SEVERAL SUGGESTIONS FOR THE REVISION OF CODE FOR SEISMIC DESIGN OF BUILDINGS, GBJ11-89, OF CHINA

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SUMMARY

The paper presents several suggestions for the revision of Code for Seismic Design of Buildings, GBJ11-89, of China. The first one is about the quantificational index of the levels of design ground motion, the reasons for abnegating the present index, the fortification intensity, are described and the peak acceleration is suggested to be an appropriate substitution. The second one is about the revision of the design response spectra. According to the results of statistical and theoretical research, the paper gives four suggestions about the long period design spectra, the method to scale the spectra with different critical damping ratios, the method to proportion the spectra between the minor and the major seismic action, and the three-dimensional design spectra. The last suggestion is about the most difficult thing to execute time history analysis in practice, that is the chosen of excitations from a database of strong ground motion records. Based on the results of time history analysis of several non-linear structures, the selecting standard of excitations is determined by a principle that the standard deviation is not greater than the mean value. With the probability of reliability of several samples not excessively less than that of a great number of samples, the minimum number of earthquake accelerograms and the minimum number of artificial waves are suggested.

INTRODUCTION

The present code of China, Code for Seismic Design of Buildings, GBJ11-89 [1989], has been executed for about ten years. It should be regarded as one of the most representative publications concentrating the achievements of the seismic research work since later 1970’ to later 1980’ in China. Since there are a lot of lessons learned from recent earthquakes and some new results in earthquake engineering investigations, the code should be revised. This work started from July 1997 and will be finished by 2000. The second author is a member of the reviser group, and the authors are responsible for the revision of several provisions concerning the seismic action and seismic checking for structures in the code. Most of the research work has been done.

In the last decade, a number of problems in the present code emerged from the practical earthquake resistant design. The quantificational index of the level of design ground motion, the design response spectra and the time history analysis method are three important problems that should be improved in the future code. Limited to paper length, only the above-mentioned problems are discussed and the relevant suggestions are presented based on the results of statistical and theoretical research.

THE QUANTIFICATIONAL INDEX OF THE LEVEL OF DESIGN GROUND MOTION

The basic philosophy that the buildings designed to be earthquake resistant should perform different behaviors under different earthquake actions has been accepted widely. To attain the goal of this basic philosophy, the Code GBJ11-89 provides three levels of seismic action. Under the action of the first level, named minor

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earthquakes or frequently occurred earthquakes, the building will be in elastic state with no damage or will be only slightly damaged and will continue to be serviceable without repair; under the action of the second level, named moderate earthquakes, the building will be in elastio-plastic state with reparable damage; under the action of the third level named, major earthquakes or seldom occurred earthquakes, the building will pass from ealsto-plastic state into plastic state and suffer irreparable damages but will neither collapse nor suffer failures that would endanger human lives. Though these criteria have been accepted by many countries, the quantification of these three levels is different for different country [Hu, 1993]. The seismic intensity is the index to quantify these levels in the code.

In order to define the values of minor and major earthquakes using probability method, seismic hazard analysis and statistical analysis were carried out based on data collected from 45 cities and towns scattered in China. It was found that the probabilistic distribution of seismic intensity fits with that of extreme-value distribution of type III. The minor, moderate and major earthquake are defined as the intensity having a probability of exceedance of 63.2%, 10% and 2% to 3% in a 50-year period, respectively. The intensity of a moderate earthquake is also called the basic intensity and is given in China Seismic Intensity Zoning Map (1990). The fortification intensity is that approved by State authority to be used as a basis for the seismic protection of a region. Normally, The fortification intensity is equal to the basic intensity. In average, the intensity of a minor earthquake is 1.55 degrees lower than the basic intensity, the intensity of a major earthquake is approximately one degree higher than the basic intensity. The code is applicable to areas with fortification intensity of 6 to 9 degrees. The peak acceleration PGA is proportional to intensity in the code. Only minor and major earthquakes are used in calculation.

The shortcomings of the intensity were revealed in last practical seismic design and were discussed by some investigators [Hu, 1993]. First of all, it causes conflict of conception. It is well known that the intensity is defined as a comprehensive index describing the natural macroscopically phenomenon and the sense perception of human during an earthquake, there is no meaning to make the intensity decimal. So it can be inferred that buildings suffer approximately same damage in regions of same intensity. But the contrary conclusion can be obtained according to the code, because there are different design spectrum, which means different damage, with the same intensity. Secondly, the intensity may be an appropriate index of seismic risk but not a good index of seismic hazard in the sense of conception, while the level of seismic action should be seismic hazard. Thirdly, there is no good correlation between the intensity and any factor of strong ground motions such as PGA, PGV or PGD [Hu, 1988], the hypothesis that PGA is proportional to intensity is very rough. Lastly, the integer intensity makes the peak acceleration not continuous, the PGA values double when the seismicity moves up by one degree from 6 to 9, so it may be not economic for some regions.

Another appropriate index should be chosen to substitute the present intensity to be the quantification index of the levels of seismic action. Compared with the equivalent peak acceleration, EPA, which is used in UBC Code and others, PGA is suggested to be a proper index for the following reasons. The first one is the requirement of ensuring the continuity of the present code and the future code. The concept of seismic intensity has been used in China for a long time, it is difficult for engineers to accept a new index completely differing from the intensity. Since Chinese engineers have habitually used the transformation between the intensity and PGA value in seismic design, it is acceptable and needs a little work to do to substitute the intensity with PGA as the quantification index in the revision of the code. The second one is related to the significance of PGA. The PGA value is equal to the spectral value when the natural period of a structure is zero, and PGA is regarded as an indispensable parameter characterising the design spectra. The third one is that the shortcomings of the intensity mentioned above may be overcomed.

To define the design spectra, PGA or EPA is not enough. The reason that some countries use EPA to quantify the seismicity of a region is partly that EPA together with EPV can determine the characteristic period of the design spectrum. It is also feasible to use PGA and the characteristic period to characterise the design spectra. Being individual parameters, two zoning maps of PGA and the characteristic period of China will be developed in the coming future. That will provide the base to use PGA directly in the future code to quantify the levels of seismic action.

THE DESIGN RESPONSE SPECTRA

The design response spectra defined by the present code are characterised by two factors: the maximum value of the seismic influence coefficient, α_{max}, depending only on the fortification intensity, and the characteristic period, T_g, depending on the category of the site and the epicenter distance (see Table 1). The α_{max} is equal to the maximum value of magnification factor β_{max} times the ratio of PGA to gravity acceleration, where β_{max} is 2.25. Four categories of building site are classified according to the stiffness and the overlying thickness of site soil. The epicenter distance is considered only to be near or far. The spectrum curve is only suitable for the critical damping ratio of 5% and period ranging from 0 to 3 seconds. The seismic influence coefficient changing with the natural period has a lower limit of 0.2 of α_{max}, which means the spectral value is not less than 0.45PGA.

It is an urgent task to revise the design spectrum in the following aspects that are not enough to satisfy the requirements of the present seismic design.

The Design Response Spectra With Long Period

As the buildings with long period are avalanching at the present day in China, the design response spectra with long period have attracted many researchers. Based on 1735 horizontal components of strong ground motions records obtained mainly in USA, the authors and their assistants have analysed the statistical characteristics of long period response spectra. In the process of analysis, the normalised acceleration response spectra were calculated with the period range from 0 to 7 seconds, and were arranged into 8 groups, corresponding to 4 site categories and near/far earthquakes, according to their first characteristic periods. The curve-fitting method was used to determine the characteristic periods for every spectrum sample. The detailed process and the relevant conclusions are given in the reference [Xiang and Li, 2000]. It should be mentioned that the digressive segment of the normalised spectra may be divided into two parts according to the second characteristic period, the former is controlled by PGV with slow attenuation while the latter is controlled by PGD with quick attenuation. As the first characteristic period increasing, the length of the former part increases and the latter part will disappear.

To revise the present design spectra, the following principles should be obeyed: the continuity between the present and the revised spectra, the simplicity of the spectrum curve, and the statistical property. Based on the present design spectra and the results obtained from the statistical analysis, the commonly used model of design spectrum developed by Newmark and Hall is here employed and improved. The suggested shape of the design spectra with long period, described by seismic influence coefficient, can be expressed as the following,

$$\alpha = \begin{cases} 0.45\alpha_{max} + 5.5\alpha_{max} \cdot T & 0 \le T \le 0.1\text{s} \\ \alpha_{max} & 0.1\text{s} \le T \le T_{g1} \\ \alpha_{max}(T_{g1}/T)^{a_1} & T_{g1} \le T \le T_{g2} \\ \alpha_{max}(T_{g1}/T_{g2})^{a_1}(T_{g2}/T)^{a_2} \ge \alpha_{min} & T_{g2} \le T \le 7.0\text{s} \end{cases} \qquad \text{(Eq. 1)}$$

where T is the natural period, T_{g1} and T_{g2} the first and the second characteristic period, $a_1$and a_2 the exponents, and α_{min} the minimum value of seismic influence coefficient. T_{g1} should be the same as the present T_g and be written as T_g directly in the following text. a_1 is suggested to be depressed from the present of 0.9 to 0.8 since the spectral values in the segment of middle frequency are smaller than some other codes. a_2 may be 2 times a_1 according to the relationship between the pseudo spectra. α_{min} is depressed from the present of $0.2\alpha_{max}$ to $0.1\alpha_{max}$ to avoid that the flat part of α_{min} appears before T_{g2}. T_{g2} is determined by the curve-fitting method. Table 1 shows the suggested values of T_{g2} together with the values of T_g defined by the present code.

Figure 1 shows the comparison of the suggested, the current and the statistical spectra. It is found that the suggested spectra is not less than the average except in the flat part including the maximum and is close to but not much larger than the average plus one standard deviation except in the flat part including the minimum.

Table 1 Characteristic Periods of Horizontal Spectral Curves

Category of Site	I		II		III		IV	
Epicenter Distance	T_g (s)	T_{g2} (s)	T_g (s)	T_{g2} (s)	T_g (s)	T_{g2} (s)	T_g (s)	T_{g2} (s)
Near Earthquake	0.20	2.00	0.30	2.60	0.40	3.50	0.65	5.00
Far Earthquake	0.25	2.20	0.40	3.00	0.55	4.00	0.85	6.00

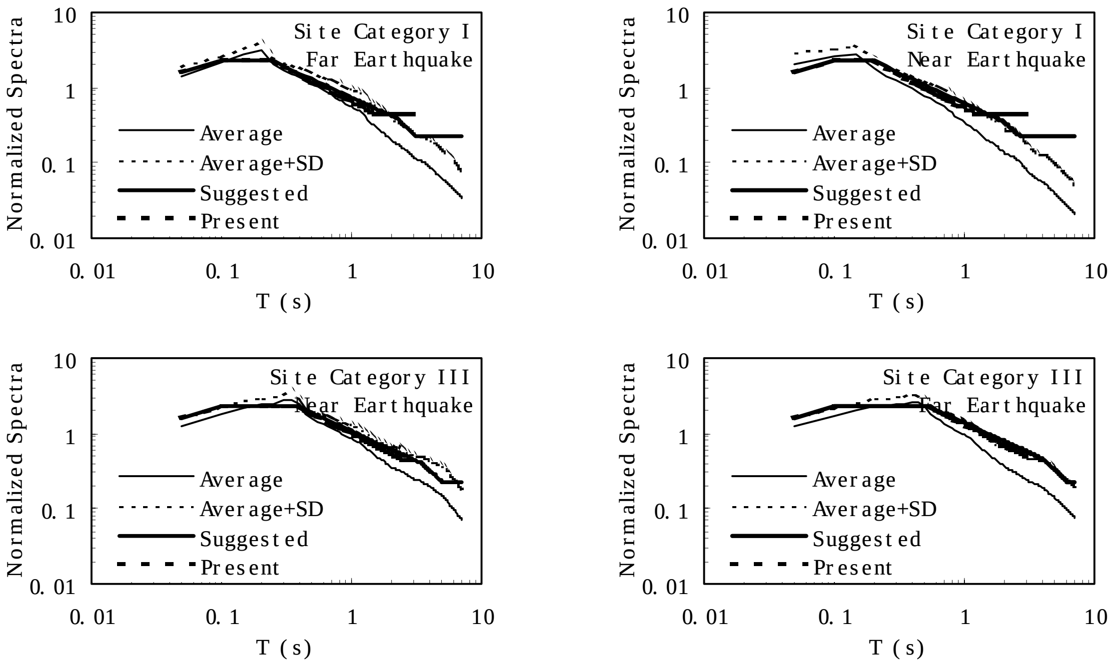

Fig. 1 Comparison of the Suggested, the Current and the Statistical Spectra

Scaling The Spectra With Different Critical Damping Ratios

As we know, the different structures may have different critical damping ratios. For RC structures, the value of damping ratio is about 5%; for steel structures, the value may be less than 5%; for the isolation structures, the value may be much large than 5%. To design a structure without 5% damping ratio, it needs the design spectrum with relevant damping ratio.

There have been a lot of literatures dealing with the relationship among response spectra with different damping ratios. The method to scale the spectra with different damping ratios can be divided into two categories: the period dependent and the period independent. The former is simple but the latter seems to be more reasonable.

Based on the same data as the above, the relationship of multi-damping ratio spectra is analyzed. The result shows strongly that the ratio of spectral value of 5% damping to that of other damping is period dependent, the ratio increases with the period increasing to a certain value of about 0.2 to 0.5 second and then decreases. The condition of site and the epicenter distance have a little influence on the ratio and can be ignored. The formula defining the relationship is calculated. Considering that the shape of the scaled spectrum must be continuous and should be similar to that of the 5% damping ratio spectrum, spectrum adjusting coefficient for damping ratios $\eta(\xi, T)$ is suggested as the following,

$$\eta(\xi, T) = a(\xi) \cdot T + b(\xi) \qquad \text{(Eq. 2)}$$

where ξ is the critical damping ratio. The coefficients, $a(\xi)$ and $b(\xi)$, are listed in Table 2. It can be found that the flat parts corresponding to the maximum and minimum spectral values are kept after scaled.

Scaling The Spectra Of Different Levels Of Seismic Action

The difference between a major earthquake and a minor earthquake defined in the code, if expressed in degrees of intensity, is 2.55 degrees in average; if expressed in α_{max} or PGA, the former is about 4 to 6 times the latter. The difference between the design spectrum of the two levels is only reflected in α_{max} or PGA and is period dependent. That is to say, the normalised spectra of the two levels are the same. The provisions similar to that of the code are defined in some other countries' code.

Table 2 The Spectrum Adjusting Coefficients for Damping Ratios $\eta(\xi, T)$

T	0 to 0.1s		0.1s to T_g		T_g to T_s *		T_s to 7.0s	
ξ	$a(\xi)$	$b(\xi)$	$a(\xi)$	$b(\xi)$	$a(\xi)$	$b(\xi)$	$a(\xi)$	$b(\xi)$
0.5%	7.0	1.0	0.0	1.70	-0.06	$1.70+0.06T_g$	0.0	$1.70+0.06(T_g-T_s)$
2%	3.0	1.0	0.0	1.30	-0.02	$1.30+0.02T_g$	0.0	$1.30+0.02(T_g-T_s)$
10%	-1.5	1.0	0.0	0.85	0.02	$0.85-0.02T_g$	0.0	$0.85-0.02(T_g-T_s)$
20%	-3.0	1.0	0.0	0.70	0.04	$0.70-0.04T_g$	0.0	$0.70-0.04(T_g-T_s)$

Notation: T_s is the minimum value of period where the seismic influence coefficient is α_{min}, $T_s \le 7.0$s.

There are two inappropriate conclusions can be educed from the above provision of the spectra of different levels. Firstly, the provision is conflict with the general attenuation laws. Generally, a minor earthquake is usually caused by an event of larger magnitude with shorter epicenter distance while a major earthquake is usually caused by an event of smaller magnitude with longer epicenter distance. Indeed, the influence of the epicenter on the shape of spectrum is much smaller than that of the magnitude. The provision implies that the shape of spectrum is magnitude dependent, while the attenuation laws show that the shape of spectrum depends on the magnitude. Secondly, the provision may result in the inconsistency of seismic design standard for structures with different periods. According to the results of seismic hazard analysis [Cornell, 1968], the hazard of structures with short periods will decrease much more than that of structures with long periods. So the present method to scale the design spectrum of minor earthquake to that of major earthquake by PGA is not appropriate and should be re-evaluated. Unfortunately, little attention to the method to scale the design spectra of different levels of seismic action has been paid.

Ignoring the difference between the acceleration response spectra and the pseudo acceleration response spectra, the standard model for scaling the design spectra by PGA, PGV and PGD developed by Newmark and Hall [Newmark and Hall, 1969] can be improved to be the following,

$$S_a = PGA \cdot \beta_a = PGA \cdot \begin{cases} 1 + b_1 \cdot T \cdot PGA / PGV & 0 \le T \le T_1 \\ b_2 & T_1 \le T \le T_g \\ b_3 \cdot T^{-r} \cdot PGV/PGA & T_g \le T \le T_{g2} \\ b_4 \cdot T^{-2r} \cdot PGD/PGA & T_{g2} \le T \le 7.0s \end{cases} \quad \text{(Eq. 3)}$$

where S_a is the acceleration spectrum, β_a is the magnification factor, T_1 is the start point of the range controlled by acceleration, b_1 to b_4 and r are constant. It is obvious that the difference between the design spectrum of two levels is period dependent: in high frequency range, the ratio depends on PGA; in middle frequency range, the ratio depends on PGV; and in low frequency range, the ratio depends on PGD. As a result, the characteristic periods of a major earthquake are different from those of a minor earthquake. Let the superscript “L” and “S” is representation of major and minor earthquake respectively, the following expression can be obtained from Eq. 3,

$$\frac{T_1^L}{T_1^S} = \frac{PGV^L}{PGV^S}\frac{PGA^S}{PGA^L}, \quad \frac{T_g^L}{T_g^S} = \left(\frac{PGV^L}{PGV^S}\frac{PGA^S}{PGA^L}\right)^{1/r}, \quad \frac{T_{g2}^L}{T_{g2}^S} = \left(\frac{PGD^L}{PGD^S}\frac{PGV^S}{PGV^L}\right)^{1/r} \quad \text{(Eq. 4)}$$

It is difficult and realistic to determine the relationship between the characteristic periods by using a great deal of results of seismic hazard analysis. Another approach is developed to estimate approximately the relationship. In general, an available attenuation law can be expressed as,

$$\lg Y = a_Y + b_Y M + c_Y \lg(R + d_Y \exp(e_Y M)) + f_Y S \quad \text{(Eq. 5)}$$

where Y may be intensity, PGA, PGV or PGD, M is magnitude, R is epicenter distance or hypicenter distance, S is site index, and the others are constant corresponding to Y. Then, Eq. 5 may be rewritten as,

$$\begin{cases} \lg(T_1^L / T_1^S) = (b_{PGV} - b_{PGA})(M^L - M^S) + f(R^L, R^S, M^L, M^S) \\ \lg(T_g^L / T_g^S) = [(b_{PGV} - b_{PGA})(M^L - M^S) + f(R^L, R^S, M^L, M^S)] / r \\ \lg(T_{g2}^L / T_{g2}^S) = [(b_{PGD} - b_{PGV})(M^L - M^S) + f(R^L, R^S, M^L, M^S)] / r \end{cases} \quad \text{(Eq. 6)}$$

where f( ) is a function having little influence on the characteristic periods (see Figure 2). The left work is only to evaluate the possible range of difference of magnitude and epicenter distance between two levels based on the attenuation laws of intensity. The details are given in the reference [Li, 1999].

According to the results of calculation, it is found that the characteristic periods of the major earthquake are larger than those of the minor earthquake. For the limitation of paper length, only an example shown in Figure 2 is given to illustrate the relationship, and the conclusion is addressed directly as the following,

$$T_1^L = T_1^S = 0.1s, \quad T_g^L = 1.10 T_g^S, \quad T_{g2}^L = 1.10 T_{g2}^S \qquad \text{(Eq. 7)}$$

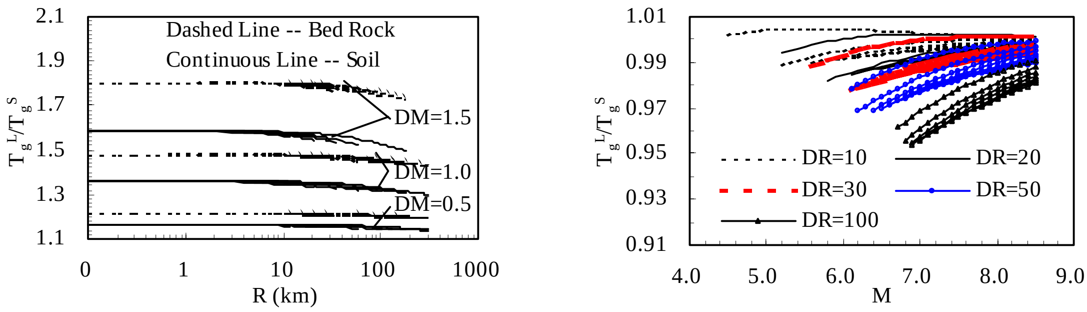

Fig. 2 The Influence of Magnitude Difference, DM and Epicenter Distance Difference, DR on Tg

The Three-Dimensional Design Spectra

For structures with obvious asymmetric and non-uniform mass and stiffness distribution, the responses of the structure subjected simultaneously to the influence of 3D seismic actions differ from the responses of the same structure subjected to the influence of 1D seismic action. It is necessary to provide the 3D response spectra for the seismic design of these structures. There is no relevant provision in the present code.

784 accelorograms with three-dimensional components recorded in the worldwide are used to study the correlativity of peak values and response spectra of three components. The formulae for correlation of peak values including PGA, PGV, PGD, PGV/PGA and PGA*PGD/PGV^2 are regressed by using the LS method and the uniform LS method in which the variables are both taken to be random. Since it is difficult to discuss the correlation of two curves, the normalized response spectra of each record is fitted with the standard spectrum model defined in Eq. 1, the correlation of response spectra may then be described by the correlation of parameters of the spectrum model.

The main conclusions from the results of the regressive analysis are: 1) the peak values and the parameters of the spectral model, except PGA*PGD/PGV^2 and T_{g2}, are all of acceptable correlativity; 2) there are distinct difference of spectral shape not only between any horizontal component and the vertical but also between one horizontal component and the other; and 3) the vertical seismic action may be larger than the horizontal one in for the case of major earthquake with small epicenter distance. Also for the limitation of paper length, the details of the relationships of three components, given in the references [Li et al., 1999; Yang, 1999], are left out here. Figure 3 illustrates the correlation of PGV/PGA values of two components calculated from the records. Since the characteristic period T_g is usually proportion to the value of PGV/PGA, the spectra of three components are correlated.

Taking the spectra suggested in section 3.1 as a standard horizontal spectrum, the 3D design spectra are determined by introducing the results above-mentioned into the standard one. Since the standard one is already not an actual spectrum but a design spectrum and the difference of the shape between two horizontal components may be involved partly in the adjusting the value of α_{max}, it is acceptable and convenient to maintain the other horizontal spectrum have the same shape with the standard one (see Table 1). For the vertical spectrum, the shape may be similar to the horizontal, but the characteristic periods, T_g and T_{g2}, are suggested to be the values defined in Table 1 minus 0.1second and 0.5 second respectively. The values of α_{max} for the suggested 3D design spectra are shown in Table 3.

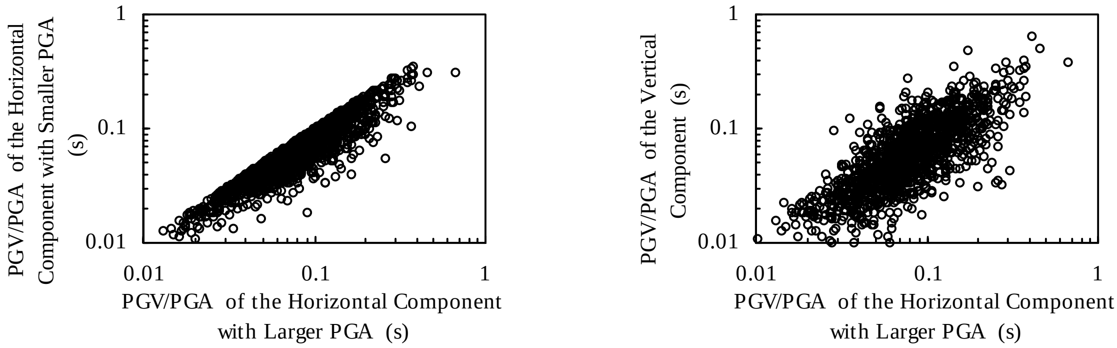

Fig. 3 The Correlation of PGV/PGA Values of Three Components

Table 3 Maximum Values of the Seismic Influence Coefficient for 3D Design Spectra

Epicenter Distance	Near Earthquake				Far Earthquake		
Fortification Intensity	6	7	8	9	7	8	9
Standard Horizontal Direction	0.04	0.08	0.16	0.32	0.50	0.90	1.40
The Other Horizontal Direction	0.05	0.10	0.20	0.40	0.60	1.10	1.70
Vertical	-	-	0.07	0.26	0.50	0.96	1.55

Notation: The values for the other horizontal direction are only used for the case of considering two horizontal seismic actions simultaneously.

THE PRINCIPLE FOR THE SELECTION OF EXCITATIONS FOR TIME HISTORY ANALYSIS

The time history analysis method is an alternative method to examine the deformations of structures subjected to the influence of a major earthquake in the present code, but is seldom used in practical seismic design. One of difficulties is the determining of earthquake ground motion excitation. There is on exercisable principle for the selection of accelerograms in the present code. In some other codes, it is required that the spectra of the selected accelerograms should be in good agreement with the design spectrum, but the standard is still fuzzy.

The problem falls into two aspects, one is the selecting standard of excitations and the other is the number of accelerograms. Based on the standard that the spectra of the selected waves should be in agreement with the design spectrum, four simplified schemes of selecting standard are developed since it is not realistic to control all point of the spectrum. The indices corresponding to the standards are the category of site, the characteristic period, two ranges of periods including T_g and the natural period of the structure, and the area under the spectrum before T_g. The differences between the indexes of the selected waves and those of the design spectrum are all less than 10%. The duration of the selected motion is required to be not shorter than 8 times the natural period of the structure for all schemes. If the structural responses are quite different from one excitation to the other, it will be difficult to determine the designing values. So the principle that the standard deviation of the responses corresponding to the selected waves is not greater than the mean value of the same responses is needed. The elasto-plastic responses of two actual structures, a 12-stoey RC frame building and a 30-story RC frame-wall building, are analysed to check the schemes, and the third one is the best. So, the selecting standard of excitations can be described as the following:

$$\overline{S}_S(T) \le (1 \pm 10\%)\overline{S}_D(T), \quad \text{while} \quad 0.1s \le T \le T_g \quad \text{and} \quad (T_1 - 0.2s) \le T \le (T_1 + 0.3s) \qquad \text{(Eq. 8)}$$

where T_1 is the fundamental period of the structure considered, $\overline{S}_S(T)$ and $\overline{S}_D(T)$ is the average of the spectrum of the selected accelerogram and the design spectrum in the defined ranges, respectively.

The number of the selected waves is then analysed. Since the actual records are not of statistical sense while the artificial waves are, the excitations should include both the actual and the artificial waves. Four acceptable schemes are compared by the elasto-plastic responses of the structures above-mentioned. They are 2 actual records + 1 artificial wave, 3 actual records + 1 artificial wave, 3 actual records + 2 artificial waves, and 4 actual records + 1 artificial wave. By the idea that the probability of reliability of several samples not excessively less than that of a great number of samples, the scheme, “3+1”, is better than the others and is suggested to be the minimum number of the selected waves. The details are given in the reference [Li et al., 1999].

CONCLUSIONS

Several suggestions about the quantification index of levels of seismic actions, the design spectra and time history analysis are presented. As a summary, the following points should be emphasized.

Intensity is not appropriate to quantify the levels of seismic actions, and PGA may be the substitution.

Four aspects should be considered for the revision of the present design spectra: 1) the horizontal spectra with long period may be defined as the model of Eq.1 with two characteristic periods given in Table 1; 2) the ratio of spectra with different critical damping ratios is period dependent, the scaling method is described in Eq. 2 and Table 2; 3) the spectral shapes of different levels of seismic actions are not same but period dependent, the relationship of the characteristic periods is described in Eq. 7; and 4) the 3D design spectra are obtained by adjusting the maximum values of seismic influence coefficient listed in Table 3.

The principle for the selection of accelerograms for time history analysis is suggested. The selecting standard of excitations may be described as Eq. 8 controlling two ranges of spectrum including respectively the characteristic period and the structural fundamental period. The minimum number of excitations for time history analysis may be 3 actual accelerograms plus 1 artificial wave.

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