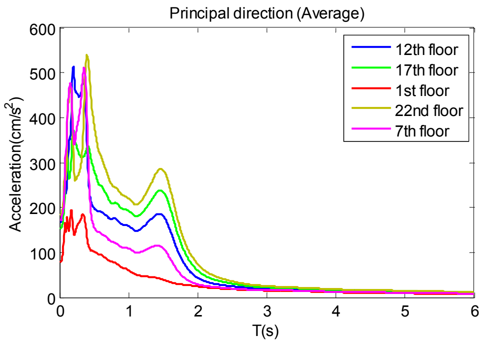

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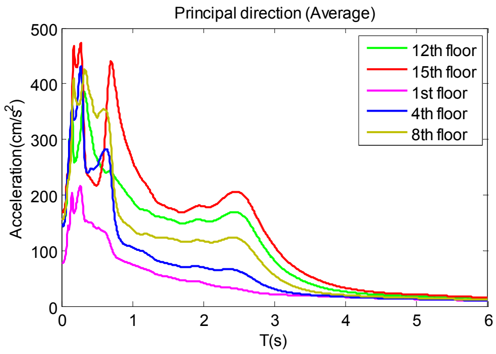

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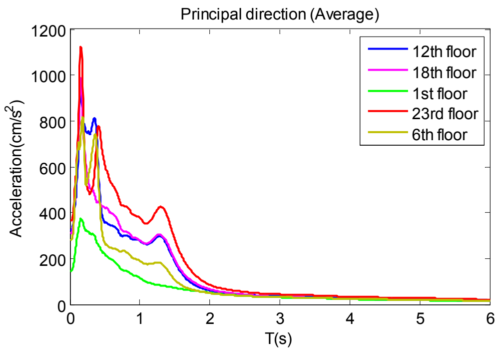

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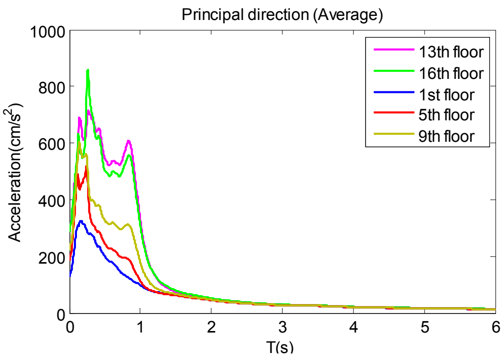

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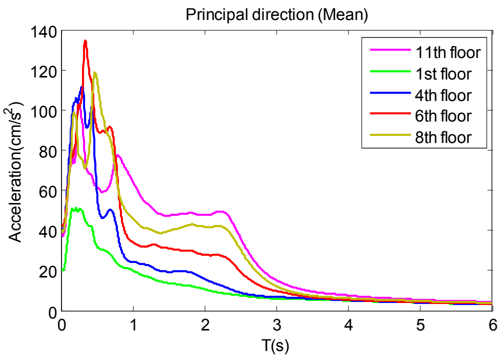

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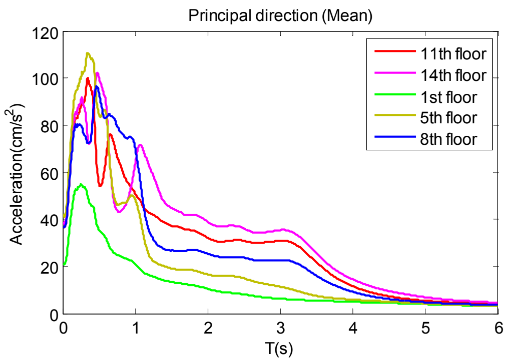

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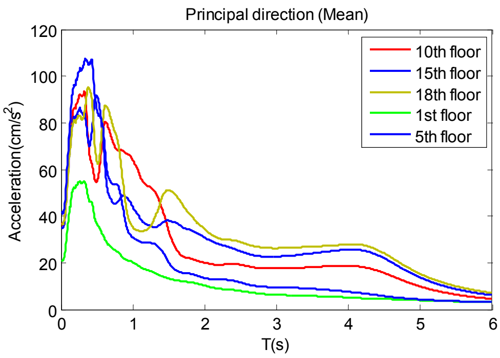

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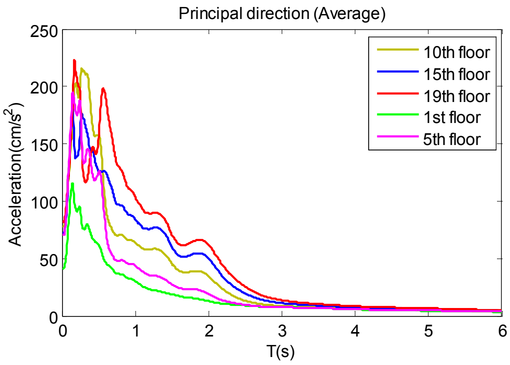

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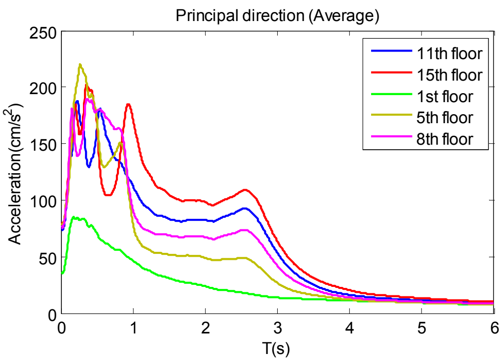

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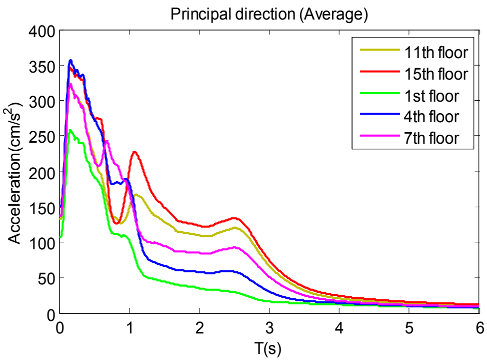

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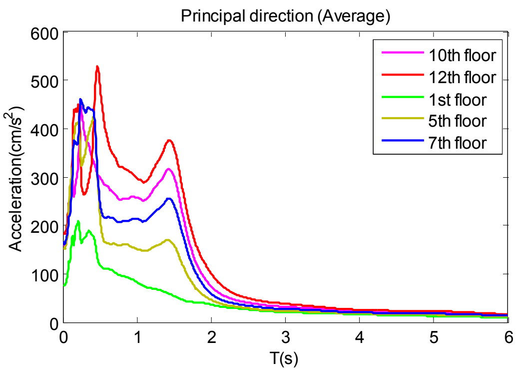

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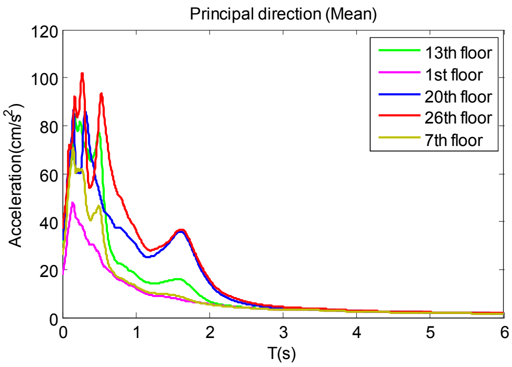

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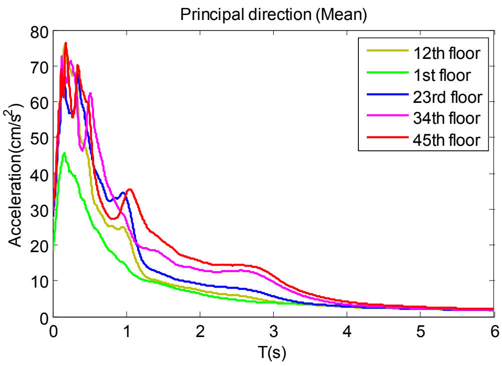

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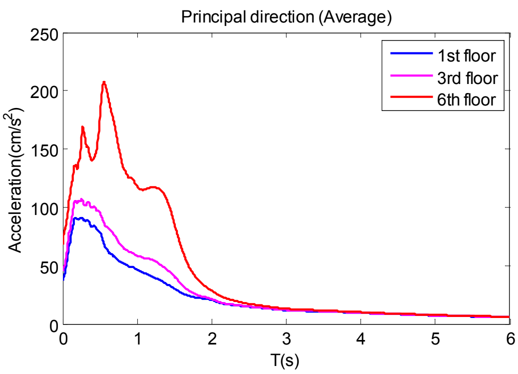

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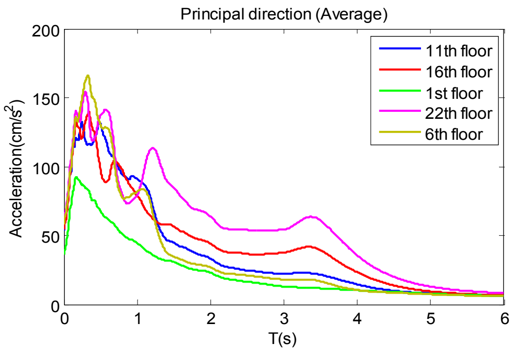

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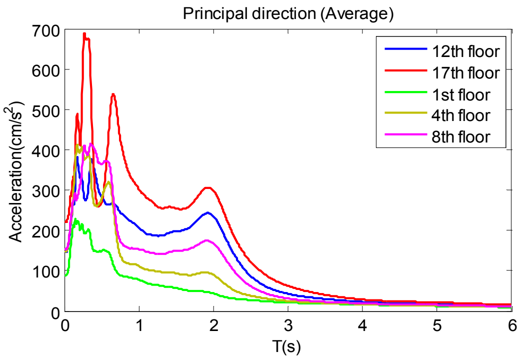

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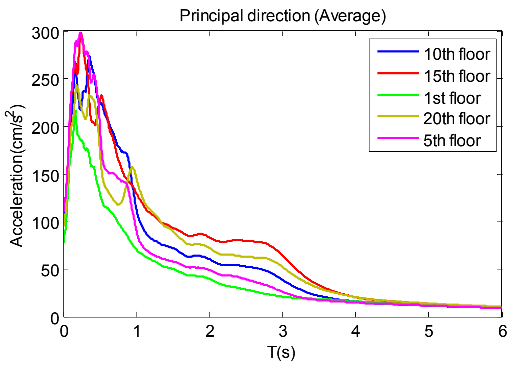

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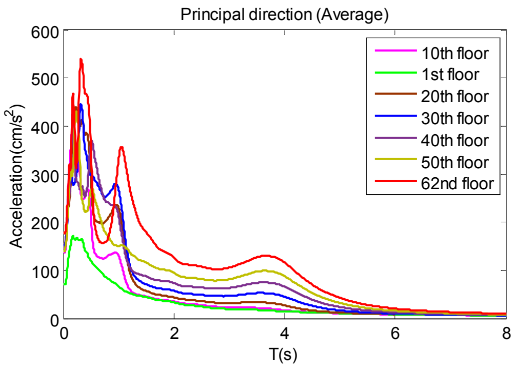

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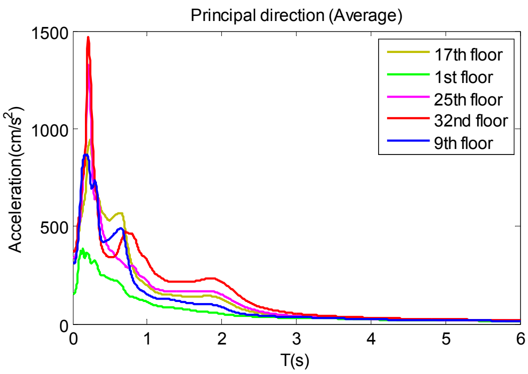

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Fig-4 the mean floor acceleration response spectra in principal direction of each floor of 23 steel structures. The number below the figure corresponds to the number of model in table 1.

Due to the dynamic filtering effect, the primary structure modifies the frequency content of the earthquake.[4] When the period is close to the vibration period of the elastic model, the base acceleration with a large frequency content amplifies the floor response. In the figure, a second peak value appears in the floor acceleration response spectra when T is approximately equal to T_1.

And another rule can be observed is that the peak value of floor acceleration response spectra does not always increase as the number of floor increase. High-order mode shapes have little difference for low buildings. But high-order mode shapes have significant effect on hige-rise buildings.

3. Summary

In this study, dynamic time history analysis for 23 steel structures are conducted. These 23 models are designed based on *Code for seismic design of buildings* (GB50011-2010), and each model is subjected to 100 ground motions.All the ground motions are obtained by comparing with the standard design spectrum.

In this study, time history analysis are conducted on 23 steel structures without infull wall. The first floor, one quarter floor, half floor, three quarter floor and top floor are selected to output result. And the average spectrum of these 5 floors are compared in the figure. For some high-rise building,the peak value of floor acceleration response spectra does not always increase as the number of floor increases. High-order mode shapes have significant effect on hige-rise buildings. There is generally a second peak in the acceleration response spectrum, and the acceleration response increase as the floor increase.

The results of this study are absolute acceleration response spectra. These result can be conducted on the non-structural components as input to do research on the seismic capacity of non-structural components.

4. Acknowledgements

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5. Copyrights

6. References

[1]Yusa Y, Takemura K, Kitaoka K, Kamiyama K, Horie S, Nakagawa I, Kobayashi Y, Kubotera A, Sudo Y, Ikawa T,Asada M (1992) Subsurface structure of Beppu Bay (Kyushu, Japan) by seismic reflection and gravity survey. Zisin (Bull. Seismol. Soc. Jpn.) 45, 199-212.

[2]Oita prefecture (2008): Progress report on earthquake damage estimation in Oita prefecture. pp.230.

[3]P.M.Calvi,D.M.Ruggiero(2017)Earthquake-induced floor accelerations in base isolated structures.*16th World Conference on Earthquake,Santiago Chile.*

[4]C.Perone, G.Magliulo, G.Manfredi (2016): Floor response spectra in RC frame structures designed according to Eurocode 8. *Bull Earthquake Eng (2016)*, Department of Structures for Engineering and Architecture, University of Naples Federico Ⅱ, Via Claudio 21,80125 Naples, Italy.

[5]Dhakal RP (2010): Damage to non-structural components and contents in the 2010 Darfield earthquake.

B. Earthq. Eng., 43(4), 404-411.

[6]Filiatrault A, Sullivan TJ (2014): Performance-based seismic design of nonstructural building components: The next frontier of earthquake engineering. Earthquake Engineering and Engineering Vibration, 13(1), 17-46.

[7]Fagà E (2013): A precast composite technology for seismic design of multi-storey buildings. Istituto Universitario di Studi Superiori di Pavia - IUSS.

[8]Yasui, Y ， Yoshihara, J ， Takeda, T. (1993) Direct generation method for floor response spectra, PROCEEDINGS OF THE JAPAN EARTHQUAKE ENGINEERING SYMPOSIUM

[9] Code for seismic design of buildings (GB50011-2010) (2016 Version)

[10] Vukobratović, V. and Fajfar, P. (2012). "A Method for Direct Determination of Inelastic Floor Response Spectra," Proceedings of the 15 th World Conference on Earthquake Engineering, Lisbon, Portugal,Paper No. 727.

[11] Fajfar, P. and Novak, D. (1995). "Floor response spectra for inelastic structures," Transactions of the 13th International Conference on Structural Mechanics in Reactor Technology (SMiRT 13), Porto Alegre, Brazil, K044/1, 259-264.

