16th World Conference on Earthquake, 16WCEE 2017
Santiago Chile, January 9th to 13th 2017
Paper N° 4959 (Abstract ID)
Registration Code: S-C1461745713

Elasto-plastic Time History Analysis of A High-rise Frame-RC Core Wall Structure Based on Two Softwares

C.T. Guo(1), D.Y. Zhou(2), X.H Wu(3), Y. Zhou(4), L.F Liu(5)

(1) *Ph.D Student, Tongji University, College of Civil Engineering, China, 0595changtuan_guo@tongji.edu.cn*

(2) *Professor, Tongji University, College of Civil Engineering, China, concrete@126.com*

(3) *Associate Professor, Tongji University, College of Civil Engineering, China, xhwu@tongji.edu.cn*

(4) *Professor, Tongji University, College of Civil Engineering, China, yingzhou@vip.163.com*

(5) *Ph.D Student, Tongji University, College of Civil Engineering, China, liulingfeitj@163.com*

Abstract

A concrete filled steel tubular laminated column and reinforced concrete core wall structure is located in a region of seismic intensity 7. There are three floors underground, and 43 floors are on the ground. The building height is 198.9 meters. In this paper, in order to study the seismic performance of the structure under rare intensity of 7 earthquake, NosaCAD and Perform-3D were used to perform elasto-plastic time history analysis. Frame element models of NosaCAD and Perform-3D are both consisted of three different stiffness segments: a linear elastic segment located in the middle and another two elasto-plastic segments located at both ends. Trilinear moment-curvature hysteretic model is adopted for the elasto-plastic segments of concrete beams and steel reinforced concrete beam. And fiber model is employed to describe the nonlinear behavior in the elasto-plastic segments of column. In NosaCAD, the flat shell finite element is used for shear wall and elastic plate element is used to simulate the structural slab. In Perform-3D, macro layered element is adopted to simulate shear wall component and rigid diaphragm assumption is employed. The results indicate that under rare earthquake, the yielding hinges firstly occur on some coupling beams. The limited concrete has crushed on few shear walls and the column was only damaged with some tensile cracks. The maximum inter-story drift of the structure can meet the requirement of Chinese code. The sequence and distribution of damages on components of the structure are reasonable, which can dissipate some dynamic energy. The structure satisfies the code requirements of no collapse under rare earthquakes. The calculation results, such as the global response, weak story and damage condition got by the two softwares show an agreement to a certain degree. It is suggested that the envelope value of multi-software analysis can be used to ensure the safety of such complex structures.

Keywords: Reinforced concrete structures; Damage assessment; Nonlinear methods

1. Introduction

1.1 Background

In the recent years, due to the needs for function of building and urban planning, as well as the shortage of construction land, the numbers of structural floor and height of high-rise building increase markedly. In order to make the design unique and add beauty to cities, many new buildings adopt novel structural styles, complexity of structures are inevitable for these special buildings. This requires structural engineers to entirely understand how these structures behave, especially in future earthquakes.

Besides theoretical and experimental investigation, with the development of software and personal computer, nonlinear analysis for complex irregular buildings has gradually come to maturity. Earthquake engineering is relying more and more on nonlinear analysis as a tool for evaluating structural performance

NosaCAD and PERFORM-3D are widely applied for nonlinear structure analysis. NosaCAD is developed with ObjectARX, second development tool of AutoCAD, and runs in the AutoCAD environment. The powerful geometric processing function of AutoCAD can be used to established and edited structure analysis model. The analysis model of PERFORM-3D can be transformed from EATBS or SAP 2000, however, only the parameters for elastic analysis are transformed from EATBS or SAP 2000, and the parameters for nonlinear analysis would be calculated and input in further step. In this paper, in order to study the seismic performance of an out-code structure under rare intensity 7, NosaCAD and Perform-3D were used to perform elasto-plastic time history analysis.

1.2 Overall Project Description

The structure, which is used as a regional main building of a state-owned group, is located in Haizhu District of Guangzhou City, Guangdong Province. It is an office building , with three on the ground. Its height is 198.9 meters, with a planned floor-space of approximately 46,000 square meters underground, and a planned floor-space of approximately 111,000 square meters on the ground. The standard layer is 4.20 meters high. The concrete filled steel tubular laminated column and the reinforced concrete core wall system can resist lateral and longitudinal loads. Joints of ring beams are used to connect filled steel tubular laminated columns and reinforced concrete beams. The plan layout of standard floor can be seen in Figure 1.

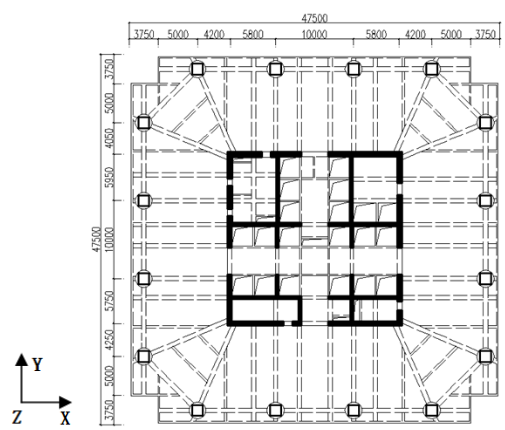


Fig. 1 – Plan Layout of the Standard floor

Because the height of this building exceeds the specified maximum height of Steel Reinforced Concrete (SRC) frame and Reinforced Concrete (RC) core wall system in Chinses code, this building belongs to code-exceeding design. An elasto-plastic nonlinear time history analysis is necessary to investigate its seismic performance, which can also provide guidance and suggestion to structural engineers for similar structure concerned. A detailed three-dimensional finite element model is built in SATWE firstly. Next the model is transformed from SATWE to NosaCAD, then the elements of shear wall are redivided. Finally the model is transformed from NosaCAD to Perform-3D. Then results of nonlinear time history analysis in the two software are compared to evaluate seismic performance of the structure mentioned above [1].

1.3 Design parameter of office building

The design parameter of office building are listed in Table 1.

Table 1 Design Parameter

Project Name	Southern Headquarters(Zone A) Building of CCCC Group				
Site Classification	II	Seismic Fortification Intensity	7	Safety Classes of Building Structure	II
Numbers of Structural layers	46	Classification of Design earthquake	The First Group	Height	208m
Classification of seismic protection of buildings	Key Project(II)		Structural System	Concrete filled steel tubular laminated column-reinforced concrete core wall system	

2. Finite element model for structural components

2.1 Finite element model built with NosaCAD [2] (version 2010)

In this paper, frame element model is consisted of three different stiffness segments, one of which is a linear elastic segment in the middle part of element, and other two are elasto-plastic segments at both ends of element (Figure 2).

The beams mainly suffer bending moment, and the plastic hinge only occurs at both ends of the beams, for which beam element is divided into three different stiffness segments [3]: a linear elastic segment located in the middle and another two elasto-plastic segments located at both ends. Trilinear moment-curvature hysteretic model is adopted for the elasto-plastic segments of concrete beams and steel reinforced concrete beam [4-5]. The trilinear moment-curvature hysteretic curve is shown as Figure 3 [4-5]. Considering columns, including inclined columns, bear bending moments in two directions as well as dynamic axial force, fiber model is employed to describe the nonlinear behavior in the elasto-plastic segments of column. The constitutive model of concrete in fiber model is shown as Figure 4. Tensile stress-strain is also considered in constitutive relation of concrete. Reinforcement yielding of column and beam means the occurrence of plastic hinge, and concrete crushing represents the failure of members. The ideal elastic-plastic constitutive model, yield hardening taken into account, is adopted for steel and rebar, and the post-yielding elastic modulus value is 1% of the initial one.

The flat shell finite element, which is composed of diaphragm and plate, is used for shear wall. The flat shell finite element model possesses rotational degrees of freedom in diaphragm [6], so that coupling beam element can be connected to shear wall with compatibility of deformation. In the nonlinear shell element, only the nonlinear property of diaphragm is taken into account and the plate is regarded as linear. For the nonlinear diaphragm, an orthogonal anisotropic concrete model based on equivalent uniaxial stress and strain relationship is adopted [7] together with a biaxial strength envelope [8], meanwhile rebar is supposed to disperse in the element in certain directions according to the reinforcement. The concrete hysteresis curve of equivalent uniaxial stress-strain relationship is the same as that in fiber model (Figure 4), and furthermore, the influence of the stress in orthogonal direction is considered. The ideal elastic-plastic model, taking into account the yield hardening, is still adopted for the reinforcement.

To improve the accuracy of structural analysis in NosaCAD, elastic plate element is used to simulate the structural slab.

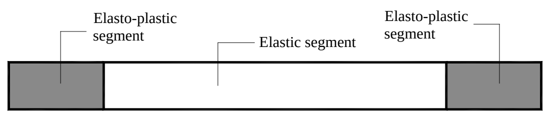


Fig. 2 – The frame element composed of three stiffness segments

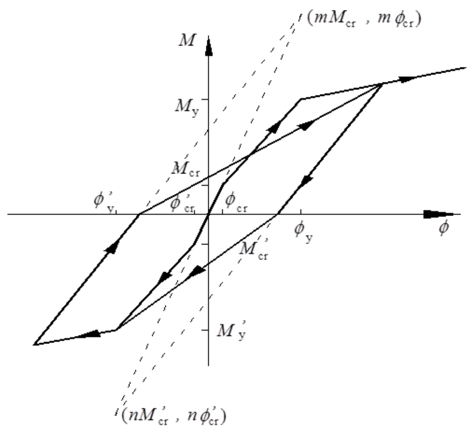


Fig. 3 –Tri moment-curvature hysteretic model

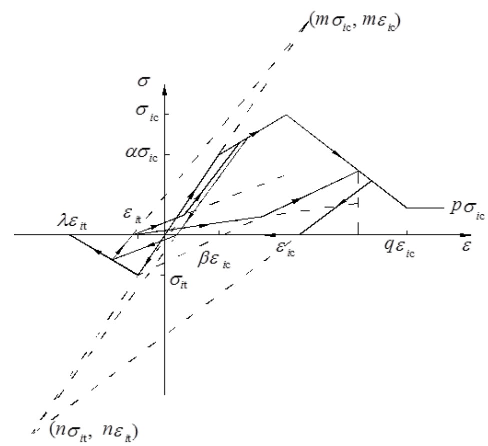


Fig. 4 – Concrete constitutive model

2.2 Perform-3D finite element model

Similarly as in NosaCAD, the moment-curvature hysteretic model are adopted for the frame members, which mainly suffers bending moment, and the fiber model is employed to describe the nonlinear behavior for the frame members, which bear bending moments in two directions as well as dynamic axial force. Unlike the elasto-plastic frame member in NosaCAD, which is consisted of two elasto-plastic segments and one elastic segment in the middle regularly, the elasto-plastic frame member in Perform-3D can be compounded of elastic segment and elasto-plastic segment in arbitrary formation. In this paper, frame member in Perform-3D also consists of three components, one is linear elastic and the others are elasto-plastic. Their distribution is the same as that in NosaCAD. The bilinear and trilinear moment-curvature hysteretic models are adopted for the elasto-plastic segments of steel beams, concrete beams and steel reinforced concrete beam, respectively.

Macro layered element is adopted in Perform-3D to simulate shear wall component. One dimensional fiber element is used for simulating the compression-bending effect, while using nonlinear or linear shear model for the shear effect in plane and elastic model for the bending and shear and torsion effect out of plane [9].

In NosaCAD, the wall element has rotational stiffness in plane at node, but Perform-3D wall element has no such stiffness, so the embedded rigid beam should be added to coordinate the deformation between the wall and coupling beam in Perform-3D model (Figure 5).

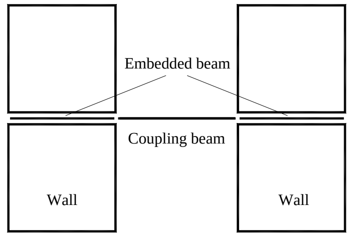


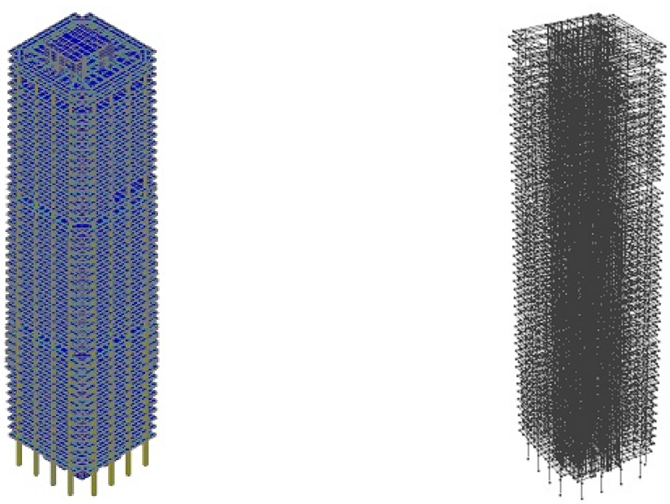
Fig. 5 – Connection between wall and coupling beam in Perform-3D

2.3 Analysis model of the structure

Models in Perform-3D and NosaCAD are both based on the assumptions as follows:

(1) In the models, the basement was also built, thereby the bottom of basement was regarded as the fix end;

(2) The calculated mass of the model was consisted of 100% dead load, 100% additional dead load and 50% live load. The mass of NosaCAD model was 160,810 tons while that of Perform-3D model is 161,040 tons, which shows littledifference.



(a) NosaCAD model (b) Perform-3D model

Fig. 6 – The model of structure

3 Natural vibration characteristics and input ground motions

3.1 Natural vibration characteristics

Modal analysis is done to obtain the fundamental dynamic characteristics, and to verify the accuracy of models in NosaCAD and Perform-3D. The results of modal analysis are shown on Table 2. In Figure 7, the first three vibration modes in NosaCAD model are given.

Table 2 – Natural vibration period of structure

Order	Period (s)		Deviation Rate	Description
	NosaCAD	Perform-3D		
1	4.891	4.891	0%	Translation in X
2	4.754	4.751	0.06%	Translation in Y
3	3.69	3.488	5.47%	Torsion
4	1.489	1.467	1.48%	Second translation in X
5	1.393	1.373	1.44%	Second translation in Y
6	1.37	1.304	4.82%	Second Torsion

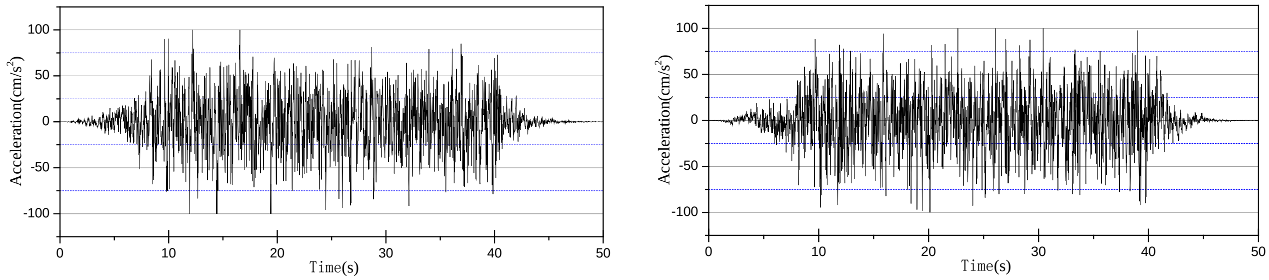
As can be seen in Table 2, first of all, the first six order periods in NosaCAD show good agreement with those in Perform-3D, and the value in NosaCAD is a little higher than that in Perform-3D. The maximum deviation rate of periods between two software is 5.47%, which occurs at the torsional mode. Because the structure is a regular structure and the ratio of torsion period to translation period is less than 0.75, which means that the torsional effect would not be motivated obviously, the deviation of torsional period between two software have little influence on the contrast analysis of numerical results in NosaCAD and Perform-3D; secondly, because the sequence of vibration mode in NosaCAD and Perform-3D is identical, and the deviation rate between two models' mass is 0.14%, the stiffness distribution and mass of structural model in NosaCAD are basically in accordance with those in Perform-3D.

(a)1st vibration mode (b)2nd vibration mode (c)3rd vibration mode

Fig. 7 – The first three vibration modes in NosaCAD model

3.2 Input ground motions

According to Chinese Code for Seismic Design of Building (CCSDB, GB50011-2001), the site soil in Guangzhou belongs to type II. It is specified in TSCSTB that no less than two earthquake records and a synthetic accelerogram should be selected for elasto-plastic time history analysis. Considering the power spectral density properties of type II site soil, three different ground motions were simulated as input accelerations to the model: (a) the NW1 record; (b) the NW2 record; and (c) the AW record, which is formed artificially according to the CSDB. According to the results of modal analysis, the overall stiffness of structure in x direction is bigger than that in y direction. X direction of structure is set as principal direction of structure. Three different ground motions were inputted in two horizontal directions. The principal direction of earthquake record is in accordance with principal direction of structure. Fig.8 show the time history acceleration of AW record in two directions.



(a)Time history of acceleration in x direction (b)Time history of acceleration in y direction

Fig. 8 – AW accelerogram (x, y-direction)

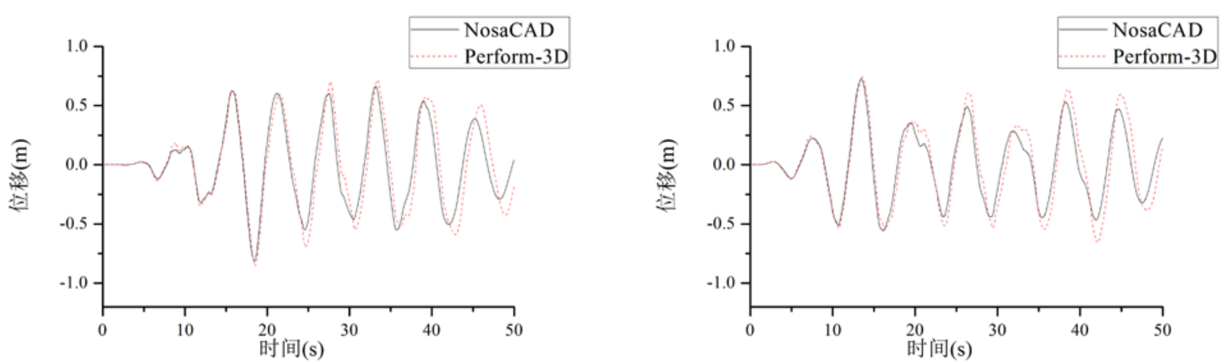
Since seismic performance under rare earthquake was mainly investigated in this paper, the peak ground acceleration (PGA) of selected earthquake accelerograms were scaled to 0.220 g, corresponding to earthquakes of rare levels. The ratio of PGA in X direction to PGA in Y direction is 1:0.85.The duration of NW1 record, NW2 record and AW record is 50s, 66.98s and 102.98s separately. As specified by Technical Specification for Concrete Structures of Tall Building (TSCSTB, JGJ3-2002) (Ministry of Construction of the People's Republic of China 2002), a damping ratio of 0.05 for concrete filled steel tubular laminated columns–RC core wall structural system was adopted, and Rayleigh damping was used in integration equation. Newmark-β method is used to solve integration equation, where γ is 0.50 and β is 0.25.

4 Result of time history analysis

4.1 Comparison of roof displacement in NosaCAD model and Perform-3D model

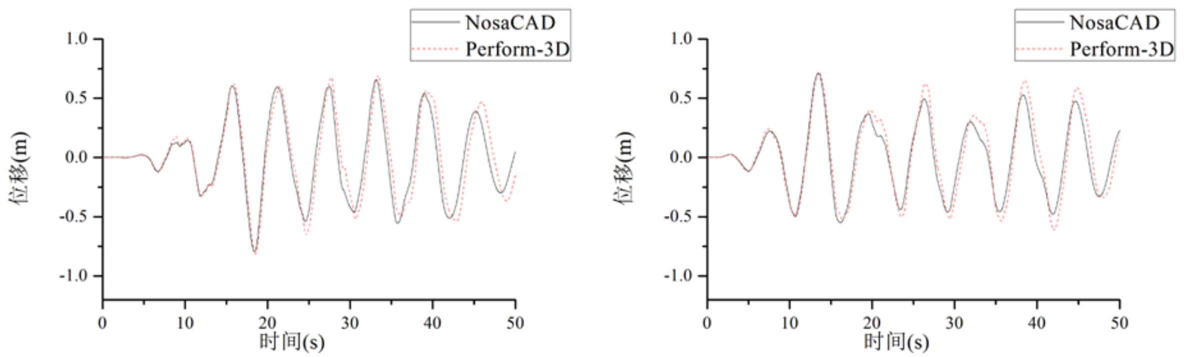
Because of the structural style, two nodes in Fig.1 are used to investigate the roof displacement of structure. For the structure is a super high-rise building and it is located in 7 seismic intensity area, seismic response of the structure is relatively large. There is a large difference among the seismic response of three different earthquake records, whose PGA are the same.

Figs.9 shows the roof displacement time history of node N1(46th floor) under rare intensity 7, which is located at the roof corner of the shear wall. Fig.10 gives the roof displacement time history of N2(45th floor) under rare intensity 7 at the side column. It can be found that the displacement response of NosaCAD is close to that of Perform-3D not only in amplitude but also in step.



(a) Displacement time history in direction X (b) Displacement time history in direction Y

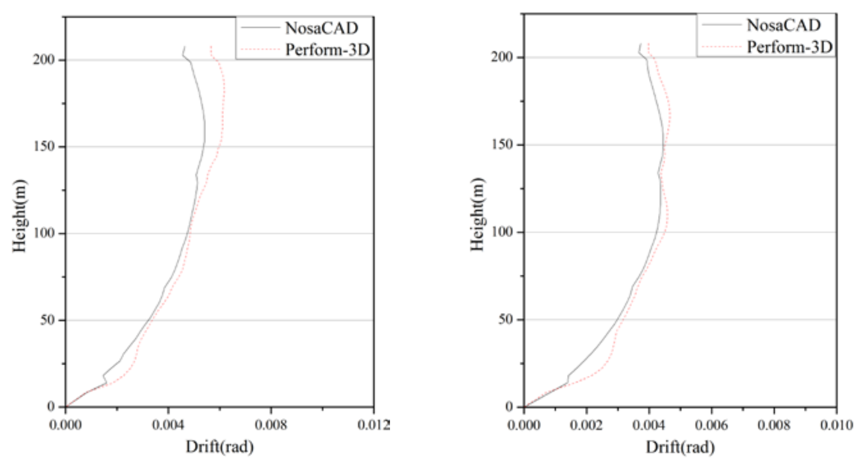
Fig. 9 – Displacement time history of node N1(46th floor) under AW of rare intensity



(a) Displacement time history in direction X (b) Displacement time history in direction Y

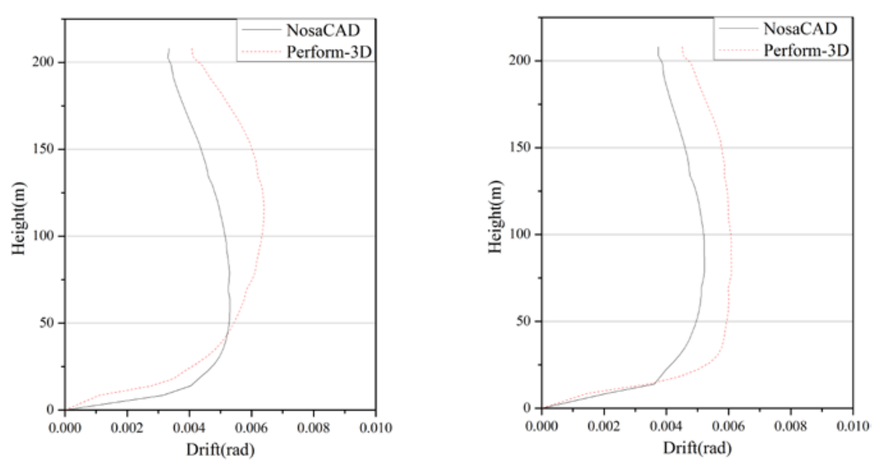
Fig. 10 – Displacement time history of node N2(45th floor) under AW of rare intensity

4.2 Comparison of inter-story drift in NosaCAD model and Perform-3D model



(a) Inter-story drift in direction X (b)Inter-story drift in direction Y

Fig. 11 – Comparison of inter-story drift of N1 under AW of rare intensity



(a) Inter -story drift in direction X (b)Inter-story drift in direction Y

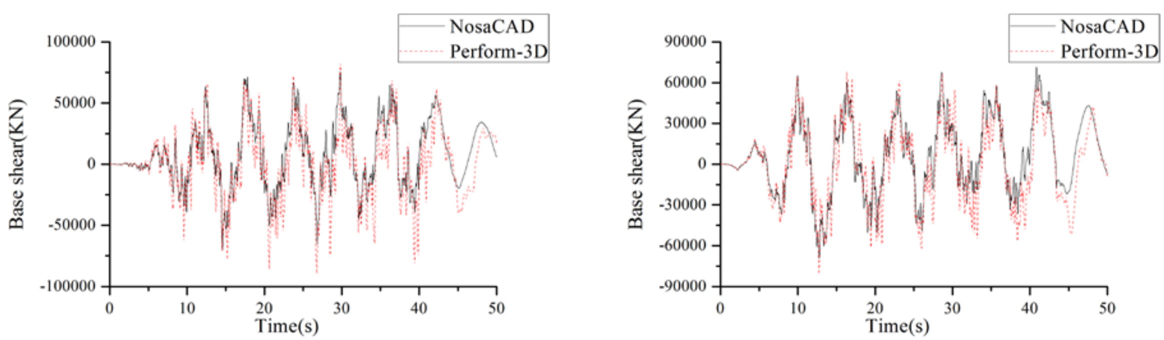
Fig. 12 – Comparison of inter-story drift of N1 under NW1 of rare intensity

For the elastic diaphragm hypothesis was applied in these floor model, the strings of nodes along the vertical direction at positions N1 and N2 (Figure 1) were chosen to investigate the inter-story drift. N1 was on the roof corner of the shear wall, while N2 was on the side column. Under rare intensity, the most serious structural damage was generated by AW. In NosaCAD model, the maximum inter-story drift, which was caused by AW, occurred in N2 and its value is 1/174. In Perform-3D model, the maximum inter-story drift was caused by NW1. It occurred in N2 and its value is 1/153. The maximum inter-story drift of the structure is smaller than the limited value of 1/100 stipulated by Chinese code.

The inter-story drift envelop curves of N1 under AW and NW1 in direction X and direction Y are shown in Fig.11 and Fig.12, respectively. Comparisons between NosaCAD model and Perform-3D model are presented as well. As can be found, the trend of inter-story envelops of N1 are almost the same between NosaCAD and Perform-3D.

4.3 Base shear in NosaCAD model and Perform-3D model

Fig.13 gives comparison between the base shear time history of NosaCAD model and Perform-3D model under rare intensity 7 .The base shear and the ratio of base shear to weight when subjected to these three excitation under rare intensity in both direction X and direction Y calculated by NosaCAD and Perform-3D are listed in Table 3 and Table 4, respectively.



(a) base shear in direction X (b) base shear in direction Y

Fig. 13 – Comparison of base shear under AW of rare intensity

Table 3 – Base shear and the ratio of base shear to weight calculated by NosaCAD

Input ground motion		Direction X		Direction Y	
		Base shear/kN	**ratio of base shear to weight**	**Base shear/kN**	**ratio of base shear to weight**
Rare intensity	**AW**	75622.3	0.0479	71790.2	0.0455
	NW1	66532.5	0.0422	69888.2	0.0443
	NW2	67696.9	0.0429	51100.1	0.0324

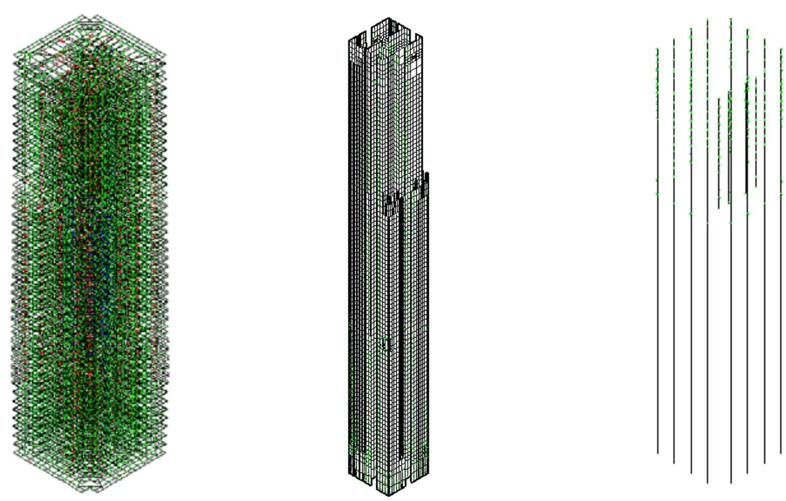
Table 4 – Base shear and the ratio of base shear to weight calculated by Perform-3D

Input ground motion		Direction X		Direction Y	
		Base shear/kN	ratio of base shear to weight	Base shear/kN	ratio of base shear to weight
Rare intensity	AW	89487.6	0.0568	80921.7	0.0513
	NW1	74466.6	0.0472	68449.3	0.0434
	NW2	63678.1	0.0404	49026.8	0.0311

It can be seen that in Fig.13, in the early stage of geological process, base shears at the same time step of two pieces of software are nearly the same. Those start to look different at 40s, after which the value of NosaCAD is bigger than that of Perform-3D at the same step, but the trend of base shear time history of two pieces of software are nearly identical. It can be found that after 40s, more members in NosaCAD model was into the nonlinear period, which results in the reduction of structural stiffness and smaller base shear.

It is shown that in Table 3 and 4, the results from each software both clearly state that the maximum base shear was generated by AW and that the base shear is stronger in direction X than in direction Y when subjected to AW. In NosaCAD model, the maximum base shears of two directions under NW1 are 66532.5kN, 69888.2kN respectively, which are both smaller than those under AW.

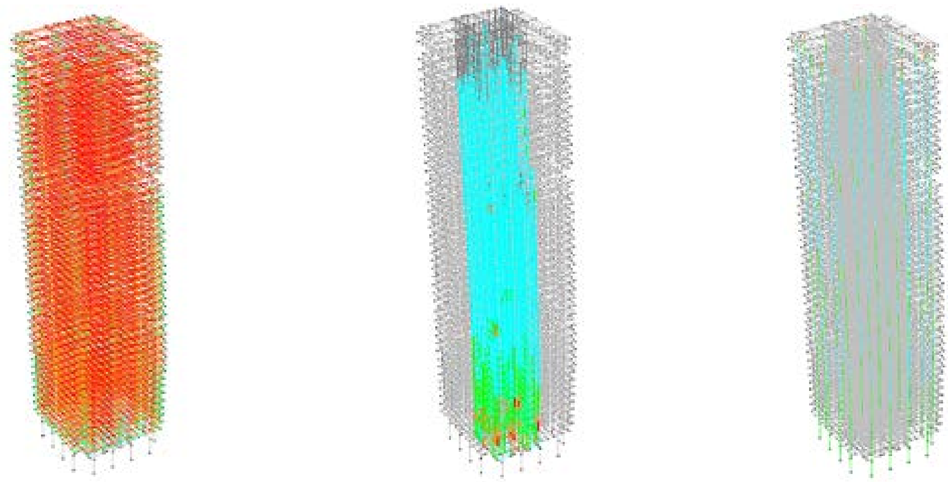
Since the severest response was generated by AW under rare intensity 7, responses caused by AW were taken to illustrate the damage development in the structure. Figure 14 and Figure 15 show the damage pattern of the structure analyzed when AW was input and direction X was set as the primary direction by both NosaCAD and Perfrom-3D, respectively. It can be deduced that the damage of the structural model calculated by these two different software develops basically to the same extent by the same sequence.



(a)beam (b)shear wall (c)column

/——Concrete cracked. ●——Plastic hinge or rebar yielded. ■——Concrete crushed.

Fig. 14 – Damage patterns of structure under rare intensity 7 (NosaCAD)



(a)Plastic hinge on beam (b) Crushing of concrete on shear wall (c)Plastic hinge on column

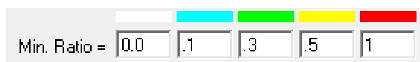
legend: 

Fig. 15 – Damage patterns of structure under rare intensity 7 (Perform-3D)

It can be seen in Fig.14 and Fig.15:

(1)In NosaCAD model and Perform-3D model, plastic hinges occur in many frame beams at the bottom of structure and in middle floor. Many coupling beams were into nonlinear period, and a few of them reached ultimate state. The frame beams and coupling beams act as the first and major anti-seismic component that dissipates a great deal input energy of rare earthquake. The damage of beam members decreases structural stiffness, which effectively reduces the earthquake force.

(2)In NosaCAD model and Perform-3D model, few crushing of concrete both occurred at the lowest part of the core wall. Tensile cracks of shear wall mainly generated at the corner of structure. In NosaCAD model, columns were only damaged with tensile cracks. In NosaCAD model and Perform-3D model, no columns came into yielding state. (In Perform-3D model, for tensile strength is not taken into account, tensile cracking behavior of core wall is not provided.)

(3)According to the damage development in the structure of NosaCAD and Perform-3D, the damage firstly occurred on some coupling beams, and then yielding hinge generated on the frame beams. From then on, the concrete of some shear walls were crushed. Judging from the sequence of damage development, the structural design meets well the principles of “strong column and weak beam” and “strong coupling wall column and weak coupling beam”.

(4)The yielding failure that firstly occurred in coupling beams dissipates a great deal of input energy of rare earthquake, which accordingly guaranteed the security of shear walls against earthquake force. After all, the overall structure can meet the requirement of “no collapse under rare earthquake”.

5. Conclusions

In this paper, elasto-plastic time history analysis conducted by two software presented numerical analysis data to obtain seismic performance of a high-rise building under ambient vibration. Based on the detailed analysis of numerical study, conclusions are summarized as follows:

(1) Indicators of the overall responses obtained from NosaCAD and Perform-3D meet each other well. Tough different basic hypotheses are adopted in these two different software, like the flat shell finite model is employed in NosaCAD to simulate shear wall and slab while macro layered element is employed in Perform-3D, illustrated by the roof displacement, little difference was observed.

(2) It is indicated from these two software that plastic hinge was first observed in the coupling beams on the upper floor, and then damage occurred on the frame beams. Thereby beam members can help dissipate the input seismic energy, consequently avoid or degrade the damage in the supporting structures on the lower part of the main structure.

(3)Under rare earthquake, although plastic hinges occurred on the coupling beams and frame beams, columns only cracks at the top floor and few concrete of shear crushed at the bottom. Therefore the structure meets the requirement stipulated in Chinese Design Codes of "no collapse under rare earthquake". Beyond that, deformation of the structure stays inside the limitation stipulated in Chinese Design Codes as well.

(4)Under rare earthquake, few concrete of core wall crushed at the bottom, and the tensile cracks of shear wall mainly generated at the corner of structure. Cracks of columns mainly occurred on the top floor. In order to reduce damage, increasing reinforcement of these parts is suggested.

6. References

[1] Wu Xiaohan. The model transformation from NosaCAD to ABAQUS and PERFORM-3D and nonlinear structure analysis by these software. Building Structure, 2012, 42 (Supplement), 207-212.

[2] Sun Fangtao, Wu Xiaohan, Lu Xilin. Research on earthquake resistance behavior of a long-span and cantilevered structure subjected to vertical motion. Structural Engineers, 2009, 25 (4), 71—77. (in Chinese)

[3] Gu Xianglin, SUN Feifei. Computer simulation of concrete structure. Shanghai: Tongji University Press, 2002.

[4] Lu Xilin. Seismic theory and application of complex high-rise structure. Beijing: Beijing Science Press,2007. (in Chinese)

[5] Lu Xilin, Jin Guofang, Wu Xiaohan. Theory and application of nonlinear finite element of reinforced concrete structure. Shanghai: Tongji University Press, 2002. (in Chinese)

[6] Long Zhifei, Cen Song. New Discussions for finite element method: Principle Program Progress. Beijing: China Water Power Press, 2001. (in Chinese)

[7] Darwin D, Pecknold D A. Analysis of RC shear panels under cyclic loading. Journal of the Structural Division, ASCE, 1976, 102 (2), 355-369.

[8] Kupfer, H.B. and Gerstle, K.H. Behaviour of Concrete under Biaxial Stresses. Journal of Engineering Mechanics Division ASCE 99 (4), 853-866.

[9] Zeng Ming, Liu Bowen. Basic Operation and Principle in Perform-3D. Beijing: Beijing Polytechnic University, 2012.