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Authors: Hao Qin, Tongji University
Xianzhong Zhao, Tongji University
Yiyi Chen, Tongji University
Bin Wang, Tongji University

Subjects: Building Case Study
Sustainability/Green/Energy

Keywords: Core
Mega Frame
Outriggers
Seismic

Publication Date: 2011

Original Publication: CTBUH 2011 Seoul Conference

Paper Type:
1. Book chapter/Part chapter
2. Journal paper
3. **Conference proceeding**
4. Unpublished conference paper
5. Magazine article
6. Unpublished

MS06-05

Elasto-plastic time-history analysis of the Shanghai Tower under rare earthquakes

Hao QIN[1], Xianzhong ZHAO[2], Yiyi CHEN[3], Bin WANG[4]

College of civil engineering, Tongji university, Shanghai, China, qhhao@163.com[1]
College of civil engineering, Tongji university, Shanghai, China, x.zhao@tongji.edu.cn[2]
College of civil engineering, Tongji university, Shanghai, China, yiyichen@mail.tongji.edu.cn[3]
College of civil engineering, Tongji university, Shanghai, China, wangbin_916@163.com[4]

Hao Qin

Biography

Hao Qin is a PhD candidate in the department of civil engineering, Tongji University of China. He has been working on the seismic perfor mance of high-rise composite structures for 3 years and was involved i n the time-history analysis of Shanghai Tower.

Abstract

The Shanghai Tower, designed to be 124stories of 636m height and currently under construction, will be the tallest building in China. The structure of the building involves mega frames, core wall and outrigger trusses. The mega frame connected with core wall by outrigger trusses, i s composed of eight super steel reinforced concrete (SRC) columns and belt truss. The Shanghai Tower locates at the Pudong area of Shanghai with a seismic intensity of 7(PGA=220gal under rare earthquake). Both the super height and the special structural system are big challenges for seismic design of the building. Consequently, its dynamic performance under earthquake requires intensive research. In this study, the mega frames, core wall and outrigger trusses of the Shanghai Tower are modeled from the material level using general purpose software ABAQUS, then elasto-plastic time-history analysis of the structure is carried out using scaled accelerograms representing the exceeding probability of earthquake events of 2% in 50-year. The computational results indicate that the super high-rise building possesses substantial reserve strength, and is predicted to satisfy the design requirements under rare earthquake. The method of structural modeling, time-history analysis and analytical results may be of considerable interest and useful for professionals and researchers involved in the seismic design of super-tall buildings.

Keywords: time-history analysis, high-rise building, earthquake

1. Introduction

The high-rise buildings stand for the economic and social civilization of the city. With the development of structural material and technology, the configurations of structures are more and more complex, the height becoming taller, yet the requirement for seismic design is stricter. Thus both traditional and innovative analytical methods are employed in the static and dynamic str uctural design. Particularly, time-history analysis using the finite element method (FEM) is the trend for seismic design with the help of rapid development of computer simulation. In the case s of some members going into plastic state under rare-earthquake, it is necessary to investigate the dynamic characteristics and seismic responses of the high-rise buildings. Elasto-plastic time-history analysis for Shanghai Tower (Fig.1) by ABAQUS is presented in this paper.

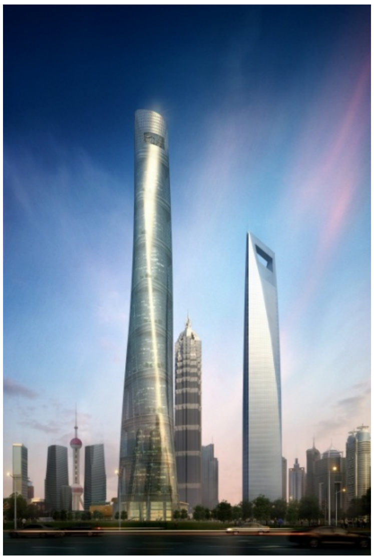


Fig.1. The Shanghai Tower

2. The structural system of the Shanghai Tower

The Shanghai Tower, 632m high and 124-storey, is a skyscraper with the integrated functions of office work, hotel, commerce and sightseeing. The lateral resistance system is a combination of "Mega-frame—Core-wall—Outrigger-trusses"(see in Fig.2).

Mega-frame is composed of 8 super columns and 4 diagonal columns connect e d by 8 belt trusses placed at strengthened stories.

Core-wall is the reinforced concrete structure, whose shape is cut down from square to cross (see in Table.1). In order to reduce the axial compression ratio and improve the shear bearing capacity and ductility, steel plates are added in the walls of basement and zone 1~2 to form steel shear walls with steel ratio of 1.5%～4.0%.

Outrigger-trusses run through the core wall and connect with the super columns, placed at strengthened stories of zone 2, zone4 and zone5~8.

In addition, radical trusses are located at the upper strengthened stories, transferring the vertical loads to belt truss, super columns and tube.

The 3D FE model of Shanghai Tower established for numerical analysis is shown in Fig.3.

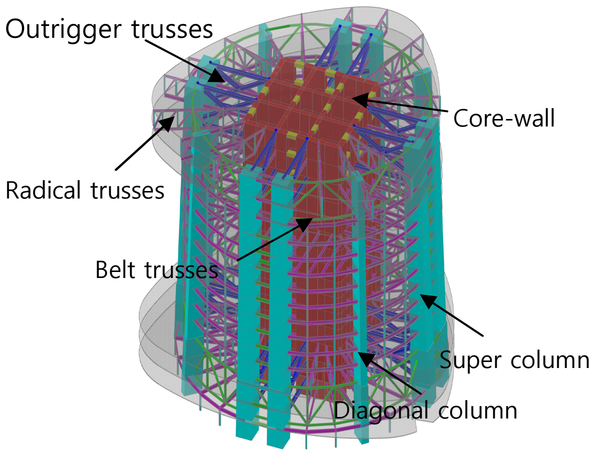


Fig.2. Structural system

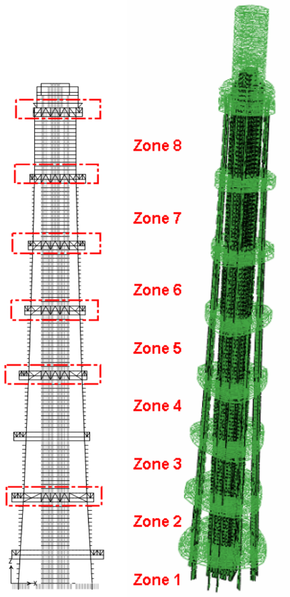
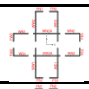
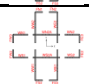
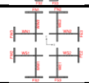
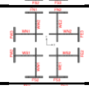
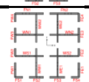
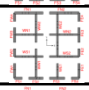
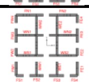
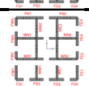


Fig.3. 3D FE model of Shanghai Tower

Table.1 Parameters of super columns and core-wall in schematic design stage

Zone Number	Section of super columns	Section of diagonal columns	Strength of concrete in columns	Core wall section	Thickness of flange wall	Thickness of web wall	Strength of concrete in wall
Zone 8	1.9×2.4		C50		0.6	0.5	C60
Zone 7	2.3×3.3		C50		0.6	0.5	C60
Zone 6	2.5×4.0		C60		0.6	0.6	C60
Zone 5	2.6×4.4	1.2×4.5	C60		0.7	0.65	C60
Zone 4	2.8×4.6	1.5×4.8	C60		0.8	0.7	C60
Zone 3	3.0×4.8	1.8×4.8	C70		1	0.8	C60
Zone 2	3.4×5.0	2.2×5.0	C70		1.2	0.9	C60
Zone 1	3.7×5.3	2.4×5.5	C70		1.2	0.9	C60

3. Structural modeling of the Shanghai Tower

3.1 Finite element model

General purpose software, ABAQUS, was used to model and analyze the Shanghai Tower. Core wall and coupling beams are simulated by means of S4R shell elements in according with th e real dimensions, and composite shell elements are used to simulate the encased steel-plate shear walls in zone1 and zone2 (see in Fig.4). Reinforcements in walls are simulated by the optio n of Rebar in ABAQUS. The embedded steel columns are simulated by B31 beam elements, which are coupled with the shell elements of walls.

The super columns are simulated by the coupling of B31 beam elements and S4R shell elements, which represents the interaction of the steel H-section and the concrete part of the column respectively

as shown in Fig.5. The members of outrigger trusses, belt trusses, radical trusses and other s teel beams are also simulated by B31 beam elements according to the real dimensions and sectio ns.

Based on the assumption of rigid diaphragm of floor (see in Fig.6), the X-DOF, Y-DOF and Rz-DOF of all the nodes in the same layer are coupled in typical floors of every zone. The effective mass of the floor are distributed to the nodes as mass elements in the same layer according to the load acting area.

All the connections are assumed to be rigid connections including the connections of the outrigger truss members and the core wall, the outrigger truss members and the super columns, the super columns and the belt truss members, the radical trusses and the core wall, as well as the connections between the adjacent members in the trusses. Rigid arms are set between the outrigger trusses and super columns, and between the belt trusses and super columns, mainly considering the extremely large thickness of super columns (see in Fig.7).

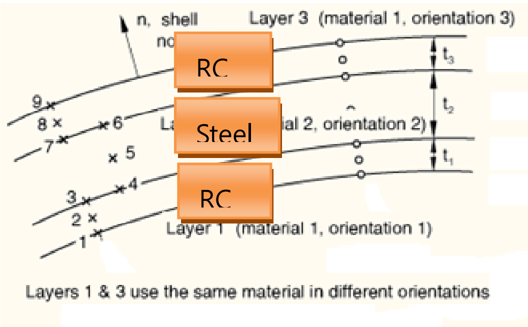


Fig.4 Composite shell

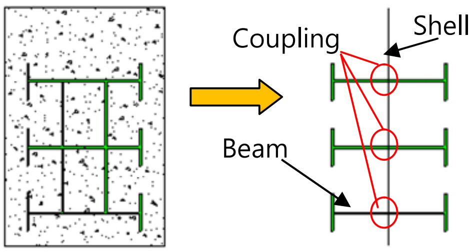
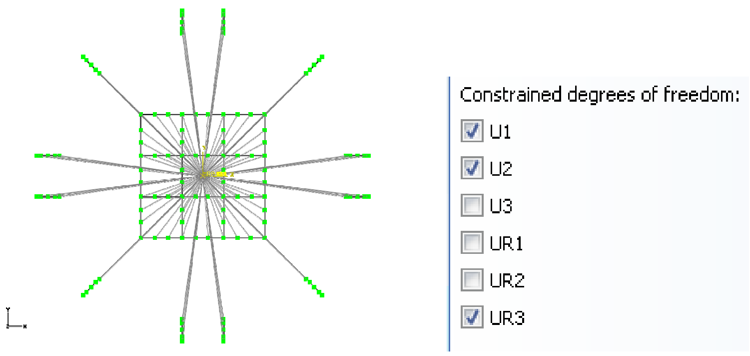


Fig.5 FE simulation of super column



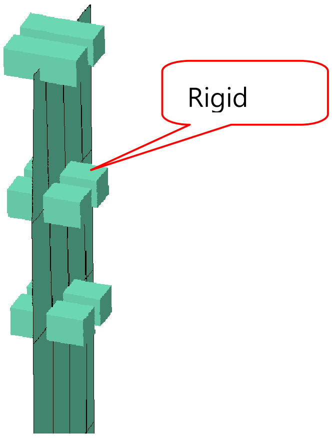


Fig.6 assumption of rigid diaphragm of floor Fig.7 Rigid arm

3.2 Property of material

The stress-strain relation adopted for the steel is BKIN model, which assumes that the plastic modulus is 1/100 elastic modulus. Bauschinger effect is simulated in this model and no stiffness degradation is considered in the loop (see in fig.8).

The stress-strain relation of concrete for core-wall is shown in Fig.9, which is named Concrete damaged plasticity model[1]. The model is a continuum, plasticity-based, damage model for concrete. It assumes that the main two failure mechanisms are tensile cracking and compressive crushing of the concrete material. The degradation of the elastic stiffness and the stiffness recovery are also considered. It is designed for applications in which concrete is subjected to monotonic, cyclic, or dynamic loading under low confining pressures. The damage variables of tensile and compressive are shown in Fig.11 and Fig.12. The parameter of this model is shown in table 2.

The stress-strain relations of concrete for super column is Stephen model (see in fig.1 0), which is simplified but accurate enough to simulate the constraint concrete in super composit e columns[2].

Concrete elastic modular E_c and tensile strength f_t are presented

$$E_c = \frac{10^5}{2.2 + \dfrac{34.7}{f_{\mathrm{cu,k}}}} \quad (\mathrm{N/mm^2}) \qquad (1)$$

$$f_t = 0.1 f_c \qquad (2)$$

where, $f_{cu,k}$ is the cube compressive strength, f_c is the characteristic compressive strength.

3.3 Damping of material

Rayleigh damping is adopted in time-history analysis, the damping coefficients α、β are defined as

$$\alpha = \frac{2\omega_i \omega_j \zeta}{\omega_i + \omega_j}, \beta = \frac{2\zeta}{\omega_i + \omega_j}.$$

Damping ratio is 0.05 in rare earthquake condition.

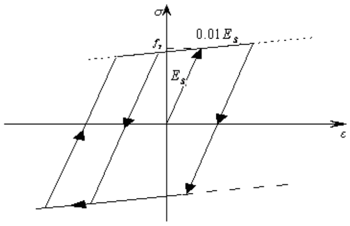


Fig. 8 . Stress-strain relation of steel

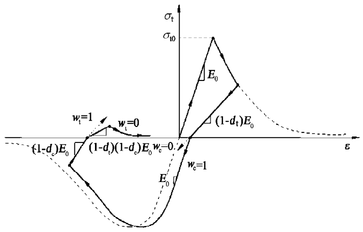


Fig. 9 . Concrete damaged plasticity model

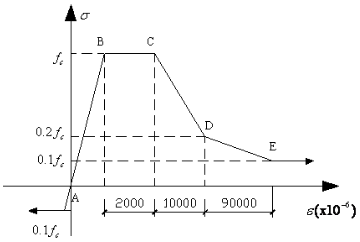


Fig.10. Stress-strain relations of concrete in super column

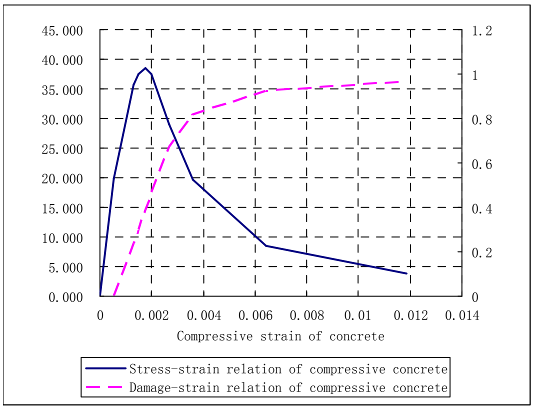


Fig.11. Compressive damage variable

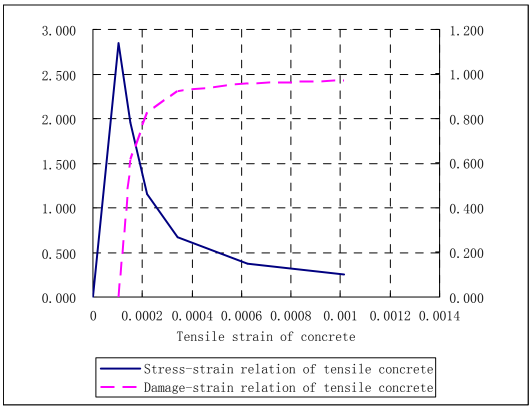


Fig.12. Compressive damage variable

Table.2. The parameter of Concrete damaged plasticity model

Poisson's Ratio	Dilation Angle	Eccentricity	f_{bo}/f_{co}	K	Viscosity Parameter
0.2	30	0.1	1.16	0.667	0

4. Dynamic characteristics of the high-rise building

The FE model contains 56936 elements with a total mass of 7.07×10^{8} kg. Table.3 and Fig.13 shows the first six mode characters of the FE mode l . Mode 1 and M o d e 2 are the translational modes in X and Y directions, respectively. Mode 3 is the torsional model. T he fundamental period of the building are 9.13s in X direction, 9.04s in Y direction, 4.50s in torsio n.

Table.3. Frequency and period of the first six modes

Mode number	Frequency	Period	Direction
1	0.10950	9.13	X direction translation
2	0.11061	9.04	Y direction translation
3	0.22210	4.50	Z direction torsion
4	0.30604	3.27	X second-order direction translation
5	0.31400	3.18	Y second-order direction translation
6	0.44928	2.23	Z second-order direction torsion

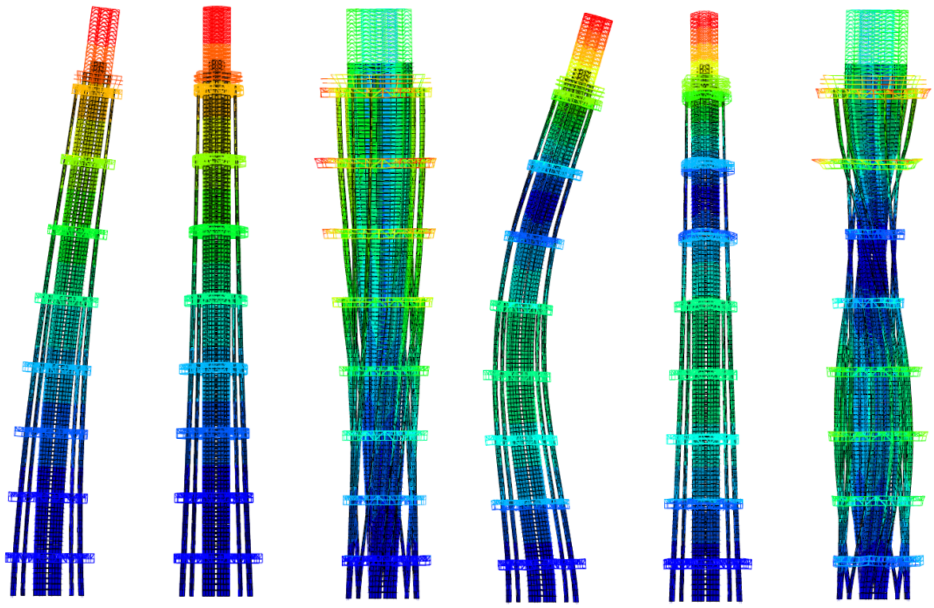


Fig.13. Modes shapes of the first six modes obtained from numerical analysis

5. Results of time-history analysis

Time-history analysis is a step-by-step direct integration in which the time domain is discretized into a large number of small incre ments, and for each time interval the equations of motion are solved to obtain the structural resp onses such as displacements, internal force and deformation of members in every time.

Seven recorded accelerograms were selected as inputs in the present time-history analysis: US257, US335, US725, US1214, MEX006 are natural seismic waves, S79010 and L7 111 are artificial seismic waves. Each accelerogram is composed of two horizontal and one vertical components of t h e ground acceleration excitation . T he peak ground accelerations is scaled to 200 gal, which is input at the ratio of 1 (primary horizon tal direction):0.85 (secondary horizontal direction): 0.65 (vertical direction).

5.1 Base shear and base moment

Displacement and velocity of the ground motion have a greater impact on the long-period structure than acceleration, which is not evaluated precisely by the response spectrum anal ysis. The ratios of base shear to self-weight subjected to the excitation of the seven seismic waves (list in Table.4) are more than 1.2%, which meet the requirement of the code [3].

5.2 Maximum lateral displacement and inter-story drift ratio

The maximum lateral displacements and inter-story drift ratios of the structure under the seven recorded accelerograms are listed in Table 5.

Table.4 Base shear and base moment

Seismic waves	Primary direction is X			Primary direction is Y		
	Vx(kN)	Ratio of base shear to self-weight	My(kN*m)	Vy(kN)	Ratio of base shear to self-weight	Mx(kN*m)
US257	289171	4.17%	31670600	304614	4.40%	31224400
US335	364863	5.27%	33965000	387593	5.59%	36597260
US725	305276	4.41%	40602300	324406	4.68%	43963287
US1214	337260	4.87%	46628600	338800	4.89%	46712200
Mex006	379698	5.48%	92475600	418790	6.04%	95210700
S79010	364384	5.26%	60295900	367691	5.31%	64853388
L7111	447312	6.46%	62094100	503120	7.26%	80476500
Average	414660.7	5.13%		377859.1	5.45%	

Table.5. Maximum lateral displacement and inter-story drift ratio

		US257	US335	US725	US1214	Mex006	S79010	L7111
Primary direction is X	Maximum lateral displacement (m)	1155	1069	1193	1026	2451	1886	1928
	Maximum inter-story drift ratio and the location	1/177	1/170	1/187	1/187	1/116	1/146	1/133
		93F	94F	94F	93F	92F	92F	92F
Primary direction is Y	Maximum lateral displacement (m)	1183	1088	1285	1664	2526	1953	2280
	Maximum inter-story drift ratio and the location	1/203	1/179	1/202	1/164	1/150	1/172	1/150
		109F	109F	92F	109F	93F	94F	106F

The lateral displacement subjected to MEX006 excitation is the largest among all the excitation. The placements are 2451mm (in X direction) and 2526mm (in Y direction), respectively, and thus the inter-story drifts, 1/116 (in X direction) and 1/150 (in Y direction). All the inter-story drift ratios are less than the corresponding criteria (1/100) stipulated in the Chinese code for seismic design[3]. The maximum inter-story drifts occur in the middle floors of zone 7 and zone 8. Fig.1 4 ~Fig.1 5 shows the curves of the inter-story drift ratios for the structure.

5.3 Damage of members

Belt trusses, outrigger trusses, most of radical trusses remain in elastic stage during the whole loading process. Core walls are damaged around the strengthened stories and the stories with large inter-story drift ratio, especially in the fourth and seventh zone.

Concrete of super columns remains elastic except that in the connection of columns and outrigger trusses strengthened zone. All the encased steel of columns remains elastic.

Most of coupling beams are seriously damaged, especially at the stories with larger inter-story drift ratio. Coupling beams are the main damaged members for energy dissipation.

Fig.16~Fig.21 show the elastoplastic time-history results of representative members in the fourth and seventh zone.

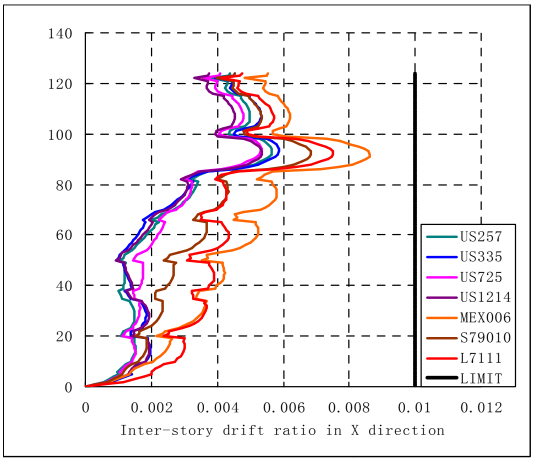


Fig.14. Curves of the inter-story drift ratio in X direction

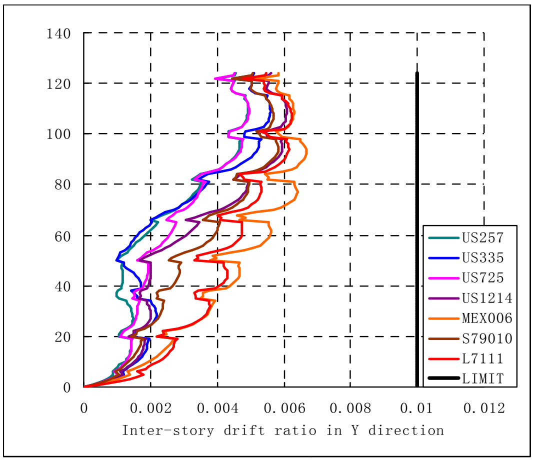


Fig.15. Curves of the inter-story drift ratio in X direction

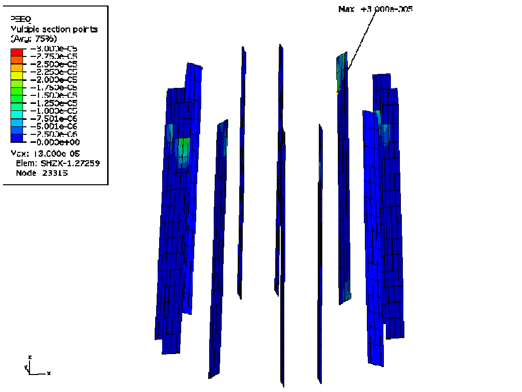


Fig.16 Plastic strain of columns in the fourth zone

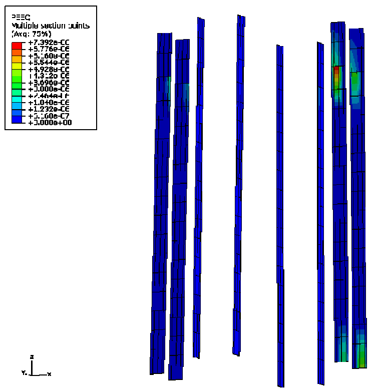


Fig.17 Plastic strain of columns in the seventh zone

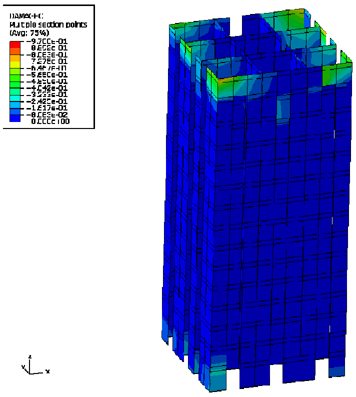


Fig.18 Damage of core-wall in the fourth zone

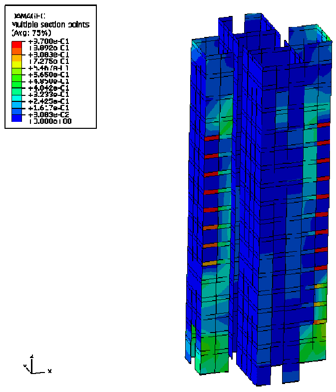


Fig.19 Damage of core-wall in the seventh zone

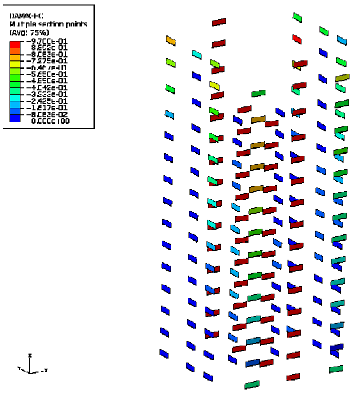


Fig.20 Damage of coupling beams in the fourth zone

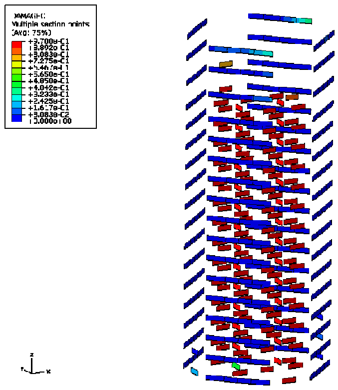


Fig.21 Damage of coupling beams in the seventh zone

6. Conclusion

A 3-D finite element model of Shanghai Tower was established and a detail study on the dynamic char acteristics and seismic responses under the rare earthquake was presented in this paper. The resul t indicated that the structural system of the Shanghai Tower, composed of Mega-frame—Core wall—Outrigger-trusses , provides sufficient stiffness to meet the earthquake resistant standard of shanghai, the inter-story drift ratios are less than the limit (1/100), the main resistance members are not crushed duri ng the analysis. The energy from the seismic excitation was mainly dissipated by the coupling beams between the walls. There are abrupt changes in stiffness near th e strengthen floors with outrigger-trusses, which should be adequately dealt with in earthquake-resistant design.

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