11th World Congress on Computational Mechanics (WCCM XI)
5th European Conference on Computational Mechanics (ECCM V)
6th European Conference on Computational Fluid Dynamics (ECFD VI)
E. Oñate, J. Oliver and A. Huerta (Eds)

SEISMIC PERFORMANCE ANALYSIS OF THE HALL-COLUMN SYSTEM OF A TEMPLE STRUCTURE

ZHI ZHOU[*] AND JIANG QIAN[†]

[*] State Key Laboratory of Disaster Reduction in Civil Engineering
Tongji University
Siping road 1239, Shanghai , P.R.China
e-mail: 2012zhouzhi@tongji.edu.cn. http://risedr.tongji.edu.cn/en/Default.aspx

[†] State Key Laboratory of Disaster Reduction in Civil Engineering
Tongji University
Siping road 1239, Shanghai , P.R.China
e-mail: jqian@tongji.edu.cn. http://risedr.tongji.edu.cn/en/Default.aspx

Key Words: *Temple hall-column system, Centrifugally prefabricated concrete, Seismic design, Socket member.*

Abstract. A temple was designed to be constructed in a seismic region of high intensity. Its main hall was constructed by RC columns that would be centrifugally prefabricated in segments and socketed together in site. In this paper, a numerical computational model for the whole main hall building had been established by FEM (Finite Element Method). Linear elastic responses for the structure under seismic action of lower intensity had been performed, as well as elastic-plastic static pushover analysis under seismic action of higher intensity. Local load performance for a single column which was at a most unfavorable place had been carried out further in detail. The interface effects were modeled by non-linear springs. It is shown that, in comparing with a complete column, stress concentration may take place at the end section of the socket for a segmented column. This degraded the lateral load bearing capacity of the column finally. It is concluded that prefabricated socket members should be used with caution in seismic design.

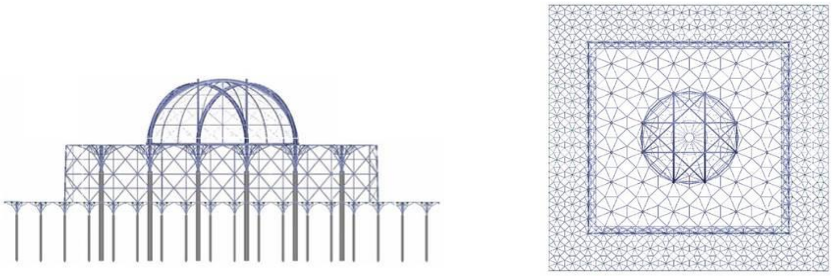
1 INTRODUCTION

A temple was designed to be constructed in a seismic region of high intensity (equivalent to category VIII of the Chinese Code for Seismic Design of Buildings[1]). Its main hall had a plan dimension of 145m×145m. 32 RC (Reinforced Concrete) columns with a maximum height of 45m were aligned in the central area of 105m×105m and 180 smaller RC columns with a height of 23m around. Umbrella-shaped capitals were designed to be steel and they were interconnected to form the roof. Columns were of hollow octagonal cross-section with a maximum section dimension of 1620mm and 810mm respectively and a wall thickness of 200mm. They would be centrifugally prefabricated in segments and socketed together in site.

Two column groups of different heights were connected with comprehensive steel trusses which could deliver the wind pressure and coordinate the deformation of the whole structure. At the top of the central column capitals laid a hemispherical steel dome with a height of 70m whose weight was mostly carried by the 8 central columns.

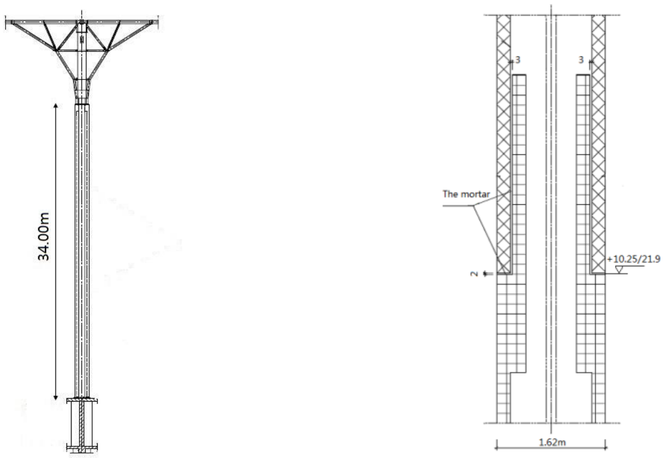
Hall columns have almost no lateral restriction along their height. The columns are connected by steel capitals and the column groups are connected by steel trusses. Most columns are of high aspect ratio. The lateral load resistance of the whole hall building therefore needs a thorough investigation. Also the functions of the steel trusses should be studied.

The structure is relatively simple and regularly distributed, which clarifies the loading condition of the columns. The weight of the dome makes the central 8 columns in the most unfavorable place. As the socketed columns are in a unique stress state, the lateral resistance performance of a single column at the most unfavorable place has to be evaluated particularly.



（a）Elevation of the Main hall （b）Plan of the Main hall

Figure 1: Structure of the main hall



（a）Schematic plot of typical long column （b）Schematic plot of the socket part

Figure 2: Structure of columns

2 FINITE ELEMENT ANALYSIS OF THE WHOLE STRUCTURE

2.1 Computational model

The simplified structural model is composed of the primary members of the main hall, including centrifugally prefabricated concrete columns, umbrella-shaped steel capitals, hemispherical steel dome, comprehensive steel trusses and concrete floors. The floors were modeled by BEAM188 elements and other members by SHELL63 elements in ANSYS[2]. The finite element model consists of 37592 beam elements and 18242 shell elements with a total mass of 15430t. Figure 3 shows the finite element model of the simplified structure. The material properties are listed in Table 1.

Table 1: Main material parameters of the structure

Members	Material type	elasticity modulus （N/m^2）	Density (kg/m^3)	Poisson ratio
Columns and floors	C50	0.343×10^{11}	2500	0.2
Steel	Q235	2.100×10^{11}	7850	0.3

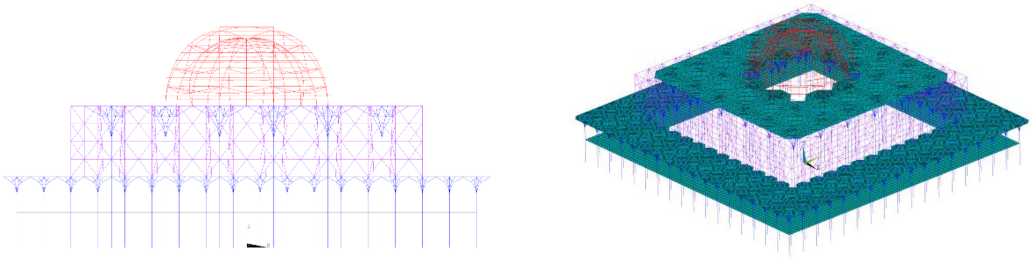


Figure 3: Finite element model of the simplified structure

2.2 Natural period and vibration modes

Two numerical computational models for the whole main hall, a Primary Structural Model (PS model) built by design modeling and a Degraded Structural Model (DS model) in which the stiffness of the steel trusses decreases by 100 times, has been established by ANSYS. Natural period and vibration modes are listed in Table 2.

Table 2: Natural period and vibration modes

PS	Mode	1	2	3	185
	Natural Period	1.629	1.629	1.320	0.313
	Modal characteristics	In Y	In X	Rotation around	In Z
DS	Mode	1	2	3	481
	Natural Period	1.701	1.701	1.335	0.313
	Modal characteristics	In Y	In X	Rotation around	In Z

Table 2 shows that the decrease of the steel trusses’ stiffness makes the fundamental translational periods a little bit longer by about 4% and it has almost no effect on the torsional vibration. It indicates that the stiffness of the steel trusses has only a limited contribution to the stiffness of the whole structure.

2.3 Response spectrum analysis

Seismic response spectrum analysis refers to the Chinese Code for Seismic Design of Buildings[2]. The mode superposition method is adopted in the computation. According to the design, the modal damping ratio is 5% and the ground motion period is 0.35s. The basic acceleration is 0.3g which is equal to the Chinese Code seismic intensity of 8 degree. The SRSS method is used to calculate structural responses. Only response in X direction is considered because of the symmetry of the structure.

Figure 4 presents the deformation of typical columns of the two different models. In PS Model the maximum displacements of the typical long column and short column are similar. But in DS Model the displacement difference caused by the decrease of the stiffness of the steel trusses increases significantly to about 14mm. The stiffness of the steel trusses plays an important part in coordinating the lateral deformation between long and short columns. Total shear force of columns in different areas are listed in Table 3. It indicates that the steel trusses don't deliver the lateral forces efficiently.

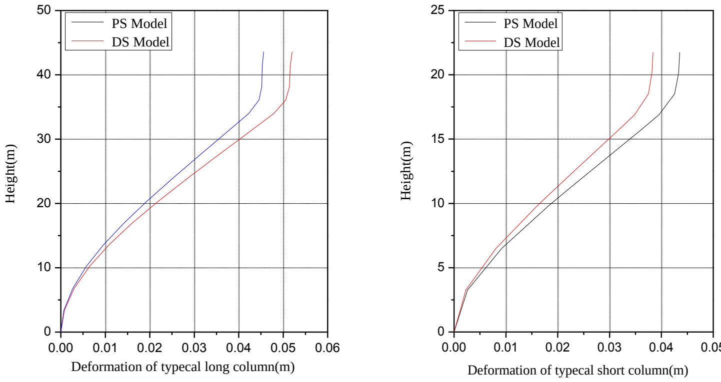


Figure 4: Deformation of typical concrete columns of different models

Table 3: Total shear force of columns in different areas (Force Unit: KN)

	Area	Regional shear force	Total shear force	Shear force ratio	Maximum shear force
PS Model	Long columns	1.716×10^{7}	8.077×10^{7}	21.3%	6.973×10^{4}
	Short columns	6.361×10^{7}		78.7%	4.677×10^{4}
	Area	Regional shear force	Total shear force	Shear force ratio	Maximum shear force
DS Model	Long columns	1.719×10^{7}	6.361×10^{7}	24.1%	5.913×10^{4}
	Short columns	5.407×10^{7}		75.9%	4.138×10^{4}

The displacement of typical nodes and maximum inter-story drift ratios are exhibited in Table 4. Figure 5 shows the typical nodes of the whole finite element structure. The number of the floors is defined by the concrete floor slabs as pictured in the followed figure. The limit of the inter-story drift ratio of reinforced concrete frame structure is 1/550 according to the Chinese Code. It is proved that seismic deformation response of the whole structure meets the regulatory requirements of the Chinese Code for Seismic Design of Buildings.

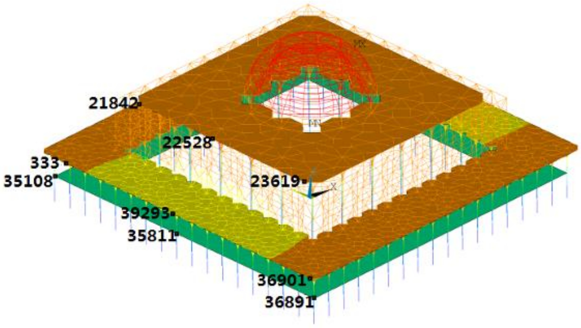


Figure 5: Plot of the typical nodes

Table 4: Displacement and inter-story drift ratios of the primal structure (Unit: mm)

Floor	Node	Node displacement	Average displacement	Inter-story drift	Floor height	Inter-story drift ratios
1	35108	20.683	20.403	20.403	10.85	1/554
	35811	19.702				
	36891	20.904				
2	333	39.194	38.125	17.722	10.90	1/615
	39293	37.715				
	36901	37.467				
3	21842	45.335	45.335	45.335	46.00	1/1014

2.4 Pushover analysis

Elastic-plastic static pushover analysis[3,6] under seismic action of higher intensity has been performed to evaluate the elastic-plastic seismic performance of the structure. In order to simplify the analysis, inverted triangle lateral force distribution was employed. The base shear and roof displacement curve is presented in Figure 6-7.

Shown at the crossover point of Figure 6, the target displacement determined by the energy spectrum method under rare earthquake of intensity 8 is 0.3755m. The maximum inter-story drift ratio is 1/59 which is within the code limit 1/50 according to the Chinese Code for Seismic Design of Buildings.

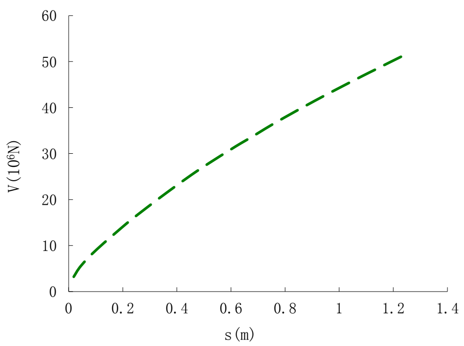


Figure 6: Pushover curve in X direction

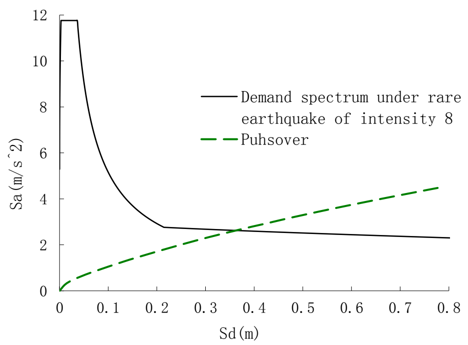
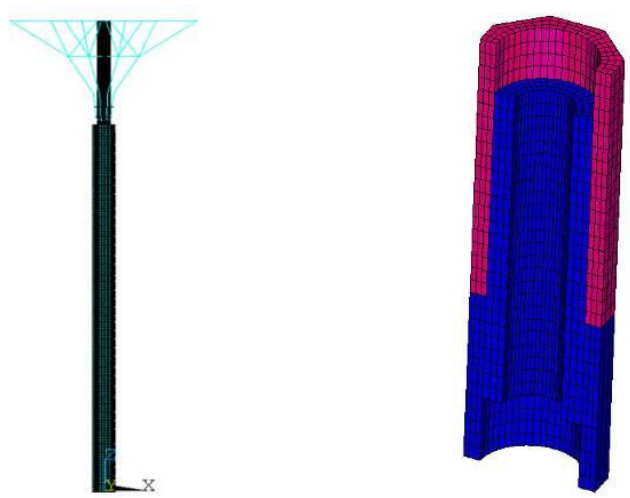


Figure 7: Demand spectrum under rare earthquake of intensity 8

3 FINITE ELEMENT ANALYSIS OF SINGLE COLUMN MODEL

3.1 Computational model

For further research into possible failure modes and seismic performance of a single column subjected to vertical load and horizontal seismic force, a fine finite element model of the concrete column which was at the most unfavorable place has been built. The SOLID65 element with CONCRETE material property was adopted to model the hollow octagonal concrete column. In the column model, the diffuse reinforcement was employed and the BEAM188 element was used to simulate steel members of the umbrella-shaped capital.



(b) Typical column model (c) Socket part model

Figure 8: Finite element model of socket column

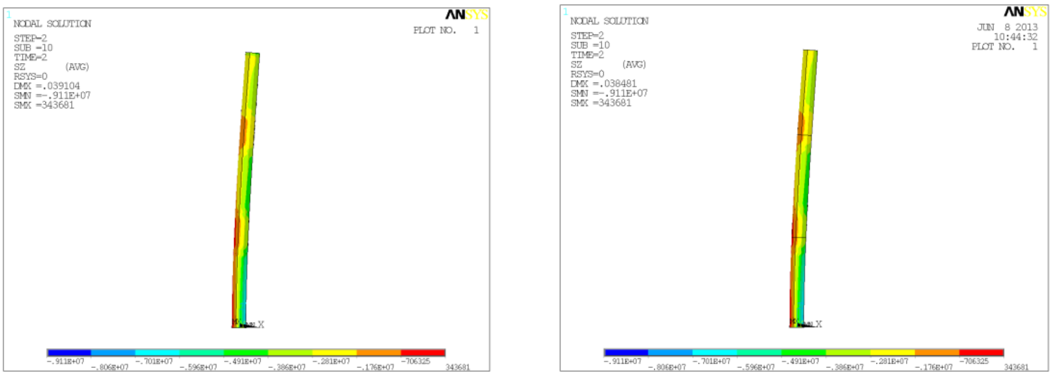
3.2 Results of finite element analysis

Two models of the socket part were built by different methods. Seamless connection which made the entire column as a whole was adopted to build a Perfect Model. A Socket Model was constructed by non-linear spring element COMBIN39 to simulate the interface effects of two separated segments of the column. A constitutive relationship of polyline curve determined by 4 points was defined for the spring element. The line starts from a point in the third quadrant. Its generalized force is equal to the pressure of the post-placing concrete under compressive strength and its generalized displacement is in correspondence with the ultimate compressive strain. The origin is the second point. The generalized force of the third point in the first quadrant is the tensile ultimate force of a 2mm-width concrete which is calculated in the situation that a 2mm-width crack occurs. And its generalized displacement is the elastic deformation of the concrete. The fourth point corresponds to the concrete tensile cracking load and the element deformation determined by the concrete ultimate tensile strain[7,8].

Resulted from the linear elastic responses for the structure under seismic action of lower intensity and elastic-plastic static pushover under seismic action of higher intensity, the base shear and vertical forces of the column at the most unfavorable place under seismic action of both lower and higher intensity were converted to inverted triangle lateral load and horizontal force of the single column. The whole finite analysis was divided into 15 load steps. The stress of the model under seismic load of two different intensities is presented as followed.

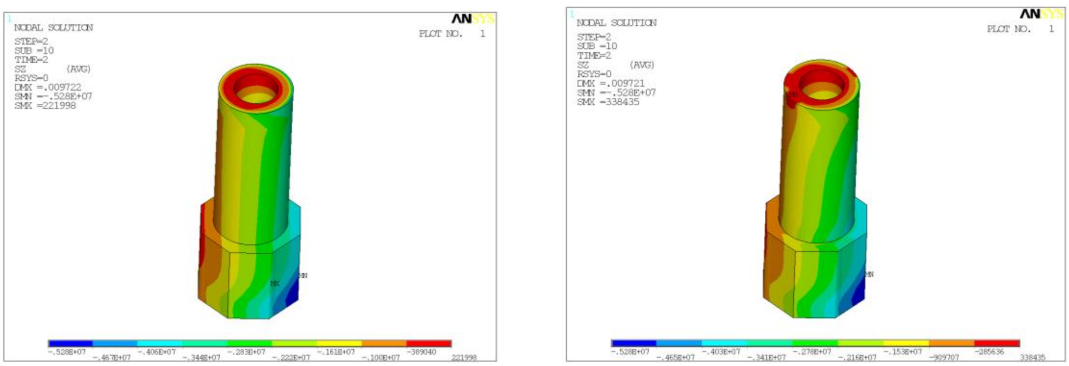
Figure 9-10 demonstrates the stress distribution of the socket part when the column is under seismic action of lower intensity. The two pictures in Figure 10 presents similar stress distribution. It indicates that the stress distributions of two models are mostly linear. But stress concentration of a small-scale part occurs in the Socket Model (Fig.10b).

Figure 12 shows the cracking state of the concrete socket part when the column is under seismic action of higher intensity while the cracking state of the whole column is presented in Figure 11. The cracking of the concrete happened in the 13th step of seismic load of higher intensity. With the increasing of the load, the bottom tensile elements of the column started to crack (Fig.11), and then the cracking gradually spread up to the socket part when the load continued growing (Fig.12b). Figure 12(b) shows that the socket cracked earlier when the base cracking was still below the socket part in the Socket Model. It indicates that stress concentration may take place at the end section of the socket for a segmented column. Then the advanced cracking of the socket part may influence the lateral resistance performance of the columns and decrease the structure's load bearing capacity. Therefore, particular methods to reinforce the socket part of the segmented columns should be taken to avoid advanced cracking of the socket.



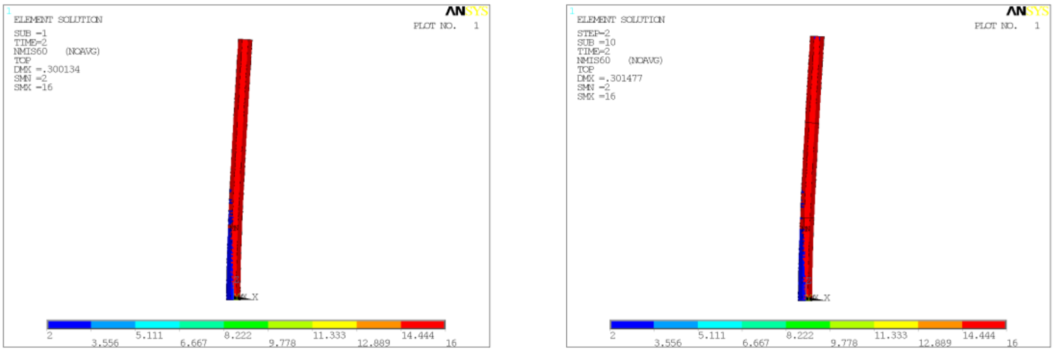
(a) Perfect Column Model (b) Socket Column Model

Figure 9: Vertical normal stress distribution of the concrete column under seismic load of lower intensity



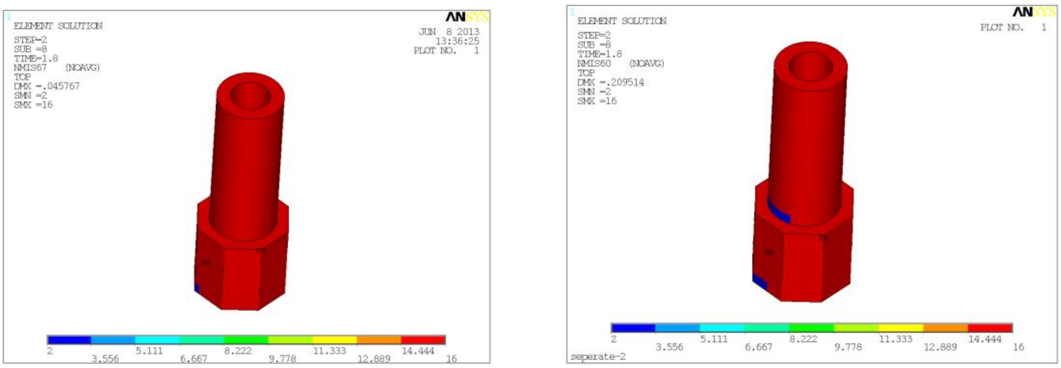
(b) Perfect Column Model (b) Socket Column Model

Figure 10: Vertical normal stress distribution of the socket part under seismic load of lower intensity



(a) Perfect Column Model (b) Socket Column Model

Figure 11: Cracking of the concrete column under seismic load of higher intensity



(b) Perfect Column Model (b) Socket Column Model

Figure 12: Cracking of the concrete socket part under seismic load of higher intensity

4 CONCLUSIONS

- The use of the prefabricated hollow columns as main bearing members improves the utilization rate of the material and the efficiency of the construction. Numerical results confirm that seismic performance of the hall-column system fulfilled the requirements of the Chinese Code for Seismic Design of Buildings.
- The steel trusses function appropriately as the lateral resisting system and coordinate the lateral deformation between long and short columns. It seems that the steel trusses don’t play a big role in delivering the base shear in the two parts.
- From the comparison of the local load performance between two different column models, stress concentration may take place at the end section of the socket for a segmented column and it might cause early cracking of the socket part. This could degrade the lateral load bearing capacity of the column eventually. It is concluded that prefabricated socket members should be used with caution in seismic design.

AKCNOWLEDGEMENT

The authors acknowledge with thanks the support from the National Natural Science Foundation of China (grant No. 91315301-4).

REFERENCES

[1] Ministry of Construction of the People’s Republic of China. *GB 50011-2010 Code for Seismic Design of Buildings*. Beijing: China Architecture and Building Press, 2010.
[2] ANSYS Inc., *ANSYS Structural Analysis Guide Release10.0*. SAP, IP Inc., 2007.
[3] FEMA237. *NEHRP Commentary on The Guidelines for The Rehabilitation of Building R*. Federal Emergency Management Agency, Washington, D.C. September,1996.
[4] ATC-40. *Seismic Evaluation and Retrofit of Concrete Buildings*. Applied Technology Council. Red Wood City, California, USA,1996.
[5] Ye Liaoyuan, Pan Wen. *Theory and Application of Structural Pushover Analysis*. Journal of Building Structures, 2000, 21(1): 37-43.
[6] Hou Shuang, Ou Jinping. *A Study of Load Pattern Selection of Pushover Analysis and Influence of Higher Modes*. Engineering and Engineering Vibration, 2004, 24(3):89-97.
[7] Zhu Bolong and Dong Zhenxiang, *Nonlinear Finite Element Analysis of Reinforced Concrete Structures*. Shanghai: Tongji University Press, 1985.
[8] Lv Xilin, Jin Guofang and Wu Xiaohan. *Theory and Application of Nonlinear Finite Element Analysis of Reinforced Concrete Structures*. Shanghai: Tongji University Press, 1997.