The application of ABAQUS in seismic analysis of connected structures

Jiachun Cui, Chengming Li, Wei Tian, Dongya An

Technical Center of Shanghai Xian Dai Architectural Design (Group) Co.,Ltd.

20F, 258 Shimen Er Road, Shanghai, China. 200041

Jiachun_cui@xd-ad.com.cn, chengming_li@xd-ad.com.cn,

Wei_tian@xd-ad.com.cn, dongya_an@xd-ad.com.cn

Abstract: The connected structure refers to the kind of building which is composed of two or more towers connected by the connecting body in a certain height, belonging to the irregular building structure system. According to "Technical Specification for Concrete Structures of Tall Building" (JGJ3-2002), the time-history analysis method should be adopted in the seismic analysis of the connected structure. The structure may have a larger plastic deformation under rare earthquake, so it is difficult to converge when the implicit solution method is used by the conventional finite element software. While the explicit integral technology provided by ABAQUS can solve the nonlinear dynamics problems better, it has a broader application in elastic-plastic dynamic analysis. Taking a specific project as an object, the application of ABAQUS in seismic analysis of connected structures is presented in detail in this paper.

Key words: ABAQUS, connected structures, elastic-plastic, seismic analysis.

1. Introduction

1.1 Connected structures

The connected structure refers to the kind of building which is composed of two or more towers connected by the connecting body, for example, a joint gallery, in a certain height, belonging to the irregular building structure system. Due to the existence of multiple towers and the connecting body, the response of the connected structure is far more complex than the monomer structure and the multi-tower structure without connecting bodies. It is shown that in the Kobe Earthquake in Japan and the Chi-Chi Earthquake of Taiwan, the connecting bodies themselves collapsed more and meanwhile, the parts linked to the main structure and connecting bodies were seriously damaged"(Ren Xu, 2006)". Therefore, in practical engineering applications, further study should be made on the stress characteristics of connected structures, especially on the dynamic response of connected structures under rare earthquake.

According to the literature [5], the following requirements should be met for the seismic analysis of high-rise buildings with complex structures:

1) At least two kinds of three-dimensional software whose mechanical models are different are adopted to calculate the overall internal forces and displacements; Due to the special shape of connected structures, the complexity of stress of connecting parts is common, as a result, it is suitable that the finite element model should be adopted to conduct an overall modeling and analysis, and that the elastic floor be used when calculated for the floor of connecting body.

2) Supplement Calculations should be made using the elastic time-history analysis method.

3) The nonlinear static or dynamic analysis should be adopted to check the elastic-plastic deformation of the weak story.

1.2 The applicability of ABAQUS

The structure may have a larger plastic deformation and injury under rare earthquake, so it is difficult to converge when the implicit solution method is used by the conventional finite element software. While the explicit integration technology provided by ABAQUS can solve the nonlinear dynamics problems better, it has a broader application in elastic-plastic dynamic analysis.

The B31and B32 elements provided by ABAQUS can simulate the beams and columns of the architectural structure and the S4R element can better simulate the shear walls and floors. The user subroutine function of ABAQUS allows users to develop the constitutive model of special materials according to demand in order that the software calls.

It is these advantages of ABAQUS that makes it one of the mainstream software for dynamic elastic-plastic analysis of building structures.

2. Engineering situation

2.1 Project introduction

The project is located in Bao'an District, Shenzhen, with a total area of 39497.45 square meters, which are composed by the West Building, North Building, podium and the connecting body. The West Building (T1) has a total of 22 stories, using reinforced concrete frame-tube structure system; North Building has a total of 18 stories, using reinforced concrete frame-shear wall structure system; the floor and roof are the cast-in-place reinforced concrete beam-slab structure.

Between the 21st and 22nd floors, the North Building and West Building are connected by two 50 meter-long steel trusses and the connection between the connecting body and the two main buildings is a weak link; the connection between the 15th to 18th floors of North Building is rigid and four 25.2 meter-long steel trusses are set on each floor.

The material of the frame columns, shear walls from the first to third story is C60 concrete, and that from the fourth to 23rd story is C50 concrete; both floor beams and floor slabs are C30; steel plate and shaped steel are Q345B and the electrode are selected as E43 and E50 types; the steel bar in the beams and columns is HRB400 (fy = 360N/mm2) and that in shear walls and floors is HRB335 (fy = 300N/mm2).

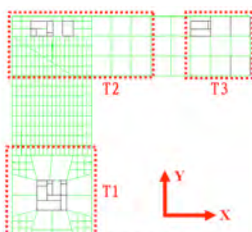
Figure 1 Architectural effect drawing

2.2 Structural characteristics

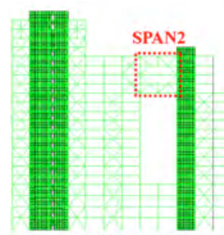
(1) Multi-tower structure. This structure is composed of three towers connected by two joint galleries which is a typical connected structure.

(2) The weak link. The tower T2 and the 50 meter-long joint gallery are merely connected by a concrete tube and the connection part is smaller.

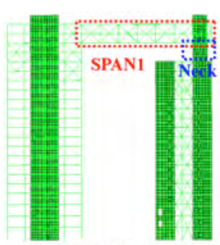
(3) The large span. The net span of the joint gallery between the towers T1 and T2 is 50m, and the height is only two-floor high. In addition, although the layout of the tower T1 is regular, the span of beams between the concrete core tube and external frame columns are all more than 10m.



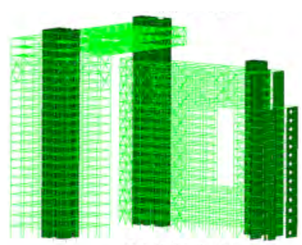
(a) Plane view of the structure



(b) Front elevation view



(c) Right elevation view



(d) three dimensional view

Figure 2 Structural arrangement drawings

2.3 Calculation model

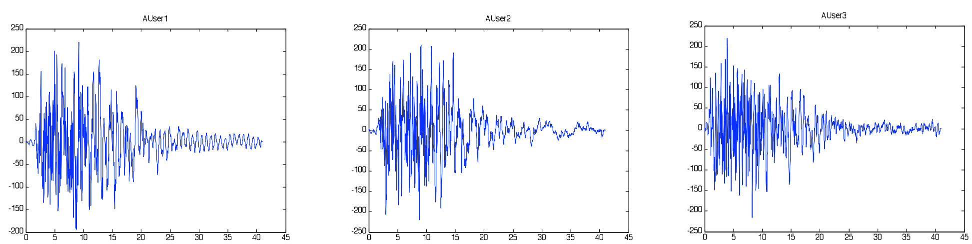
S4R element is selected for the shear walls, considering the reinforcement in size and distribution in the shear wall; beams and columns adopt B31 element to simulate, in which the steel bar use B31 to simulate and is set as Box section; the S4R element is adopted for the floor and the size and distribution of the reinforcement should be taken into consideration. In the dynamic analysis process, the dead load and live load on the structure use Mass element to simulate.

The floor where the connecting body lies is seen as the elastic floor, while for other floors rigid diaphragm assumption is adopted.

In the elastic-plastic dynamic time-history analysis, the explicit integral technology provided by ABAQUS is introduced to solve it.

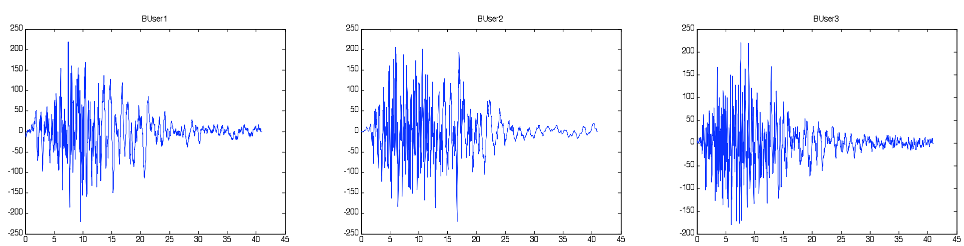
2.4 Seismic wave used in this paper

The seismic wave in calculation derives from the report of risk assessment, whose peak is 220cm/s2, and duration is 40s, which meets the requirements that the duration is 5 to 10 times greater than the natural vibration period (the first cycle of the structure is 1.86s); in the analysis the input model of three directions of seismic waves is always adopted, and the ratio of the seismic wave peak in level principal direction, secondary direction and vertical direction is 1:0.85:0.65.



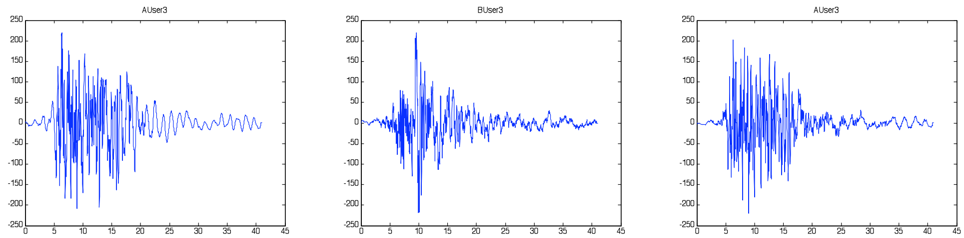
X direction **Y direction** **Z direction**

Figure 3 The first group of natural wave



X direction **Y direction** **Z direction**

Figure 4 The second group of natural wave



X direction **Y direction** **Z direction**

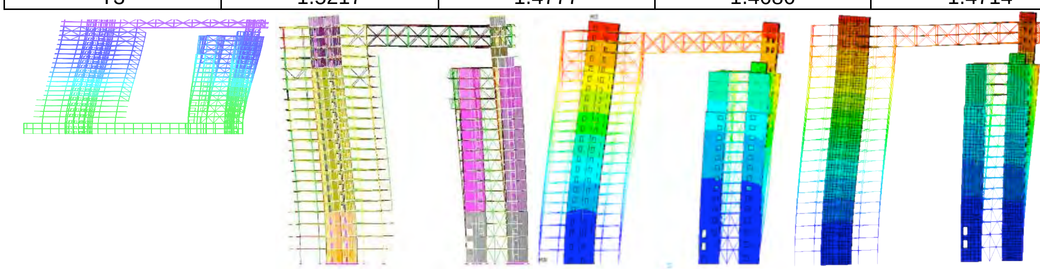
Figure 5 Artificial wave

3. Period and vibration mode

In the process of static calculation SATWE and ETABS9.2.0 are used for member design, and then ANSYS10.0 is adopted for adding plate and bar elements. At last, ABAQUS6.6.1 is used for dynamic analysis. At each step the calculation of the period and vibration mode is made so as to ensure the accuracy at each step in the model transformation progress. Different results by different software are shown in Table 1 and Figure 6.

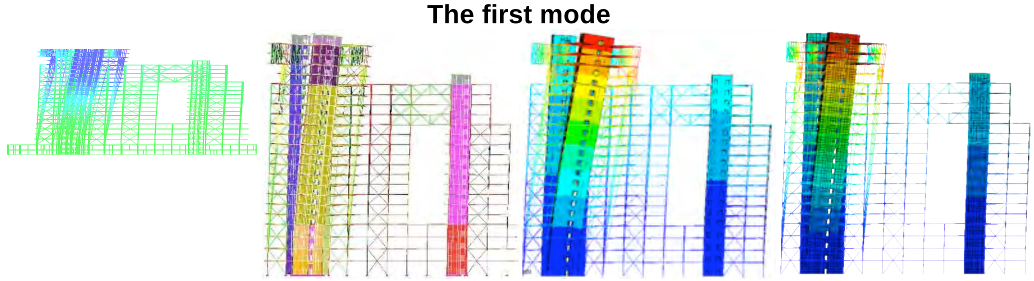
Table 1 Comparison of the first three orders of period（s）

Period （s）	SATWE	ETABS	ANSYS	ABAQUS
T1	1.8343	1.8632	1.8834	1.8612
T2	1.8262	1.8006	1.8116	1.8287
T3	1.5217	1.4777	1.4686	1.4714



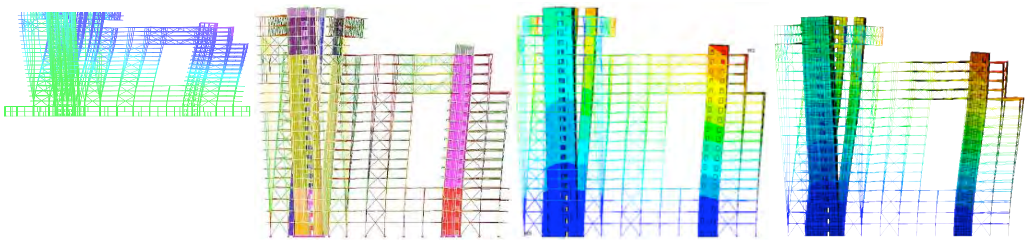
SATWE **ETABS** **ANSYS** **ABAQUS**

The first mode



SATWE ETABS ANSYS ABAQUS

The second mode



SATWE ETABS ANSYS ABAQUS

The third mode

Figure 6 Vibration modes by different software

We can see from Table 1 and Figure 6 that the first three orders of the natural vibration period and the vibration mode are basically the same, which indicates that the completeness, correctness and accuracy are guaranteed in the model transformation progress.

4. Calculation results under rare earthquake

Simulation environment as follows:

(1) Information of CPU: Intel(R) Xeon(R) CPU X5355@2.66GHz. Four CPU are used in the analysis.

(2) Information of Memory: 8.00GB.

(3) System of computer: Microsoft Windows Server 2003 Enterprise x64 Edition Service Pack2.

(4) CPU time for a seismic wave calculation: about 6 hours.

Since the project consists of three towers, the top displacement of the structure should be extracted respectively from the three towers, which adopts the average displacement of the four corners of shear wall core tube at the top of the tower.

Also the maximum inter-story drift ratio is extracted respectively from the three towers. Because there exist a number of elastic floors in the structure and simultaneously when the irregularity of the towers T2 and T3 is taken into account, the data of T1 is selected as the average maximum inter-story drift ratio corresponding to the nodes of the four corner column and that of T2 and T3 should be valued as the average of the maximum inter-story drift ratios of the four corner-displacements of the shear-wall core tube.

4.1 Base shear

Table 2 shows the maximum base shear of the structure under the action of the three groups of seismic waves. As we can see the maximum base shear in X and Y directions are respectively -145158.8kN and -137092.4kN and the corresponding shear-weight ratios are 11.5% and 10.8%. Therein, the base shear under the action of the first group of natural wave is the maximum and in this case the base shear-time curve is shown in Figure 7.

Table 2 The maximum base shear corresponding to each seismic waves (kN)

Seismic wave groups	Principal direction	Shear force	Shear-weight ratio（%）
The first group of natural wave	**X**	-145158.8	11.5
	Y	-137092.4	10.8
The second group of natural wave	**X**	-101192.6	8.0
	Y	-126964.7	10.0
Artificial wave	**X**	-123005.5	9.7
	Y	-118027	9.3

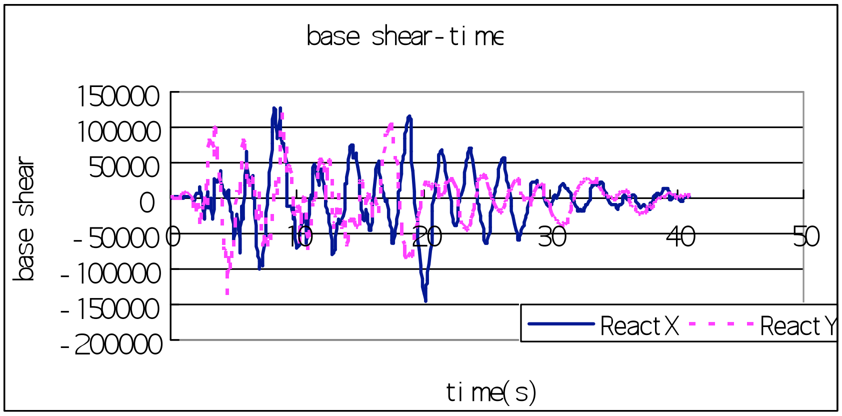


Figure 7 The base shear-time curve under the action of the first group of natural wave

4.2 The inter-story drift ratio

The inter-story drift ratios and their corresponding floor numbers are listed in Table 3.

Table 3 shows that the maximum inter-story drift ratio of T1 in X direction is 1/175 (on the fourth floor) and that in Y direction is 1/199(on the sixth floor); that the maximum inter-story drift ratio of T2 in X direction is 1/231 (on the fourth floor) and in Y direction 1/148 (on the twentieth floor); and that the maximum inter-story drift ratio of T3 in X direction is 1/204 (on the sixth floor) and in Y direction 1/270 (on the fifth floor). The maximum inter-story drift ratios of the three towers are all less than 1/100 that is the regulation of Literature 5. Figure 8 gives the curves of the maximum inter-story drift ratios of the three monomer structures under the first group of natural wave.

Table 3 The inter-story drift ratio corresponding to each seismic waves

Seismic wave groups	Direction	T1		T2		T3	
		Inter-story drift ratio	Floor number	Inter-story drift ratio	Floor number	Inter-story drift ratio	Floor number
The first group of natural wave	X	1/195	4	1/231	4	1/204	6
	Y	1/199	6	1/168	20	1/270	5
The second group of natural wave	X	1/234	5	1/318	20	1/284	6

	Y	1/220	8	1/148	20	1/401	11
Artificial wave	X	1/175	4	1/284	6	1/254	9
	Y	1/212	6	1/240	20	1/335	13

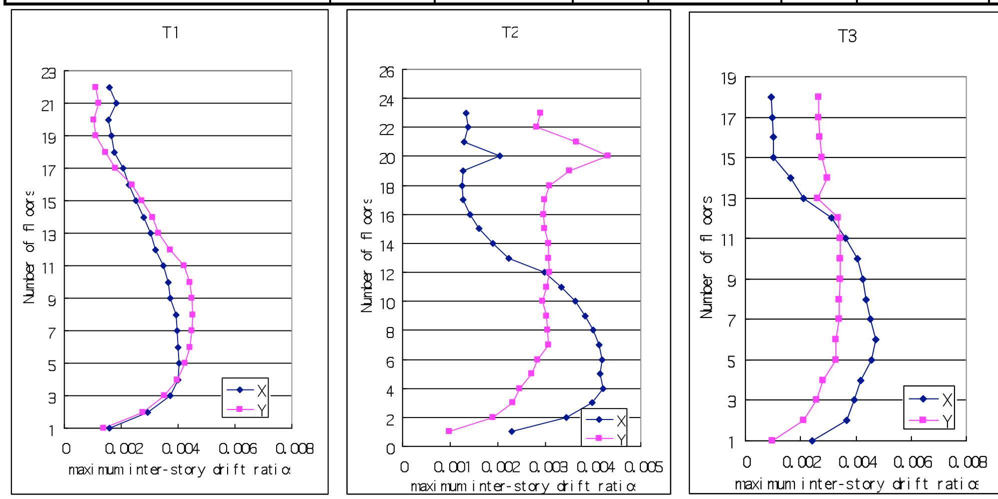


Figure 8 The curves of the maximum inter-story drift ratios under the action of the first group of natural wave

4.3 Top displacement and torsion

Table 4 shows the maximum top displacement under action of the three groups of seismic waves.

The maximum top displacements of T1 are respectively in X and Y direction 267mm and 266mm, which are 1/377 and 1/378 of total height of the structure separately; the maximum top displacements of T2 are respectively in X and Y direction 252mm and 270mm, which are 1/400 and 1/373 of total height separately; the maximum top displacements of T1 are respectively 247mm and 249mm, which are 1/341 and 1/337 of total height. Figure 9 shows the curves of the top displacements under the first group of natural wave.

Table 4 The maximum top displacement (m)

Seismic wave groups	Direction	T1		T2		T3	
		U	**U/H**	**U**	**U/H**	**U**	**U/H**
The first group of natural wave	**X**	0.234	1/430	0.252	1/400	0.247	1/341
	Y	-0.266	1/378	-0.27	1/373	-0.249	1/337
The second group of natural wave	**X**	-0.23	1/437	0.167	1/603	0.162	1/518
	Y	-0.244	1/413	-0.25	1/402	-0.185	1/455
Artificial wave	**X**	0.267	1/377	0.214	1/471	0.194	1/432

	Y	-0.253	1/398	-0.257	1/391	0.167	1/502

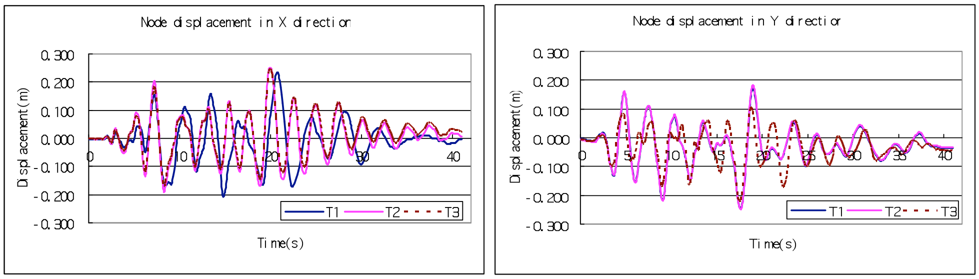


Figure 9 The top displacement time-history curves under the action of the first group of natural wave

We can see from Figure 9 that the top displacement time-history curves in X direction of T2 and T3 are basically in coincidence and that the top displacement time-history curves in Y direction of T1 and T2 are basically in coincidence. Therefore, in X direction, T2 and T3 are in fully cooperative work, however, the peak time of the displacement of T1 is lagging behind that of other two towers and in Y direction, T1 and T2 are in fully cooperative work, but the peak time of the displacement of T3 has a relatively delay.

This phenomenon shows that there are obvious torsional properties among the towers under earthquake.

4.4 The mid-span vertical displacement of the joint gallery

Under the action of each group of seismic waves, the maximum vertical displacement of the 50 meter-span joint gallery is shown in Table 5; under the action of the first group of natural waves, the vertical displacement-time curve of the 50 meter-span joint gallery is shown in Figure 10. We can see that the maximum vertical displacement is -0.130m, which is increased by 61% compared with that under the static action, and the corresponding U/L equals to 1/385, which is still within the allowable range.

Table 5 Large-span vertical displacements corresponding to each seismic waves(m)

Seismic wave groups	TianAx	TianAy	TianBx	TianBy	RenAx	RenAy
UZ	-0.130	-0.130	-0.118	-0.118	-0.126	-0.129
UZ/span	1/385	1/385	1/424	1/423	1/397	1/386

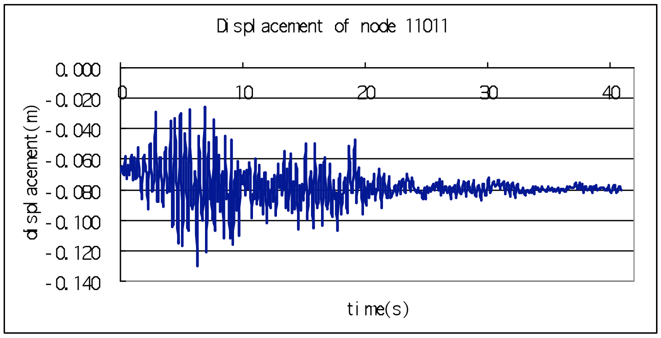


Figure 10 The mid-span vertical displacement of the 50 meter-span joint gallery

4.5 Structural damage

4.5.1 The shear wall damage

Under the action of the first group of natural waves, the size and distribution **o**f the compressive damage of shear wall structure is shown in Figure 11. Therein, compressive damage of shear wall structure has following characteristics:

(a) The compressive damages of shear wall structure appear according to the orders as follows: coupling beams of T1 in Y direction → coupling beams of T2 and T3 in X direction → coupling beams of T1 in X direction → coupling beams of T2 and T3 in Y direction → walls of the first story structure;

(b) The compressive damages of shear wall are concentrated on the parts such as coupling beams, the connection between T2 and the 50m joint gallery, and walls at the bottom of strengthened stories;

(c) The compressive damages of shear wall of T2 occur early from the 19th to 22nd floor and developed gradually until a closed layer of compressive damages forms. To increase its mechanical behavior under large earthquakes, it is suggested that the additional seismic diagonal bracing be set in the shear wall.

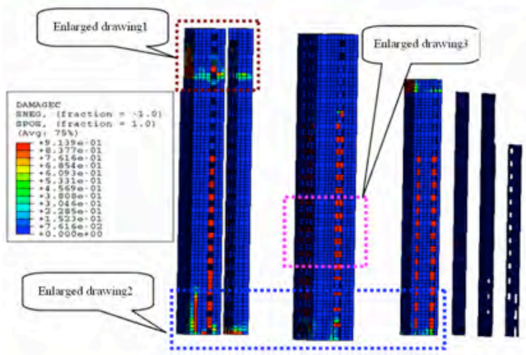
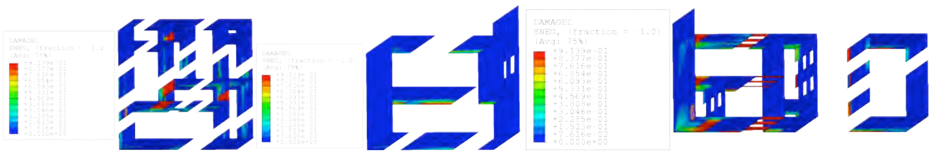


Figure 11 The compressive damages of shear wall

(a) The view in the southeast direction (b) The view in the northwest direction

Figure 12 Enlarged drawing1: The compressive damages of the shear wall from 19^{th} to 23^{rd} floor of T2



(a)T1 (b) T3 (c) T2

Figure 13 Enlarged drawing2: The compressive damages of the shear wall of strengthened areas from 1^{st} to 3^{rd} floor

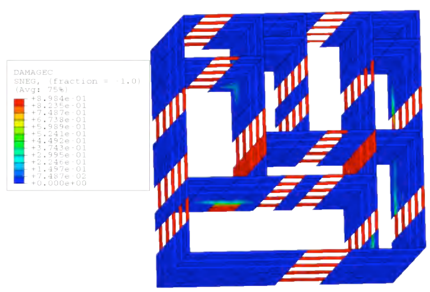


Figure 14 Enlarged drawing 3: The compressive damages of the shear wall of T1 from 1^{st} to 3^{rd} floor

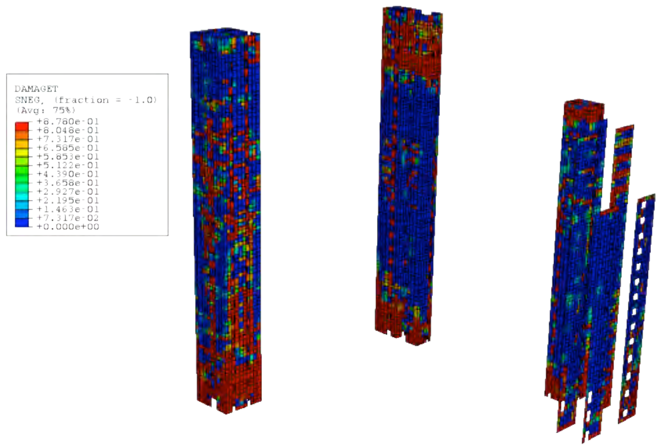


Figure 15 The tensile damages of the shear wall

The tensile damage of the shear wall is shown in Figure 15, from which we can conclude that the maximum tensile damage coefficient is 0.878, and the regions where the damage coefficient ranges from 0.4 to 0.88 mainly distribute in such areas as the walls from the 1^{st} to 3^{rd} floor, the walls of T2 from the 20^{th} to 23^{rd} floor and the coupling beams.

4.5.2 Frame member damage and plasticity

The main frame columns and beams are not damaged, the steel bars in which are all under elastic state. The compressive damage occurs at the coupling beams of the core tube of T2, where the steel bar steps into plasticity and the maximum plastic strain is 1.734E-3, which is less than the plastic limit strain 0.025. The whole frame members show a good performance under large earthquakes.

4.5.3 The plastic strain of the connected truss

Under large earthquakes the plastic strain occurs at diagonal bracing members at the end of the main truss of SPAN1 and reinforced diagonal bracing in the shear walls and the maximum plastic strain is -7.344e-4, which is less than the plastic limit strain. The plastic strain has not appeared in the other steel members of SPAN1. Since the occurrence of plastic strain is at the location of the 50m main truss, it is proposed to increase the cross-section of the diagonal bracing members.

There is a plastic strain at the reinforced diagonal bracing at the NECK position, so it is suggested that the reinforced diagonal bracing extend to the 21^{st} and 22^{nd} story.

The steel members of SPAN2 don't develop any plastic strain.

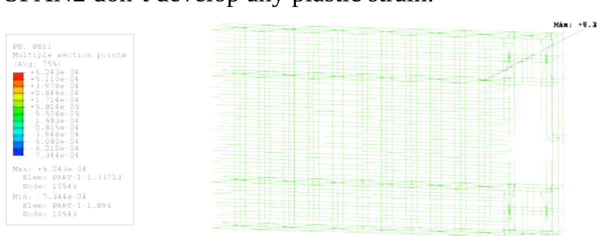


Figure 16 The plastic strain of the 50 meter-span steel structure

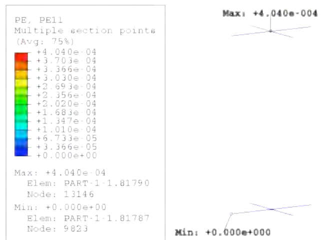
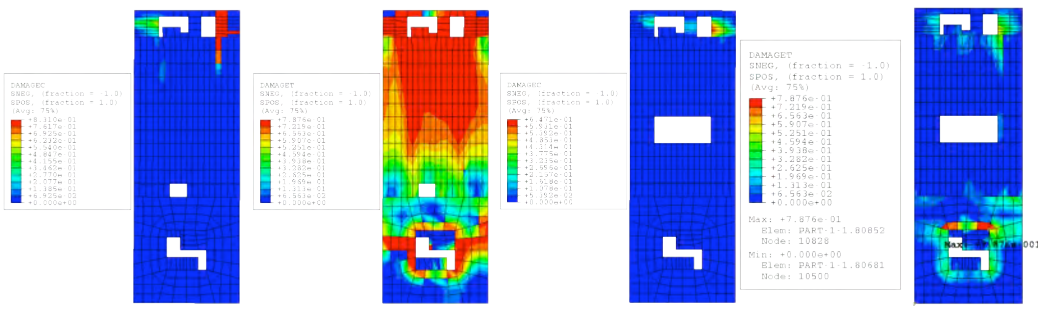


Figure 17 The plastic strain of the reinforced diagonal bracing

4.5.4 Floor damage

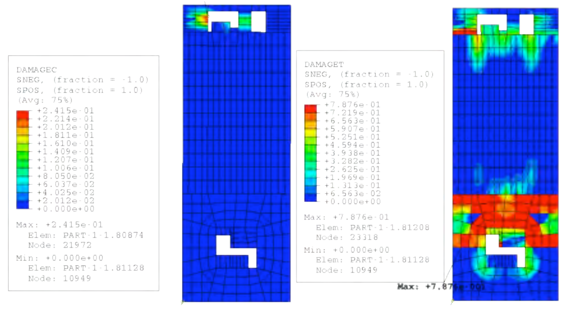
At the area where the floor is connected with T1 and T2, the floor experiences the compressive and tensile damages, and the maximum compressive damage coefficient is 0.83. The plastic strain doesn't occur to the steel bars in the floor.

Under rare earthquake, the floor is responsible for the distribution and coordination of the seismic shear force among the various pieces shear walls, so there will exist inevitably tension crack phenomenon for the floor. After the slab cracking, its tensile stiffness is significantly weakened and then the earthquake force will be unloaded from the floor, which will not cause the crack propagation. At the same time the compressive bearing capacity of the cracking slab has not been affected and therefore, the floor can still bear the vertical load in the way that the steel bar is subjected to tension and the concrete is subjected to compression without collapse phenomenon.



DC DT DC DT

The floor damage of the 20th floor The floor damage of the 21st floor



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The floor damage of the 22nd floor

5. Conclusions

In the paper the dynamic elastic-plastic time-history analysis is made on a certain connected structure under rare earthquake with the application of ABAQUS6.6.1. Through the analysis, conclusions can be made as follows:

(1) After the dynamic elastic-plastic analysis under rare earthquake, the structure remains vertical and the inter-story drift angle is less than 1/100, which meets the fortification level requirements "No collapsing with strong earthquake".

(2) The structural damage, born in the coupling beams, develops and is mainly concentrated on the parts such as coupling beams, the connection between T2 and the 50m joint gallery, and the bottom strengthened areas from the first to third floor. The main concrete frame is not damaged and the steel bars don't yield. The plastic strain occurs to the diagonal bracing of the steel truss of the 50 meter-span joint gallery SPAN1 and the steel members of SPAN2 don't develop any plastic strain. The floor of the 50 meter-span joint gallery experiences the compressive damage, but no plastic strain occurs to the steel bars.

According to the above damages, it is proposed to increase the wall thickness at the NECK position and in the bottom reinforced areas and to increase the cross-section of the diagonal bracing of the 50m-span steel truss.

Overall, we can see that ABAQUS can be applied well to make the dynamic elastic-plastic time-history analysis. The elastic-plastic inter-story drift angle can reflect the overall performance of the structure under rare earthquakes; according to the concepts such as the damage provided by ABAQUS, we can better evaluate the stiffness degradation of the shear wall, floor and other components, and thus the change of the overall stiffness can be reflected. By comparing the displacement-time curves of several tower vertices, the cooperative work ability between the towers is clearly exhibited.

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