THE SEISMIC BEHAVIOR ANALYSIS OF THE MAIN TOWER BUILDING OF BEIJING YINTAI CENTER

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ABSTRACT :

In this study, the seismic behavior of the main tower building of Beijing Yintai Center is presented with regard to the dynamic characteristics analysis and seismic response analysis. Firstly, by means of three-dimensional finite element analysis software, the dynamic properties and seismic responses under frequent earthquake action of the structure are obtained, respectively. It can be seen that the structure has a rational arrangement for structural elements and has a good aseismic behavior. Then, the seismic behavior of the structure is studied through the dynamic elastio-plastic analysis method and static elasto-plastic ananlysis method under rare earthauake. Analysis results of both analysis methods show that the behavior of the structure accords with the earthquake performance objectives and the structure would not collapse under the rare earthquake action.

KEYWORDS: Dynamic properties, Seismic response, Dynamic elasto-plastic, Static elasto-plastic

1. INTRODUCTION

Fig. 1. Drawing of Beijing Yintai Center

Beijing Yintai Center project which is located in the core of Beijing's Central Business District (CBD) is composed of various large buildings for the purpose of office, service and apartments. The total building area of the project is 350,000 m^2 which including 86,000 m^2 underground and 264,000 m^2 above ground. The project is 247 m from east to west and 146 m from north to south, whose total land area is 30,000 m^2. Beijing Yintai Center consists of three towers as shown in Fig.1. The main tower is all-steel structure with the height of 249.5 m and 60 floors, which can be used as shopping malls, ordinary apartment, luxury apartments, Baiyue level hotels and super entertainment place[1].

The structural system of the main tower is tube-in-tube frame structure. The plane dimension of outer frame tube is 39.5m×39.5m, whose height-width ratio is 6.32, standard storey’s height is 3.3 m and column spacing is 5m. The four corner columns of the inner frame tube use steel box column while other columns use flat cross section or I-section steel column. The plane dimension of the inner frame tube is 15.6m×15.6m and column spacing is less than 4.725m.The storey height below the fifth floor is 4.5m, which is larger than the height of standard storey. Hence, In order to improve the lateral stiffness of the lower levels of the structure, column bracings are all located around the inner frame tube below the fifth floor. The main tower has four equipment levels, which are located on the floor 17, 33, 46 and 55, respectively. Cantilever truss structures which connect the inner and outer frame tubes are located on the floor 17, 33 and 46, respectively, to form strengthened stories in order to improve the floor stiffness[1]. Planar graph of main tower is shown in Fig.2.

The design parameters of the main tower are as follows. Design working life is 50 years, safety classification of structural system is grade one, seismic fortification intensity is eight degree, classification of design earthquake is class one, aseismic grade is three, site-class is Ⅱ and characteristic period of the ground T_g is 0.38s. Finite element model of the main tower is Fig.3.

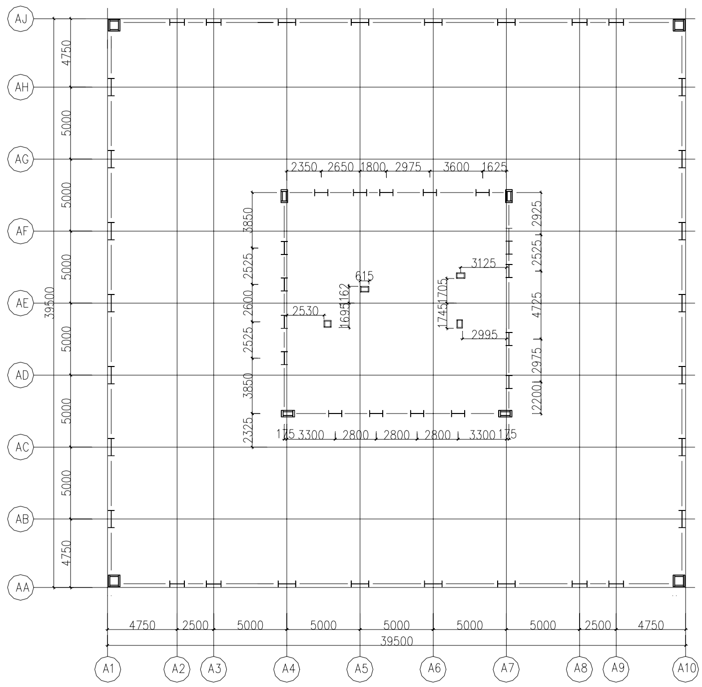

Fig. 2. Planar graph of main tower

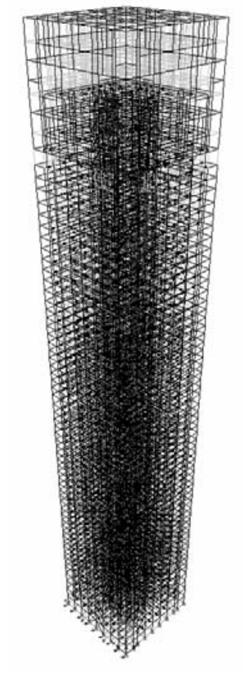

Fig. 3. Finite element model of main tower

2. ANALYSIS OF THE DYNAMIC CHARACTERISTICS OF THE MAIN TOWER

The analysis results of the structural dynamic characteristics are the important ways to interpret the structural characteristics and also the basis of mode-superposition response spectrum method and some other analysis. Nonlinear finite element analysis software was employed to perform the analysis of the dynamic characteristics of the main tower. Analysis results is shown in Table 1[2-3].

Table 1 Analysis results of dynamic characteristics

Mode number	period（s）	Model mass		accumulated modal participation mass		Spindle angle
		X dimension	Y dimension	X dimension	Y dimension	
1	6.2961	0.4968	67.8075	0.4968	67.8075	93.94
2	6.2178	68.1373	0.4784	68.6341	68.2858	3.93
3	3.4267	0.0000	0.0192	68.4361	68.3051	88.76
4	2.1104	0.8499	14.5922	69.4840	82.8951	102.86
5	2.0902	15.3158	0.8087	84.7998	83.7038	13.00
6	1.2815	0.0014	0.0036	84.8012	83.7074	63.73

From table 1 it can be seem:

(1) The basic period of x dimension and y dimension is 6.2178s and 6.2961s, respectively. The main vibration mode of the structure is maily translatory motion. The displacement of the top floor and relative displacement between two adjacent floors are mainly control parameters in the structural design. P-△ effect is also indispensable during the design.

(2) The ratio between the period of torsion mode and translation mode is 0.54. The period of translation mode is greatly larger than the torsion mode, which means the torsional rigidity of the structure is relatively strong.

(3) With regard to the mode participation coefficient and mass, it can be seen that low order modes of the

structure are almost mutually uncorrelated, which means the lateral stiffnesses of the structure in both x dimension and y dimension are symmetric and quality center almost coincide with the stiffness center. It is good for structural seismic performance.
(4) High order modes is indispensable for the super high rise structure with more flexibility. According to calculation results, the accumulated modal participation mass in x-y dimension reached 97% with the 30 modes. Therefore, the first 30 modes are considered in the seismic analysis.

3. ELASTIC ANALYSIS OF THE STRUCTURE UNDER FREQUENT EARTHQUAKE ACTION

3.1 Response Spectrum Analysis of Structure Under Frequent Earthquake Action

Based on the SRSS method of seismic spectrum analysis, design response spectrums are provided by code for seismic design of buildings, high-rise steel structure regulation and site evaluation report for Beijing Yintai Center, respectively. Fig. 4 shows different design response spectrums for main tower according to three design response spectrums, respectively, whose seismic fortification intensity is eight degree and the 50-year transcendental probability is 63%. In the figure, the curve of GB1-0.02-50 is based on the code for seismic design of buildings with the damping ratio 0.02 and design period 50 years. The curve of GB2-0.02-50 is based on the high-rise steel structure regulation with the damping ratio 0.02 and design period 50 years. The curve of CD-0.02-50 is based on the site evaluation report for Beijing Yintai Center with the damping ratio 0.02 and design period 50 years [3-5]. It can be seen that earthquake affecting coefficient is maximum under design response spectrum from high-rise steel structure regulation when the structure has a natural vibration period of 6s. Hence the design response spectrum of high-rise steel structure regulation is employed to perform seismic analysis.

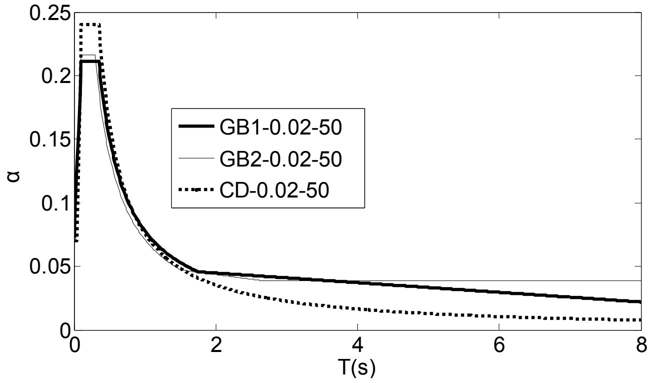

Fig. 4. Comparison diagram of design response spectrum

3.2 Time History Analysis of Structure Under Frequenct Earthquake Action

Time history analysis is performed based on the El Centro wave (NS) and two site assessment waves (Func1 and Func4). The peak values of three waves are adjusted to 70cm/s^2.

3.3 Analysis Results

The analysis results are compared using mode-superposition response spectrum method and time history analysis method. Table 2 shows the main results of the main tower structure under frequent earthquake action. Fig. 5 shows the interlayer shear in X dimension for the main tower. Fig. 6 shows the interlayer shear in Y dimension for the main tower. Fig. 7 shows the inter-story displacement angle in X dimension for the main tower. Fig. 8 shows the inter-story displacement angle in Y dimension for the main tower.
It can be seen from the figures and the table that:
1. Indexes in the X dimension and Y dimension are close to each other for the structure because of the symmetric in both two dimensions.
2. On the whole, the results which are calculated using response spectrum method are the maximum values

because the response spectrum is artificial enlarged during long period in the code.
3. Each index of the main tower meets the requirements of the code, which means the structural arrangement is reasonable and has a good seismic behavior.

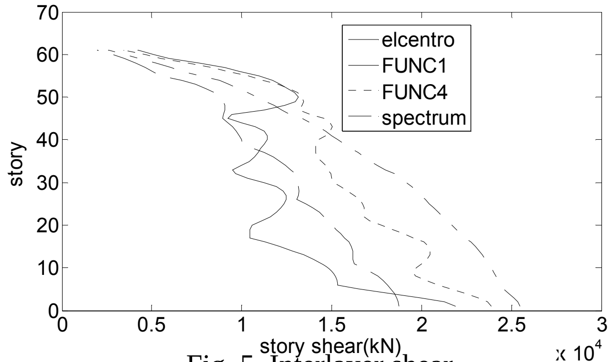

Fig. 5. Interlayer shear in X dimension

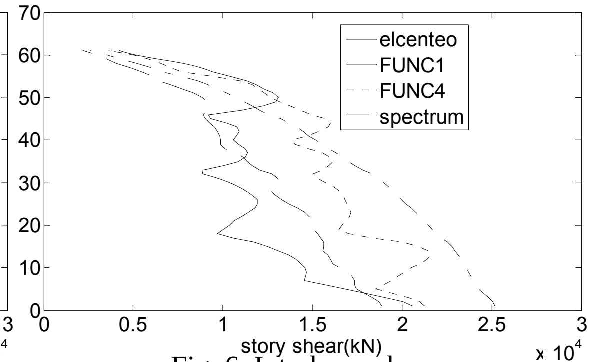

Fig. 6. Interlayer shear in Y dimension

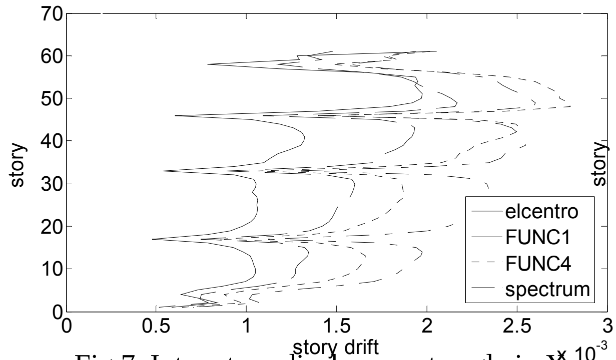

Fig.7. Inter-story displacement angle in X dimension

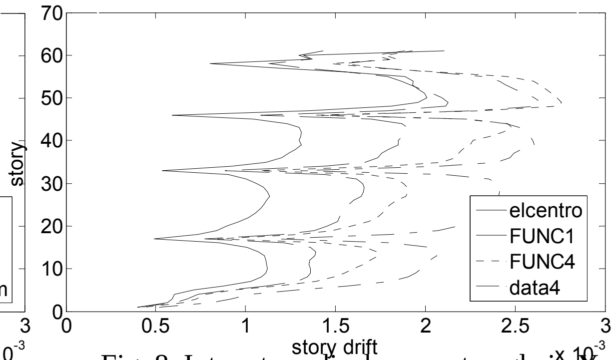

Fig. 8. Inter-story displacement angle in Y dimension

Table 2 Main results of the main tower structure

	base shear（kN）		displacement of top floor(m)		maximum story drift angle	
	X dimension	Y dimension	X dimension	Y dimension	X dimension	Y dimension
response spectrum	25473	25164	0.4374	0.4410	1/384	1/378
Elcentro	18738	18854	0.1504	0.1502	1/506	1/501
Func1	21884	20580	0.2959	0.2899	1/461	1/470
Func4	23863	21285	0.3484	0.3453	1/358	1/362

4. ELASTO-PLASTIC ANALYSIS OF THE STRUCTURE UNDER STRONG SEISMIC ACTION

4.1 Establishment of Elastic-plastic Model

In order to perform elasto-plastic analysis of the structure, finite element model is established using plastic hinge analysis method which accurately reflects the elasto-plastic bearing performance of the structure.
Elasto-plastic development of bar with rod structure is concentrated in the end,mid-span or short zone of the bar. In plastic hinge analysis method, according to the different bearing component simulated, plastic hinge can be divided into four types. P hinge simulates plastic performance in axial tension and compress, V hinge simulates bearing performance under shear, M hinge simulates bearing performance under bending moment and PMM hinge simulates the coupling effect of bending moment and axial force bearing performance at the same time. In the analytical model, plastic hinge unit is set on such position to reflect plastic bearing performance of the bar

while full-length elastic beam unit is set to reflect the elastic bearing performance of the structure.
Different plastic hinges using different type of yield surface have the same five-polyline skeleton curve[6-7]. There are five control point A,B,C,D,E in the curve. A is the point of origin. B is the yeild point of the hinge. C is the ultimate strength point of the hinge. D is residual strength of the hinge. E represents the point where the bar is completely destroyed and out of action. There is no deformation between the point A and B, that is, the hinge is assumed to be rigid before yielding and all the elastic deformation is happened within the frame element. Point IO, LS and CP represent the avility level of the hinge. IO indicates there is slight damage in bar which can be used without repair. LS indicates there is relatively serious damage in bar but still not endanger life safety which is suitable to continue to use after repair but not always economic. CP indicates the bar is close to collapse stage under serious damage or strength degration which can not be used any more even after repairment but the bar still can bear gravity and avoid collapsing.
During the analysis, the initial vertical load of the structure is the standard values of dead loads plus half standard values of live loads. At the same time, press bending coupled yielding PMM hinge is located at each column end, pure bending yielding M3 hinge is located at each beam end and tension compression yielding P hinge is located at brace.

4.2 Elasto-Plastic Analysis

The main tower's elasto-plastic response under strong seismic action is analyzed using elastoplastic time history analysis method and static pushover analysis method, respectively. The three waves mentioned above are used to perform analysis whose maximum value are adjusted to 70cm/s^2.
When performing static pushover analysis, lateral load is acted using inverted triangular distribution model, uniform load model and modal combination distribution model. At the same time, we can determine the target displacement of the structure under strong seismic action according to target displacement method and capacity spectrum method. We can also determine the displacement, inter-story displacement angle and elasto-plastic bearing states of the structure according to the target displacement under earthquake action.
Table 3 shows the maximum inter-story displacement angle of the structure under various operation conditions influenced by strong earthquake action. Fig. 9 shows the distribution of plastic hinges for the structure influenced by Fun4 wave. Fig. 10 shows the inter-story displacement of the structure influenced by Fun4 wave.
From the results we can see:
1. In the elasto-plastic analysis, plastic hinges did not appear on the column of the structure, which means the design of the structure is reasonable. The braces of the structure and the coupling beams around the core tube are relative weak components which are the main dissipative members. Most plastic hinge on such components developed fully or even decomposed.
2. In the elasto-plastic time-history analysis and nonlinear static analysis, weak stories are floor 49 and 50. The developments of the plastic hinges are almost the same case, that is, first appear on the brace and the coupling beam and then appear on the coupling beam of the inner core tube of each floor. There is no plastic hinge on the column.

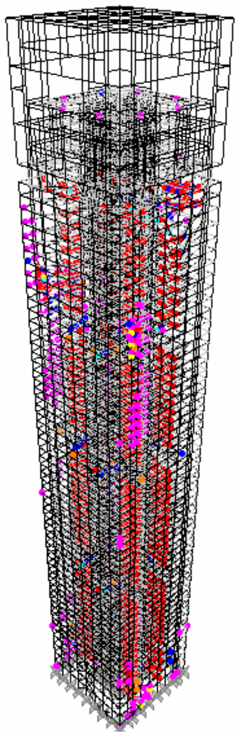

Fig. 9. Distribution of plastic hinge for the structure

Table 3 Maximum inter-story displacement angle of the structure

	Earthquake wave			Lateral loading mode		
	Elcentro	Func1	Func4	Inverted triangular	uniform load	modal combination distribution model
Maximum inter-story angle	1/154	1/137	1/112	1/189	1/258	1/135

3. The results of nonlinear static analysis have larger differences between each other. Compared with the results of elasto-plastic time-history analysis, the two results are much smaller with regard to the inverted triangular distribution model and uniform load model, because of the inverted triangular distribution model and uniform load model can not take high order mode into consideration. Considering the main tower stucture is influenced significantly by high order mode according to the modal analysis results, the results obtained from modal combination distribution model is more close to the results of elasto-plastic time-history analysis.

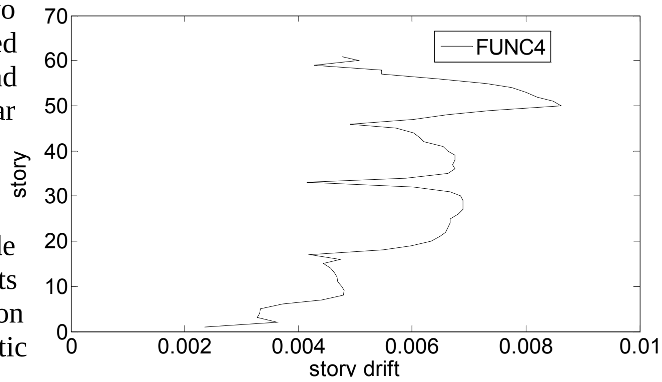

Fig. 10. Inter-story displacement in X dimension

4. The analysis results mentioned above show the maximum inter-story displacement angle under strong earthquake action can meet standard requirements.

5. CONCLUSION

1. According to the results of dynamic characteristic analysis, the structural arrangement is fundamental symmetric and the torsional vibration mode is basically uncoupled with the lateral vibration modes, which is good for structural seismic performance.
2. According to the analysis results under frequent earthquake action, various indexes have met regulation requirements which show the design of the structure is reasonable and has a good seismic behavior.
3. According to the analysis results under strong earthquake action, plastic hinge mostly appeared on the spray and the coupling beam of the inner tube and seldomly appeared on the column. The main tower can meet the aseismic criterions under strong earthquake action.
4. Analysis results with various lateral loading modes have larger difference. Hence, for the high-rise structure such as Yintai center, it's more accurate to make nonlinear static analysis using the modal combination distribution model which take high order mode into consideration.

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