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Elasto-plastic time history analysis of a 117-story high structure

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Abstract. In Chinese Design Codes, for super high-rise buildings with complex structural distribution, which are regarded as code-exceeding buildings, elasto-plastic time history analysis is needed to validate the requirement of "no collapse under rare earthquake". In this paper, a 117-story super high-rise building is discussed. It has a height of 597 m and a height-width ratio of 9.5, which have both exceeded the limitations stipulated by the Chinese Design Codes. Mega columns adopted in this structure have cross section area of about 45 m^2 at the bottom, which is infrequent in practical projects. NosaCAD and Perform-3D, both widely used in nonlinear analyses, were chosen in this study, with which two model were established and analyzed, respectively. Elasto-plastic time history analysis was conducted to look into its seismic behavior, emphasizing on the stress state and deformation abilities under intensive seismic excitation.From the comparisons on the results under rare earthquake obtained from NosaCAD and Perform-3D, the overall responses such as roof displacement, inter story drift, base shear and damage pattern of the whole structure from each software show agreement to an extent. Besides, the deformation of the structure is below the limitation of the Chinese Codes, the time sequence and distribution of damages on core tubes are reasonable, and can dissipate certain inputted energy, which indicates that the structure can meet the requirement of "no collapse under rare earthquake".

Keywords: super high-rise building; elasto-plastic time history analysis; seismic behavior; rare earthquake

1. Introduction

In the recent decades, a large amount of tall buildings have been constructed in Mainland China. In order to make the design unique and add beauty to cities, many new buildings adopt novel architectural styles, such as the wheel-shaped Icon Hotel in Dubai (Berahman 2010). However, irregularity and complexity of structures are inevitable for these special buildings. From past experiences, structure irregularities could directly or indirectly cause severe damage or the collapse of these structures under strong earthquakes. Therefore, it requires structure engineers to entirely understand how these structures respond, especially in future earthquakes.

The seismic response of complex and irregular buildings has been theoretically investigated by several researchers (De Llera and Chopra 1995, Das and Nau 2003, Tremblay and Poncet 2005). Most of these studies focus on seismic performances of multi-story reinforced concrete (RC) or steel buildings. For the experimental studies on irregular high-rise buildings, Lu *et al*. (1999) studied the dynamic response of a complex structure with U-shaped floors and specially shaped slant columns. Ko and Lee (2006) preformed a shaking table test on a 1/12-scale model to investigate the seismic Performance of a 17-story high-rise RC structure with a high degree of torsional eccentricity and soft-story irregularity in the bottom of two stories. Besides theoretical and experimental investigation, with the development of software and personal computer, nonlinear analysis for complex irregular buildings has gradually come to maturity. Krawinkler (2006) believes that earthquake engineering is relying more and more on nonlinear analysis as a tool for evaluating structure Performance and nonlinear analysis will be a good trend. Yahyai *et al*. (2009) used finite element model analysis to study the nonlinear seismic response of Milad Tower. Epackachi *et al*. (2010) conducted a seismic evaluation of a 56-story residential reinforced concrete building based on nonlinear dynamic time history analysis.

Yet, no software can avoid drawbacks, to assure the security, reliability and stability of the structure under earthquake, more than one structural analysis softwares were adopted in this study, namely NosaCAD developed by Tongji University and Perform-3D developed by Computers and Structures Corporation which are both widely applied in nonlinear analyses. Wu *et al.* (2009) conducted elasto-plastic time history analysis of the structure of China Pavilion at the 2010 World EXPO by NosaCAD to study its seismic behavior, Guo, Wu *et al.* (2013) studied the seismic performance of an irregular plane structure by NosaCAD. NosaCAD is developed with ObjectARX, second development tool of AutoCAD, and runs in the AutoCAD environment, accordingly the powerful geometric processing function of AutoCAD can be used to established and edited structure analysis model. Moreover, it can transform nonlinear analysis model to other softwares such as ABAQUS and Perform-3D, which ease the model building process greatly. Wu (2012) introduced the

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Fig. 1 Perspective view of the building

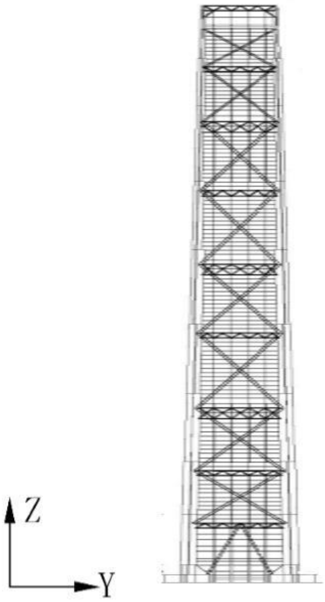
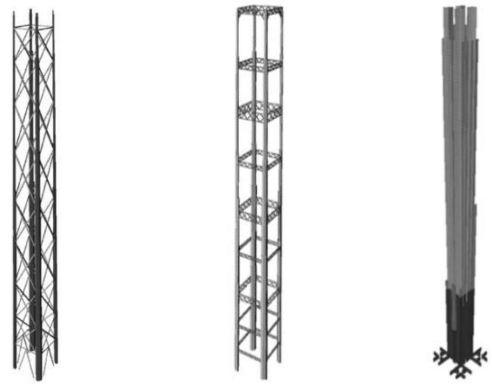


Fig. 2 Elevation of the structure



(a) Mega-frame tube (b) Mega frame (c) Core tube

Fig. 3 Multiple stability structural system

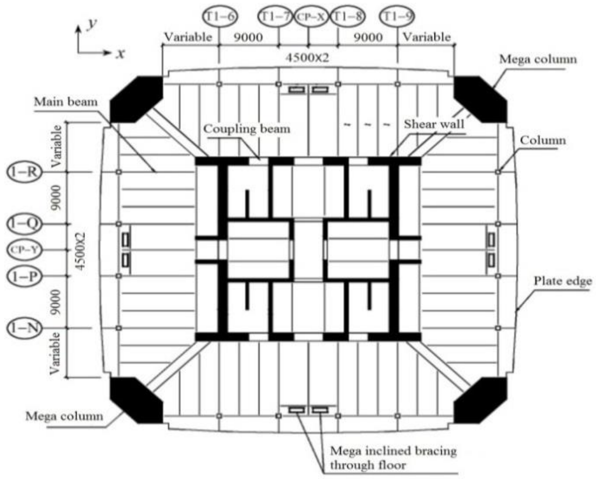


Fig. 4 Typical structural plan layout

transformation of model by NosaCAD at 15th WECC. As the development progressing, more portals to other structural analysis softwares would be carried out in the near future. Elasto-plastic time history analysis method was adopted in this study, by comparing the results from each software, better evaluation can be drawn on the seismic behavior of the structure.

In this paper, elasto-plastic time history analysis was conducted on a super high-rise building named the Tianjin Goldin 117. This building has a height and height-width ratio exceeding the Chinese Codes, which is considered as a code-exceeding building. Thereby detailed investigation is necessary and essential to verify the feasibility of preliminary design and guarantee its safety and also to provide guidance and advice to engineers for similar projects concerned.

Three main parts are included in this paper. First, two refined finite element analysis models of this building were developed by using NosaCAD and Perform-3D, respectively. Then, nonlinear dynamic time history analysis under rare earthquake was conducted on this complex structure. Finally, the nonlinear dynamic responses such as displacement, inter story drift and damage patterns are presented and discussed.

2. Description of the structure

The Tianjin Goldin 117 (Fig. 1), one of the 10 tallest skyscrapers in Mainland China on the way, is located in the Tianjin High-tech Industry Park, with 117 floors over the ground and 4 floors under the ground. The designed height is 597 m, the planned footprint is approximately 830,000 m^2, and the planned total floor-space is approximately 1,830,000 m^2.

With regard to the main structure (Fig. 2), the reinforced concrete core tube, the mega frame consists of mega columns and transfer trusses and mega-frame tube consists of mega columns and inclines mega bracings comprise the multiple stability structural system (Fig. 3). Four mega columns are set at each corner of the building plan, extending to the top of the structure. Nine sets of transfer trusses are distributed evenly at approximately every 12 to 15 floors, resisting the gravity loading from each area and transferring the loads to the corner mega columns. The transfer trusses also create a frame with the corner mega columns, enhancing the torsional stiffness of the structure. Inclined mega bracings are arranged on the four elevations of the tower between two corner mega frame columns, rendering the lateral stiffness for the structure. However, the mega inclined bracings is separated with the perimeter mega frame, providing lateral support for the floor system to restrain out of plane buckling of the mega braces. In additions, the height of main structure is almost 600 m, which is far more than the limitation of the Chinese Codes.

For controlling the axial compression ratio of the shear walls, two sets of 40-50 mm thick steel plates are arranged inside the shear walls of the outer core tube below the 40th floor, the steel ratio of the shear walls is 6.2%-7%, and the reinforcement ratio is 1% over the 44th floor. The thicknesses of shear walls in core tube are shown in Table

Table 1 Thickness of shear walls in core tube

Thickness of shear wall/mm			Thickness of shear wall/mm		
Floor	Outer wall	Inner wall	Floor	Outer wall	Inner wall
B3-L14	1400	600	L59-L74	700	400
L15-L28	1200	500	L75-L85	500	300
L29-L53	1000	500	L86-L94	400	300/250
L54-L58	850	500	L95-Top	300	250

Table 2 Size of cross section in mega inclined bracings

Label of mega inclined bracings	Floor	Size (B×H×tw×tf)/mm
MB1	B1-L7	900×1300×100×50
MB2	L7-L19	900×1800×150×50
MB3	L19-L32	900×1800×150×50
MB4	L32-L48	900×1800×150×50
MB5	L48-L63	900×1800×120×50
MB6	L63-L79	900×1800×120×50
MB7	L79-L94	900×1800×120×50
MB8	L94-L106	900×1000×100×50
MB9	L106-L114	900×1000×100×50

Table 3 Parameter of cross section in mega columns

Label of mega columns	Floor	Area (m2)	Steel ratio (%)	Steel plate thickness (mm)	Reinforcement ratio (%)
MC1	B1-L9	45	6	60	0.8
MC2	L10-L21	45	4	40	0.8
MC3	L22-L35	41	4	40	0.8
MC4	L36-L50	36	4	40	0.8
MC5	L51-L66	27	4	30	0.8
MC6	L67-L81	18	4.5	30	0.8
MC7	L82-L97	11	5	30	0.5
MC8	L98-L108	11	5	30	0.5
MC9	L109-L116	5.4	6	30	—
MC10	L116-117	3	12	50	—

1. The design of mega columns is an external steel plate enclosure with internal inter-connected plates forming separate chambers in accordance with the construction requirements, the parameter of cross section in mega columns is shown Table 3. The six-sided polygonal concrete filled tube composite member has sufficient capacity to resist axial, bending and shear forces generated by seismic excitations. The biggest length and width of mega column sections is 11.2 m and 5.2 m, respectively. To avoid vertical loads, the mega inclined bracings just cross the holes in floors, which coincidently comprises a space frame system with the mega columns. Box section is adopted for mega inclined bracings, and the sizes of cross sections are shown in Table 2. Fig. 4 shows the typical structural plan layout.

According to the Chinese Code for Seismic Design of Building (CCSDB, GB50011-2001) (Ministry of Construction of the People's Republic of China 2001) and Technical Specification for Concrete Structures of Tall Building (TSCSTB, JGJ3-2002) (Ministry of Construction of the People's Republic of China 2002), the main characteristics of this structure, which are beyond the limitation of Chinese code, can be summarized as follows

(1) The height of this building is beyond the specified maximum height of 190 m for SRC frame and RC core tube system.

(2) The structure is super slender, the height of the main structure is 584 m, and the width (extended to the outskirt of the mega columns) of the first floor is 61.24 m, leading the height-width ratio to about 9.5, which is far more than the limitation ruled in the TSCSTB of 7.

(3) To satisfy the architectural profile and structural connection requirements, the plan shape of the mega columns resembles a six-sided polygon with a cross sectional area of about 45 m^2 at the bottom. Neither the size nor the complex construction of the mega columns has broken the records in China, and there lacks experience on building them.

Table 4 Parameter of concrete

Strength grade	Characteristic value f_{ck}/ (N·mm^{-2})	Design value (N·mm^{-2})		Elastic modules E_c/(N·mm^{-2})
		f_c	f_t	
C30	20.1	14.3	1.43	3.00×10^4
C60	38.5	27.5	2.04	3.60×10^4

Table 5 Parameter of rebar

Sort of rebar	Diameter/ mm	Characteristic value f_{yk}/ (N·mm^{-2})	Design value f_y/ (N·mm^{-2})	Elastic modules E_c/(N·mm^{-2})
HPB235	8-12	235	210	2.10×10^5
HRB335	10-32	335	300	2.00×10^5
HRB400	12-32	400	360	2.00×10^5

Table 6 Parameter of profile steel

Profile steel		Design value of yielding strength f_y/(N·mm^{-2})	Design value of shear strength f_v/(N·mm^{-2})	Bearing strength at ends f_{ce}/(N·mm^{-2})
Grade	Thickness or diameter/mm			
Q345	≤16	310	180	400
	>16-15	295	170	
	>35-50	265	155	
	>50-100	250	145	

3. Analysis model

3.1 Parameters of materials

In elasto-plastic analysis, characteristic value was adopted for the strength of materials. In this paper, concrete C30 was adopted for plates, and C60 was adopted for beams, columns and shears walls. The normal rebars were

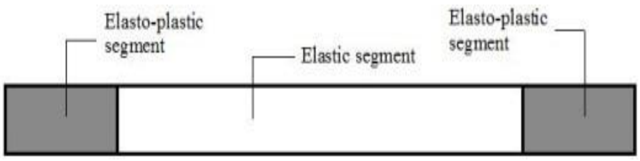


Fig. 5 The frame element composed of three stiffness segments

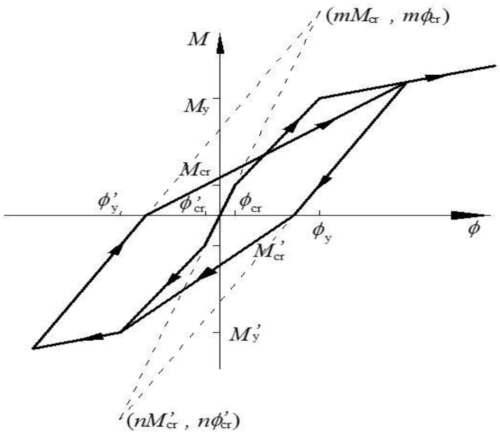


Fig. 6 Trilinear moment-curvature hysteretic model

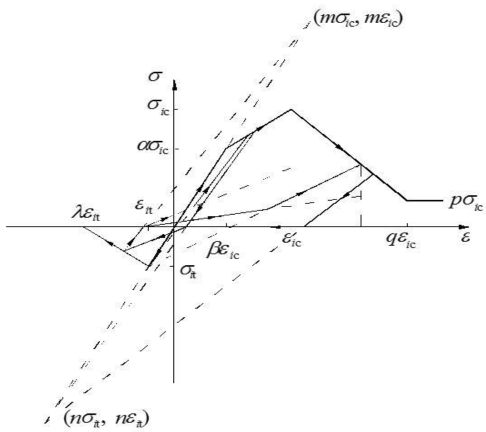


Fig. 7 Concrete constitutive model

employed with HPB235, HPB335, HRB400, and profile steels were employed with Q345B, of which the yield strength is 345 MPa. Parameters of materials are shown in Tables 4-6.

3.2 Finite element model for structure components

3.2.1 Finite element model built with NosaCAD (version 2010)

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 (Fig. 5).

For the frame members mainly suffer bending moment, such as beam, bilinear and trilinear moment-curvature hysteretic model are adopted for the elasto-plastic segments of steel beams, concrete beams and steel reinforced concrete beam, respectively. The trilinear moment-curvature hysteretic curve is shown as Fig. 6. (mM_{cr}, mϕ_{cr}) and (nM'_{cr}, nϕ'_{cr}) are the unloading track guiding point coordinates, M_{cr} and ϕ_{cr} are the cracking bending moment and the corresponding curvature of the section and m and n are 5.0 in this analysis. 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 Fig. 7. σ_{ic} and ε_{ic} are the concrete compressive strength and the corresponding strain. σ_{it} and ε_{it} are the concrete tensile strength and the corresponding strain. (mσ_{ic}, mε_{ic}) and (nσ_{it}, nε_{it}) are the unloading track guiding point coordinates in the tension and compression zone. m and n are both set as 5.0 in this analysis. α and β are set as 0.7 and 0.4, respectively. p and q are set as 0.2 and 3.0, respectively. λ is set as 0.7. The ideal elastic-plastic constitutive model, yield hardening taken into account, is adopted for steel and rebar, and the elastic modules value is of 1 percent of the initiative one. Fiber model is also adopted to simulate the elasto-plastic behavior of trusses.

The relevant parameters of elastic-plastic analysis of the elements, such as cross-section parameters of fiber model and bending moment-curvature curve parameters are generated by NosaCAD according to sectional geometric parameters, material parameters and reinforcement.

The flat shell finite element, which is composed of diaphragm and plate, is used for shear wall and slab. The flat shell finite element model possesses rotational degrees of freedom in diaphragm, 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 (Darwin and Pecknold 1976) together with a biaxial strength envelope (Kupfer and Gerstle 1973), 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 (Fig. 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. It is assumed the smeared cracking is initiated once the tensile strength is reached and the second crack is permitted to appear orthogonal to the first one. This flat shell finite has been used for shear wall analysis under cyclic loading (Wu and Lu, 1996) and prestressed concrete beam analysis (Wu and Lu, 2003), effectively.

3.2.2 Finite element model built with Perform-3D

Same as that in NosaCAD, the moment-curvature hysteretic model are adopted for the frame members, and the fiber model is employed to describe the nonlinear behavior for the frame members. Unlike the elasto-plastic frame member in NosaCAD, the elasto-plastic frame member in Perform-3D can be compounded of elastic segment and elasto-plastic segment in arbitrary formation.

Beam-column element is usually end plastic region

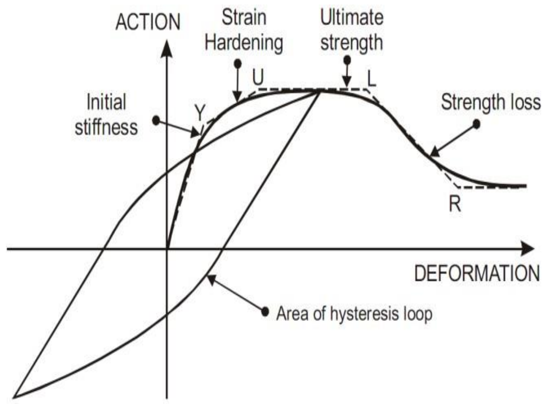


Fig. 8 Main aspects of inelastic behaviour in perform-3D

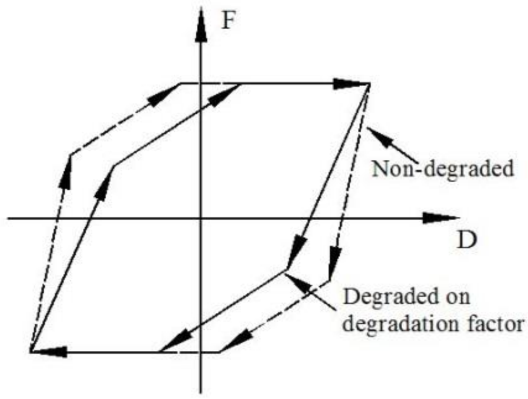


Fig. 9 Hysteretic loop of energy degradation

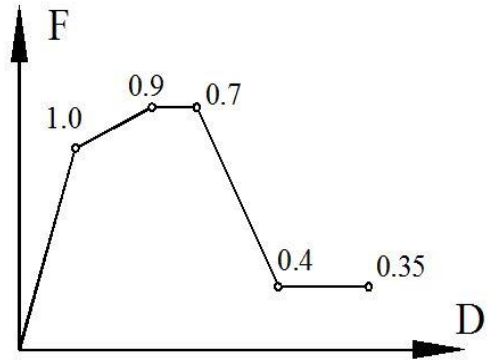


Fig. 10 Degradation coefficients of concrete

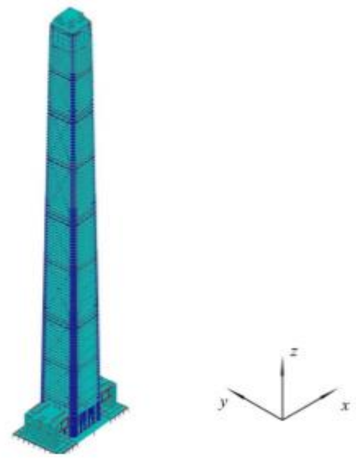


Fig. 11 Model developed with NosaCAD

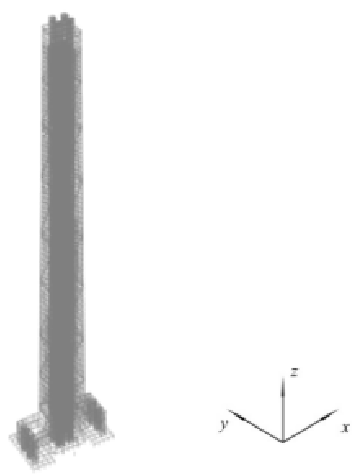


Fig. 12 Model developed with Perform-3D

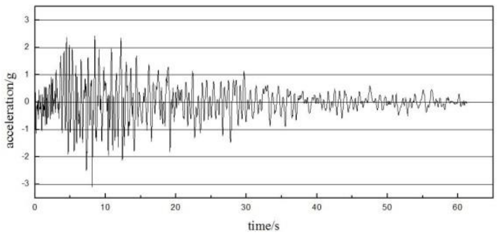
model or multi-segment plastic region model based on different segment assembly. The bilinear and trilinear moment-curvature hysteretic models are adopted for the elasto-plastic segments of steel beams, concrete beams and steel reinforced concrete beam, respectively.

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.

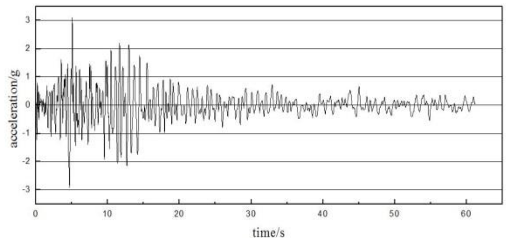
The intent of the Perform-3D action-deformation relationship, with points Y, U, L and R, is to capture the main aspects of the behavior, namely the initial stiffness, strain hardening, ultimate strength and strength loss (Fig. 8). The main intent of the Perform-3D hysteresis loop is to capture the dissipated energy (the area of the loop). This area is affected by stiffness degradation under cyclic loading. By the different parameters set, the certain action-deformation relationship and hysteresis loop are defined for actual inelastic behavior.

Steel constitutive model with bulking or non-bulking are available in Perform-3D. Non-bulking steel model is applied for reinforcement. Concrete constitutive model with Mander stress-strain relationship should be transferred in the action-deformation relationship of Perform-3D which can be determinate by 5 parameters and strength loss is taken into account. The moment-curvature hysteretic relationship for frame element section should also be defined by the action-deformation relationship of Perform-3D, which can be determinate by 3 parameters or 5 parameters.

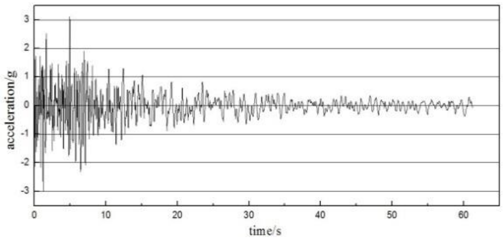
As known that energy can be dissipated by nonlinear component under cyclic loading, and amount of the dissipated energy can be represented by the area of hysteretic loop. In Perform-3D, parameters of energy degradation are determinate by the maximum deformation and can be specified optionally (Fig. 9). Perform-3D gives the required energy degradation through adjusting the



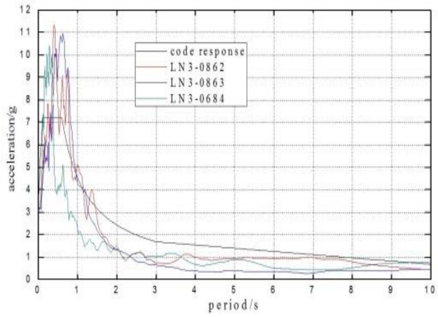
(a) Time history of acceleration in primary direction



(b) Time history of acceleration in secondary direction



(c) Time history of acceleration in vertical



(d) Response spectrums comparing to the design spectrum in CSDB

Fig. 13 LN3 accelerogram

Table 7 Natural vibration characteristic from different software

Period/s	T1	T2	T3	T4	T5
Etabs	8.77	8.72	3.36	2.88	2.83
NosaCAD	9.15	9.09	3.37	2.99	2.91
Perform-3D	8.71	8.63	3.10	2.94	2.89

unloading-load stiffness, and the coefficient of energy degradation is taken as the ratio of the area of degraded and non-degraded hysteretic loop, which can be obtained from experiment and numerical simulation. In this paper, parameters of energy degradation are defined according to the degradation rule of unloading-load stiffness in Mander model, as shown in Fig. 10.

3.3 Analysis model of the structure

The structural model was established by NosaCAD (Fig. 11), then the model was transformed to Perform-3D (Fig. 12) by NosaCAD transforming portal. These two models are both based on the assumptions as follows:

(1) In these models, the basement were not contained, thereby the ground was regarded as the fix end.

(2) The calculated mass of the model was consist of 100% dead load, 100% added dead load and 50% live load. The mass in NosaCAD was 758,000 tons and in Perform-3D is 758,000 tons.

4. Nonlinear time history analysis

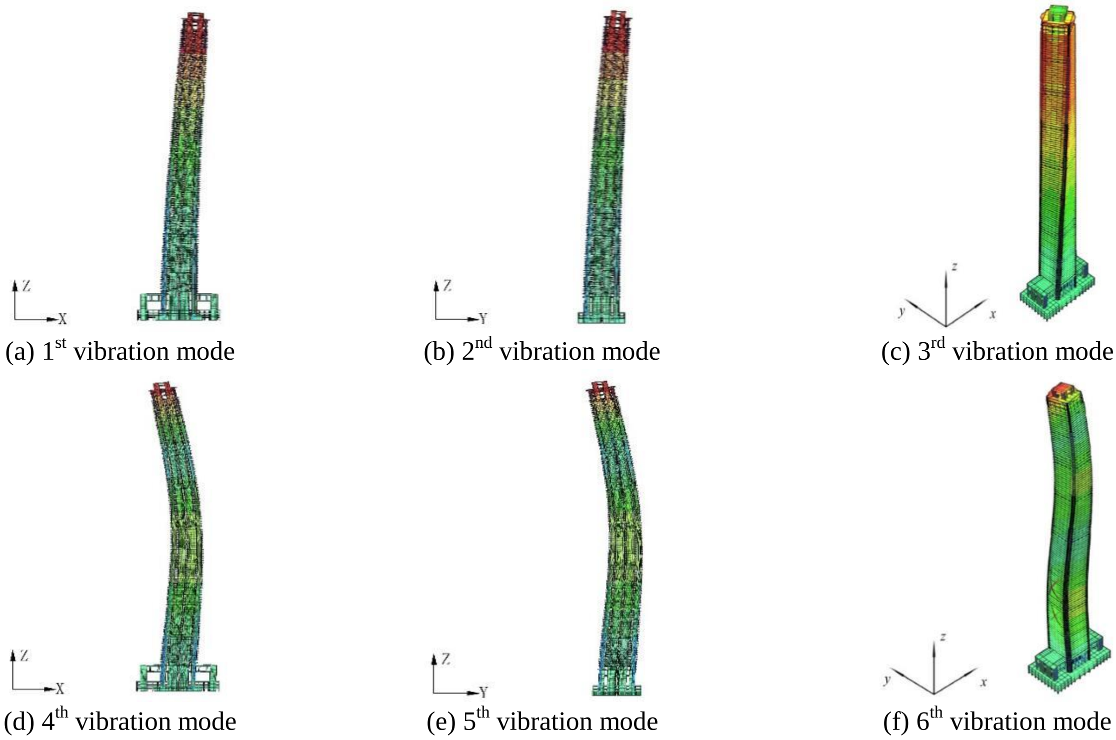
4.1 Input ground motions

According to the CSDB, the site soil in Tianjin belongs to type IV, which is defined that the overlaying thickness of the site is more than 80 m, and the average velocity of shear wave in the soil layer is not more than 140 m/s. 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 IV site soil, three different ground motions were simulated as input accelerations to the model: (a) the LN3 record; (b) the LN4 record; and (c) the LA record, which is formed artificially according to the CSDB. Fig. 13 shows the time history acceleration of LN3 in three directions.

Table 8 Mass participation of first six modes

Direction application	Vibration mode	X direction	Y direction	Z direction
NosaCAD	1st vibration mode	0.4978	0.0000	0.0000
Perform-3D		0.4809	0.0005	0.0000
NosaCAD	2nd vibration mode	0.0000	0.4927	0.0000
Perform-3D		0.0005	0.4944	0.0000
NosaCAD	3rd vibration mode	0.0000	0.0001	0.0000
Perform-3D		0.0000	0.0002	0.0000
NosaCAD	4th vibration mode	0.2272	0.0000	0.0000
Perform-3D		0.2303	0.0000	0.0000
NosaCAD	5th vibration mode	0.0000	0.2198	0.0000
Perform-3D		0.0000	0.2266	0.0000
NosaCAD	6th vibration mode	0.0747	0.0000	0.0000
Perform-3D		0.0883	0.0000	0.0000

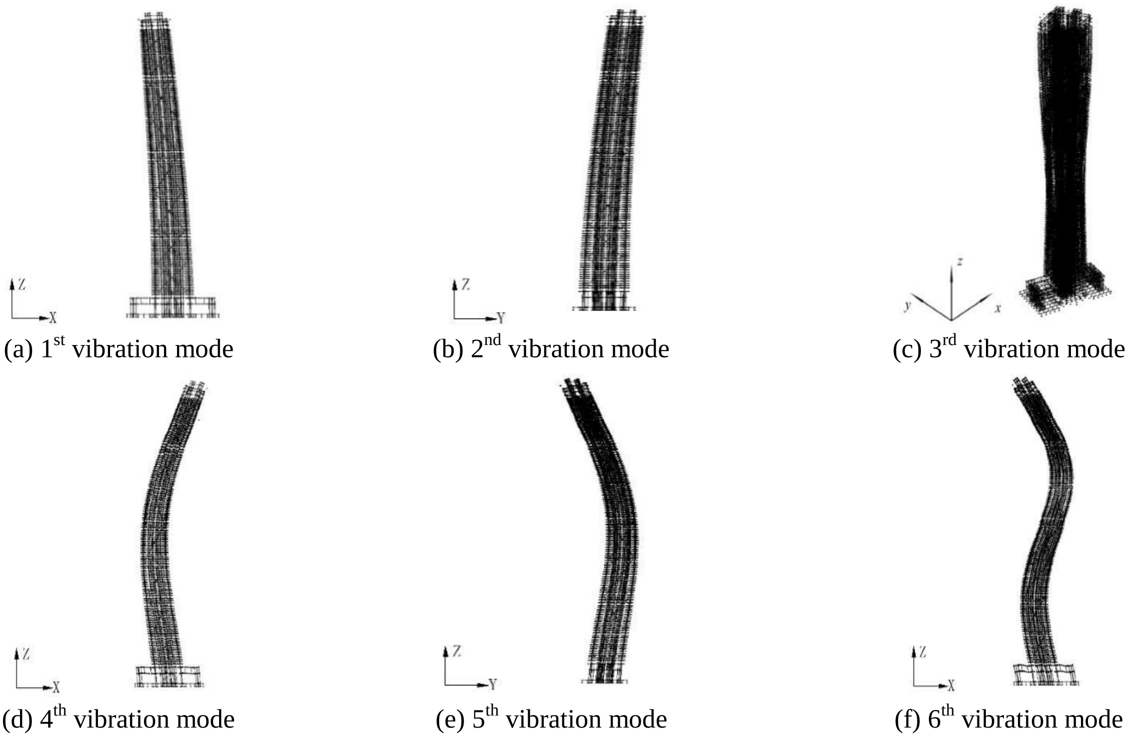
According to the CSDB, buildings in seismic regions should be designed to sustain earthquakes of frequent, moderate and rare levels, which correspond to 63.2, 10% and 2% probability of being exceeded in 50 years, and



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

(d) 4th vibration mode (e) 5th vibration mode (f) 6th vibration mode

Fig. 14 The first six vibration modes from NosaCAD

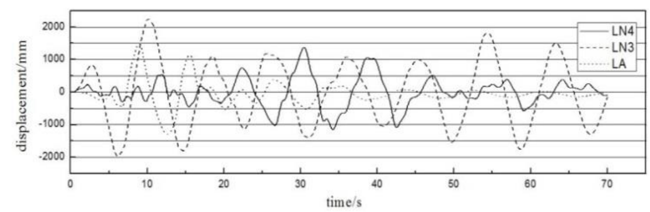


(a) 1st vibration mode (b) 2nd vibration mode (c) 3rd vibration mode

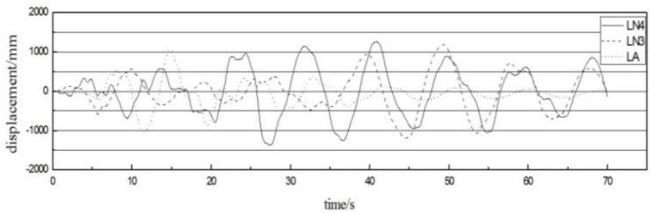
(d) 4th vibration mode (e) 5th vibration mode (f) 6th vibration mode

Fig. 15 The first six vibration modes from Perform-3D

return period of 50, 475 and about 2475 years, respectively. That is to say: when buildings are designed to be subjected to the influence of frequently occurring earthquakes with an intensity of less than the design intensity, the buildings will not be damaged, or will be only slightly damaged and will continue to be serviceable without repair; when they are subjected to the influence of earthquakes equal to the design intensity, they may be damaged but will still be serviceable after ordinary repair or without repair; when they are subjected to the influence of expected rare earthquakes with an intensity higher than the design intensity, they will neither collapse nor suffer damage that would endanger human lives. Tianjin belongs to the seismic zone of intensity 7 (roughly equivalent to a modified Mercalli

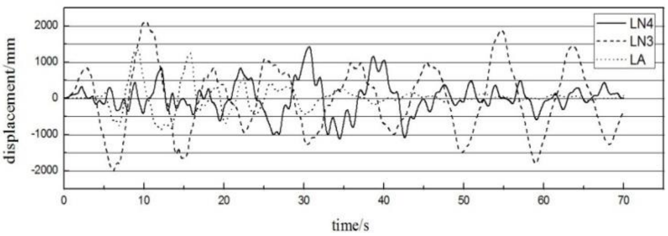


(a) Displacement time history in direction X

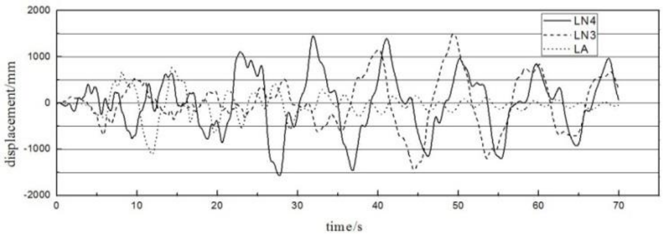


(b) Displacement time history in direction Y

Fig. 16 Displacement time history of node 98909 under rare intensity by NosaCAD



(a) Displacement time history in direction X



(b) Displacement time history in direction Y

Fig. 17 Displacement time history of node 98909 under rare intensity by Perform-3D

Table 9 Maximum displacement responses of node 98909

Input ground motion	Software	Direction X (mm)	Direction Y (mm)
LN3	NosaCAD	2240.75	1182.17
	PERFORM-3D	2083.01	1439.64
LN4	NosaCAD	1371.06	1356.93
	PERFORM-3D	1418.78	1560.07
LA	NosaCAD	1471.75	1042.13
	PERFORM-3D	1312.67	1095.37
Mean value	NosaCAD	1694.52	1193.46
	PERFORM-3D	1604.82	1365.03

Table 10 Maximum envelop value of inter story drift

Input ground motion	Software	Direction X (mm)	Direction Y (mm)
LN3	PERFORM-3D	1/130	1/203
	NosaCAD	1/175	1/317
LN4	PERFORM-3D	1/130	1/137
	NosaCAD	1/201	1/217
LA	PERFORM-3D	1/157	1/186
	NosaCAD	1/186	1/273
Mean value	PERFORM-3D	1/138	1/171
	NosaCAD	1/187	1/263
Maximum value	PERFORM-3D	1/130	1/137
	NosaCAD	1/175	1/217

intensity of 7), the peak ground accelerations (PGAs) corresponding to the earthquakes of frequent, moderate and rare levels are specified to be 0.035, 0.100 and 0.220 g, respectively.

Since seismic Performance under rare earthquake was mainly investigated in this paper and no damage on structure under frequently earthquake must be guaranteed, the peak ground acceleration (PGA) of selected earthquake accelerograms were scaled to 0.035 and 0.220 g, corresponding to earthquakes of frequent and rare levels.

During the analysis, the three earthquake records were inputted in three principal directions simultaneously (direction X, Y and Z) with the PGA ratio of 1:0.85:0.65, and 310 cm/s^2 were adopted for the peak accelerograms.

As specified by TSCSTB, a damping ratio of 0.04 for SRC frame-RC core tube structure system was adopted, and Rayleigh damping was used in integration equation.

4.2 Analysis result

In general, when the structure encounters the earthquake, static load such as gravity and service load has already acted on the structure. Therefore, prior to the nonlinear dynamic time history analysis, a nonlinear static analysis was performed to obtain the initial stress state in structure members, which serves as the initial state of nonlinear dynamic analysis. Meanwhile, a modal analysis was conducted to get the natural vibration characteristics of the structure.

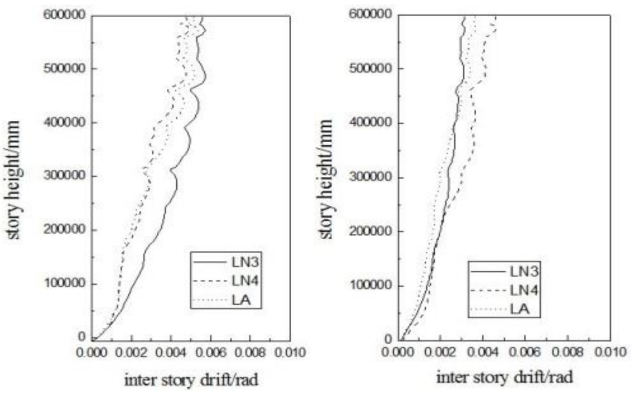
The overall responses of the structure in terms of the dynamic properties and torsion, as well as the failure patterns of the test structure, are discussed as follows.

4.2.1 Natural vibration characteristic of the structure

To validate the accuracy of the models in NosaCAD and Perform-3D, ETABS was chosen in this study as well, hence better comparison could be drawn. The result of modal analysis, from which an initial judgment could be made on the fundamental dynamic characteristic of the structure, is shown on Table 7. The mass participation of first six modes in NosaCAD and Perform-3D is shown on Table 8. In Figs. 14-15, the first three vibration modes from NosaCAD and Perform-3D are given.

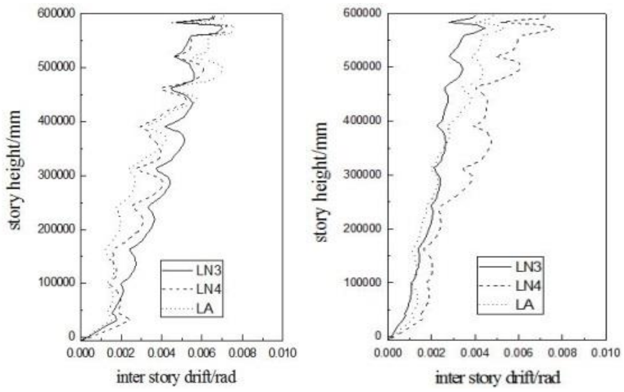
4.2.2 Roof displacement

As shown in the results, the overall stiffness in direction



(a) Direction X (b) Direction Y

Fig. 18 Inter story drift calculated by NosaCAD



(a) Direction X (b) Direction Y

Fig. 19 Inter story drift calculated by Perform-3D

X is smaller than in direction Y, which means when direction X was chosen as the primary direction, the roof displacement response is much more intensive when inputting the same ground motion. Figs. 16-17 show the displacement time history of node 98909 at the roof corner of the shear wall under rare intensity 7 when direction X was chosen as the primary direction by NosaCAD and Perform-3D, respectively. The maximum roof displacement responses under each record calculated by both NosaCAD and Perform-3D are shown on Table 9. It is inferred from the results of roof displacement time history and maximum roof displacement value that there exist certain dissent in the structural responses under different ground motions at the same PGAs (peak ground accelerations), among which the response under LN3 is far severer than the rest. Furthermore, the results from NosaCAD and Perform-3D are basically the same.

4.2.3 Inter story drift

The strings of nodes along the vertical direction at position N1 (Fig. 4) were chosen to study the inter story drift. The maximum deformations were generated by LN3 under rare intensity. The inter story drift envelop curves under different ground motions in direction X and direction Y calculated by NosaCAD and Perform-3D are shown in Fig. 18 and Fig. 19, respectively, comparisons between NosaCAD and Perform-3D are presented as well.

Accounting for certain distinctions between the peak values of inter story drift from NosaCAD and Perform-3D, it is suggested in this paper that the distinction of software assumptions mattered, namely NosaCAD adopts elastic floor while Perform-3D adopts rigid floor. However, for the whole structure, the inter story drift-story height curves have the similar shape, more importantly, the maximum inter story drift calculated by NosaCAD was 1/175 and by Perform-3D was 1/130 (Table 10), which are both smaller than the limited value of 1/100 stipulated by the codes.

Table 11 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
LN3	7.74e4	0.001	5.15e4	0.00067
LN4	8.47e4	0.0011	9.16e5	0.0012
LA	6.85e4	0.00089	5.14e5	0.00067

Table 12 Base shear and the ratio of base shear to weight calculated by Perform-3

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
LN3	4.02e5	0.055	2.86e5	0.039
LN4	4.61e5	0.063	4.75e5	0.065
LA	3.88e5	0.054	2.90e5	0.039

4.2.4 Base shear

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 11 and Table 12, respectively. The results from each software both clearly state that the maximum base shear was generated by LN4 and that the base shear is stronger in direction Y than in direction X when subjected to LN4.

According to the CSDB, design seismic Performance is determined by using seismic coefficient, which is derived from acceleration response spectrum. Seismic shear factor, introduced to guarantee the safety of designed buildings, is defined as the ratio of horizontal seismic shear force to the representative value of gravity load of the structure and which is used to prevent the design seismic Performance from being too small. Usually, there is a minimum value for seismic coefficient in seismic design code. As far as seismic protection intensity 7 is concerned, the factor should be no less than 0.016 for structures with obvious torsion effect or fundamental period of less than 3.5s and 0.012 for structures with fundamental period greater than 5.0s. The seismic shear factors at ground floor of this structure satisfied the requirement of 1.2%.

4.2.5 Damage patterns

Since the severest response was generated by LN3 under rare intensity 7, responses caused by LN3 were taken to illustrate the damage development in the structure at length. Figs. 20-21 show the damage pattern of the structure

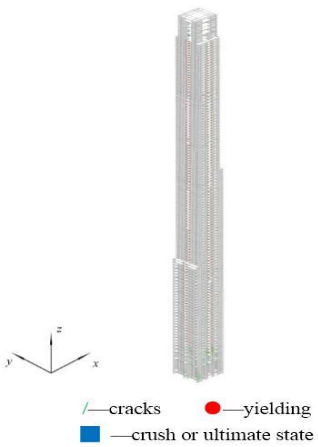


Fig. 20 Damage patterns of core tube under rare intensity 7 LN3 (NosaCAD)

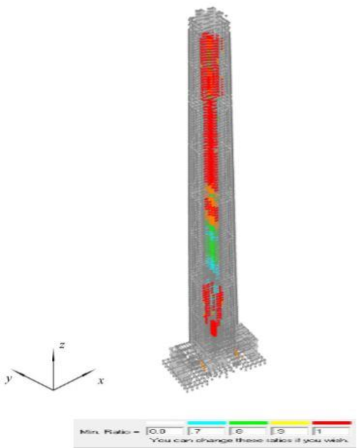


Fig. 21 Damage patterns under rare intensity 7 LN3 (Perform-3D)

analyzed when LN3 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 pieces of different software develops basically to the same extent by the same sequence.

The damage was first observed at the ends of the coupling beams, where plastic hinges initiated, then failure was found in the rebars inside the main beams and the concealed columns of the shear walls. The damage pattern can be concluded as follows: (1) for the core tube, most of the coupling beam progressed plastic hinge at the ends, in term of the lower parts of the structure, some coupling beams reached the ultimate strength, of which the concrete was crashed as well; (2) for the shear walls, only crack were observed and no failure occurred; (3) for the mega columns and mega inclined bracings, no yield failure was found, which means they could work still.

Judged from the sequence of damage development, the structural design meets well the principles “strong column and weak beam” and “strong coupling wall column and weak coupling beam”. The yield failure first found in coupling beams which is regard as the first and chief anti-seismic component dissipated a great deal of input energy of rare earthquake, which accordingly guaranteed the security of shear walls and mega columns against marvelous 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 was carried out by NosaCAD and Perform-3D to study the seismic performance of the Tianjin Goldin 117. According to the analyses, conclusions can be drawn as follows:

(1) The free vibration period and free vibration mode analyzed by ETABS, NosaCAD and Perform-3D are basically equivalent.

(2) Tough different model hypotheses are adopted in these two different pieces of software, for example, to simulate shear wall and slab, NosaCAD adopts the flat shell finite model while Perform-3D adopts the macro layered element, indicators of overall responses obtained from NosaCAD are in good agreement with that from Perform-3D.

(3) Under rare intensity 7, although plastic hinges occurred at the ends of coupling beams, no other vertical force resisting components are damaged besides the cracks in the core tube. Thereby it 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, which meet the requirement stipulated in Chinese Design Codes of “no collapse under rare earthquake”. Beyond that, the maximum inter story drift meet the code requirement of 1/100.

(4) Under rare intensity 7, no mega columns were broken, and certain safety was contained inside, which could assure the security of the whole structure efficiently.

(5) For this special building, preliminary structural design can guarantee the safety under seismic excitation. In addition, multiple stability structural system adopted here can offset the disadvantage of excessive height, which could be introduced to future homologous structure.

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