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RESEARCH SUMMARY ON LONG-SPAN CONNECTED TALL BUILDING STRUCTURE WITH VISCOUS DAMPERS

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SUMMARY

The HUBα-Sightseeing Gate in Guangzhou, which is a connected tall building structure, is composed of two main frame–shear wall towers, which are 86·5 m high, and two steel galleries at top and bottom, respectively. The top gallery is a 120 m long-span steel truss. Vicious dampers are set in the two main towers and the top gallery. It is an ultra-limit structure with complicated structural shape and hybrid structure system. Design idea of performance-based seismic design is presented in this paper. Seismic performance of the structure under different levels of ground motion is analysed and verified by different methods, such as elastic analysis under frequent earthquake, unyielding and elastic check of structure element under medium earthquake and inelastic dynamic time history analysis under rare earthquake. Taking the result of shaking table test as a reference, reasonable enhanced measures of seismic design are suggested. Meanwhile, wind-induced vibration time history analysis under different directions based on wind tunnel test is applied in this paper, and comfortableness problem can be well resolved. The control effects of vicious dampers on internal force, deformation, acceleration and energy dissipation of the structure are also studied. By means of the analysis methods mentioned above, and other analyses such as whole structure stability, temperature effect, floor vibration and so on, relevant measures of ultra-limit structure design are suggested, and the performance objective of the structure can be well realized.

1. STRUCTURE SYSTEM

Zhongzhou Phase α project is located in Xingang East Road, at the crossroad of the Keyun Road. The tour gantry structure is connected to the main structure with tour gallery at the 21st storey, which forms a mega-gantry structure. It would be a landmark building of Guangzhou when it is finished, as shown in Figure 1.

Research summary of ultra-limit structure design of Zhongzhou Phase α is presented in this paper. The tour gantry structure is composed of two main towers, which are 86·5 m high, and two galleries at the top and bottom, respectively. The top gallery, which has a span of 120 m, is in the plane of the two towers, and the bottom one, which has a total span of 90 m (two spans, 36 + 54 m) at the height of 16·5 m, is out the plan mentioned above. Reinforced concrete frame shear wall structure with vicious dampers is adopted in the two towers, and steel truss structure with and without vicious dampers is adopted in the top and bottom galleries, respectively, as shown in Figure 2.

The top gallery is 10·5 m wide, 9·5 m high and has a span–height ratio of 12·6. The bottom gallery is 7·8 m wide, 6·2 m high and has a span–height ratio of 8·7. There are 12 vicious dampers set at the

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Figure 1. Rendering view of Zhongzhou Phase α

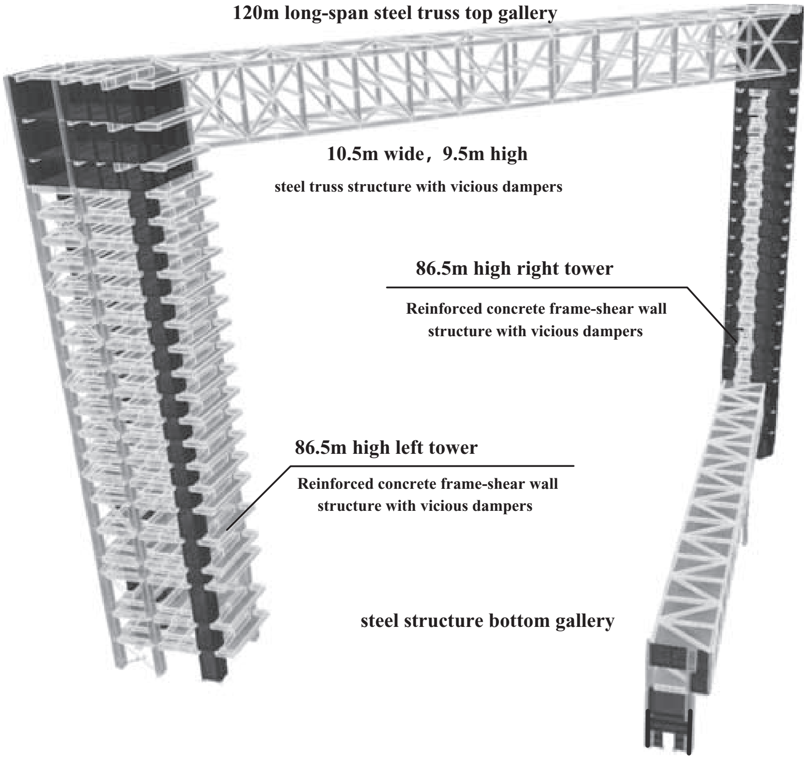

Figure 2. Three-dimensional schematic diagram of the connected tall building structure

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top and bottom surface of the top gallery, respectively. Damper's parameter: damping coefficient $C = 2500\ \mathrm{kN/(m/s)}^{\alpha}$, where damping exponent $\alpha = 0{\cdot}4$. There are 22 vicious dampers set in the two towers, respectively. Damper's parameter: damping coefficient $C = 2000\ \mathrm{kN/(m/s)}^{\alpha}$, where damping exponent $\alpha = 0{\cdot}3$ (Han *et al.,* 2009).

The following structural strengthening measure is proposed after plenty of analyses: (a) steel-reinforced concrete tube structure would be adopted at the location where the top gallery and the two towers are connected, and it would be extended to the next storey below. Equal strength and reliable connection of the steel components and the top gallery are required to make sure that the enormous end moment could be transmitted to the main tower efficiently and evenly. (b) The steel-reinforced concrete structure is adopted in the next two storeys below. These two storeys should be designed according to the code with anti-seismic grade raised by one to make sure the bearing capacity under rare earthquake. The steel pipe of the top steel-reinforced concrete structure should be extended to the storey below the non-tensile region. The tensile stress of the steel pipes under the most adverse condition should have a strict control with no more than 200 MPa, to avoid crack in the wall or column, and meet the needs of normal service condition. (c) Reinforced concrete floor at the end of the top gallery should be strengthened as follows: 200 mm thick, two-way reinforcement with double layer and reinforcement ratio no less than 0·3%. (d) For the main tower that has a regular plane of rectangle, shear wall in 'L' shape should be set at the corner as far as possible. The thickness of the shear wall would be about 600~800 mm to ensure the torsional stiffness of the whole structure.

2. LOAD AND LOAD COMBINATION

2.1 *Gravity load*

To reduce the adverse effect of the top gallery under gravity load on the main tower, lightweight materials are adopted on the floor of the top gallery (two storeys). Additional dead loads of the floor take value of $1{\cdot}5\ \mathrm{kN/m^2}$, and live load of the bottom, middle and ceiling floor takes value of 3·5, 2·5 and $0{\cdot}5\ \mathrm{kN/m^2}$, respectively. The load value of glass curtain wall is taken as $1{\cdot}0\ \mathrm{kN/m^2}$. The tower mainly included an elevator and a staircase, with load value that can be taken according to the Load Code for the Design of Building Structures (GB 50009-2001).

2.2 *Seismic action*

Parameters of seismic action are as follows: design reference period, 50 years; seismic fortification intensity, 7 degrees; anti-seismic classification, C-class; structure safety grade, second grade; design basic acceleration of ground motion, 0·1 g; seismic design classification, first group; site classification, α-class; characteristic period of ground motion, 0·35 s. According to the Code for Seismic Design of Buildings (GB50011-2001) (National Standard of PRC, 2001) (hereafter the seismic code for short) and the Seismic Safety Assessment Report as a reference, (National Standard of PRC, 2002) the parameters of seismic analysis are listed in Table 1.

Based on the results of scale model shaking table test (Zhou *et al*., 2007), the damper ratio of different levels can be taken as 0·035. Two groups of artificial seismic waves and five groups of natural seismic waves are chosen for time history analysis. In trial calculation, 20 groups of two-way natural seismic waves are adopted in elastic model, which is modelled in ETABS. Based on comparison of base shear result from time history analysis and response spectrum analysis, these seismic waves which meet the needs of the seismic code are chosen. The needs are as follows: base shear result form time history analysis of single wave should be no less than 65% of the result from the response spectrum analysis, and the mean value of the base shear results from the time history analysis of the chosen

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Table 1. Parameters of seismic analysis

Seismic intensity	Exceeding probability based on design reference period of 50 years (%)	Earthquake affecting coefficient	Peak acceleration (gal)
Frequent earthquake	63	0·08	35
Medium earthquake	10	0·23	100
Rare earthquake	2	0·50	220

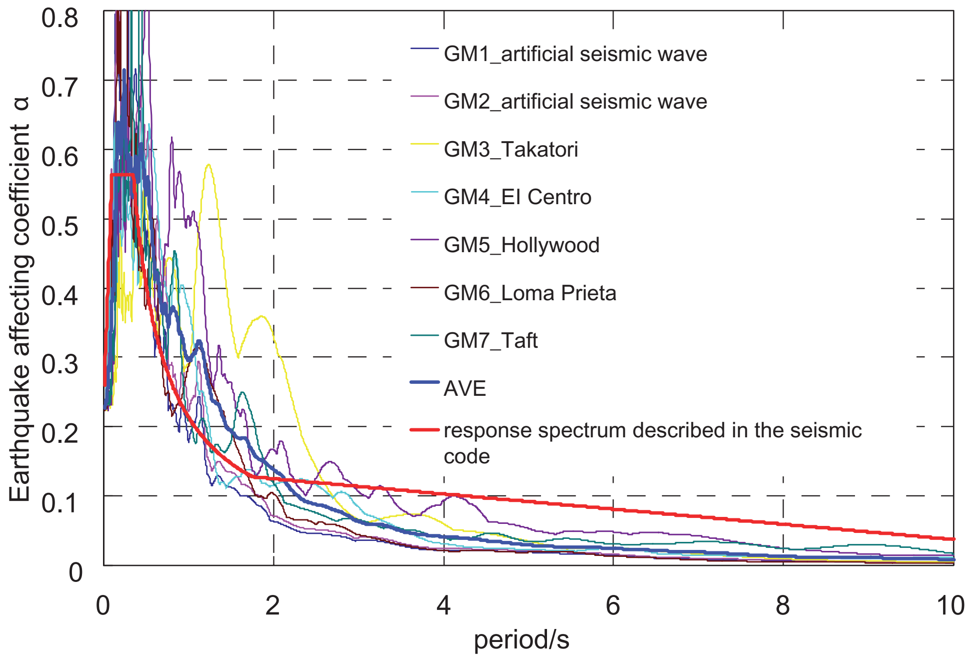

Figure 3. Comparison of response spectrum from the seismic code and seismic waves

waves should be no less than 80% of the result from the response spectrum analysis. Response spectrum of dominant wave from each chosen group and response spectrum described in the seismic code are shown in Figure 3, from which we can see that near the characteristic period of ground motion ($T_g = 0{\cdot}35$ s) and the structure nature vibration period ($T_1 = 2{\cdot}46$ s), the mean value of response spectra gained from seismic waves marched well with the value of the response spectrum described in the seismic. Because of the complex structural shape, vertical seismic action must be taken into consideration for the long-span top gallery. Three-dimensional seismic wave input is applied in the analysis. In order to have a better inspection of structure seismic performance, four directions of horizontal input are taken into account, which are as follows: 0° (*X*-direction), 90° (*Y*-direction), 45° and 135°.

2.3 *Wind load*

Values of basic wind pressure in Guangzhou: return period of 100 years for bearing capacity analysis, ω_0 = 0·60 kN/m^2; return period of 50 years for stiffness analysis, ω_0 = 0·50 kN/m^2; return period of

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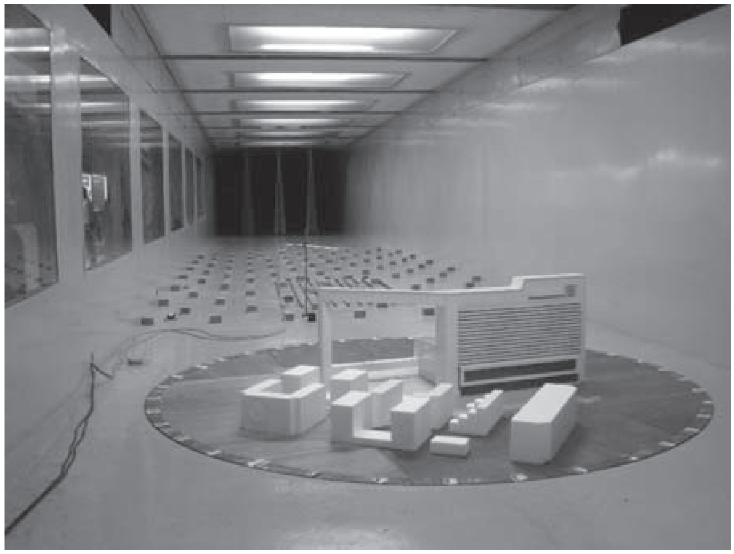

Figure 4. Wind tunnel test model

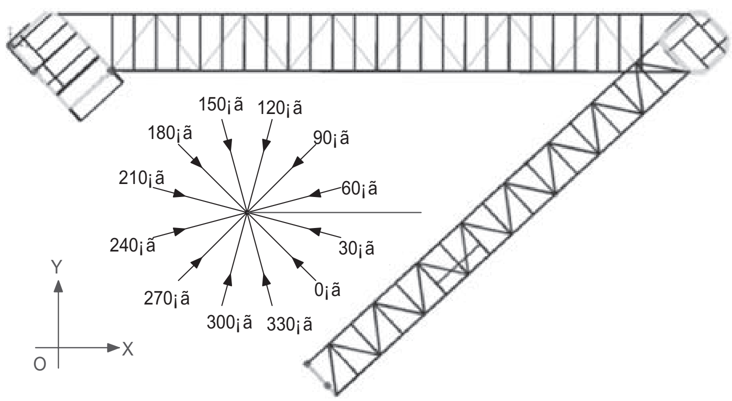

Figure 5. Directions set

10 years for comfort analysis under normal service condition, $\omega_0 = 0{\cdot}30\ \mathrm{kN/m^2}$. Ground roughness, C-class.

Due to the complex shape, structure is very sensitive to wind load, so a rigid model wind tunnel test is proposed (Xie and Shi, 2008). To ensure the safety and economical rationality of the structure under wind load, wind vibration time history analysis based on fluctuating wind pressure time history gained from wind tunnel test is proposed in this paper. In the wind tunnel test, except for the connected structure, the main structure built earlier is also included, as shown in Figure 4. The geometry scale ratio is 1/30. Ground roughness of B-class is modelled for boundary layer flow field in the test, and the ground roughness coefficient is 0·16. Because of the irregularity of the structure, wind direction, and *X*- and *Y*-directions are set as shown in Figure 5.

2.4 Temperature action

Inside and outside temperature difference effect of local structure members can be reduced by measures such as thermal insulation. Temperature difference effect of the whole structure is considered in this paper, that is, the influence of the top gallery under different temperature on the towers is mainly included. The difference in temperature of the whole structure can be valued as ± 30°C.

2.5 Load combination

All structure elements should be designed under the most unfavorable load combination. Partial coefficient for the loads, internal force magnification and adjustment coefficient of elements and structure should be taken into account according to current codes, and partial coefficiant for the loads can be taken from code. Under medium earthquake action, structure elements should be designed under elastic or unyielding limit. Under rare earthquake action, as to the important part of the structure such as shear walls and columns of top strengthened region and the top gallery, their shear and flexural capacity should be designed under unyielding limit; as to those shear walls and columns which do not belong to the strengthened region, their shear capacity should be designed under unyielding limit; flexural capacity of these elements and shear and flexural capacity of frame beam can be designed in yielding state, but they should have a deformation check according to ASCE-α (American Society of Civil Engineers, 2007) and deformation control.

3. STRUCTURE ANALYSIS

The key points in the design process are as follows: (a) the structural performance under seismic and wind action in different directions; (b) the overall stability of the structure; (c) comfort evaluation of the top structure (the top gallery included) under wind-induced vibration and human normal activities; and (d) connection between the main tower and the top gallery, construction scheme of the top gallery and its influence on the main towers (temperature influence included).

According to the ultra-limit conditions and irreqularity of the structure mentioned earlier, performance-based method is applied in the structure analysis and design. Take relevant regulations as a reference, according to performance level of element ductility (non-ductility); performance level of structure can be described in four levels as follows: (a) operational (OP); (b) immediate occupancy (IO); (c) life safety (LS); and (d) collapse prevention (CP).

The seismic performance objectives of different elements under three seismic levels are defined according to the importance of each part of the structure, as shown in Table 2.

Table 2. Seismic performance objectives of different elements

Region	Elements		Frequent earthquake	Medium earthquake	Rare earthquake
Top and bottom gallery	Steel truss		OP	OP	IO
	Steel truss		OP	OP	IO
Top strengthened region	Shear wall		OP	OP	IO
	Frame column		OP	OP	IO
Non-strengthened region	Shear wall	Shear	OP	OP	IO
		bending	OP	IO	LS
	Frame column	Shear	OP	OP	IO
		bending	OP	IO	LS
	Frame beam	Shear	OP	IO	LS
		bending	OP	LS	CP
—	Vicious damper		Safe work condition		

Table 3. Contrastive analysis of each model

Software	Total mass (ton)	Period (s)			
		T_1	T_2	T_3	T_4
ETABS	20 531	2·465	1·877	1·672	0·885
SATWE	19 900	2·638	1·924	1·767	1·177
PERFORM-3D	20 531	2·409	1·776	1·589	0·959

Table 4. Main performance under frequent earthquake

Direction (°)	0		90		45		135	
Towers	Left tower	Right tower	Left tower	Right tower	Left tower	Right tower	Left tower	Right tower
Drift (rad)	1/1508	1/2762	1/1287	1/3030	1/1470	1/3086	1/1386	1/2985
Top displacement(mm)	38·9	23·7	45·7	20·3	39·8	21·1	42·5	21·8
Base shear (kN)	3 149		2 719		2 827		2 906	
Base moment (kN·m)	379 536		368 448		364 015		373 447	

Base shear and base moment are a summation of results of two towers.

Three structural analysis softwares (SAP2000, ETABS, PERFORM-3D) have been used for a comprehensive analysis. To ensure the reliability of models, results of modal analysis and under gravity load from each model are contrasted, as shown in Table 3.

3.1 *Response spectrum analysis*

The structure is in elastic state under frequent and medium earthquake according to bearing capacity check, so elastic model in both ETABS and SATWE is adopted in response to spectrum analysis. Taking no account of vicious damper activity, the main results gained under frequent earthquake are listed in Table 4.

The analysis results show that the maximum axial compression ratios of shear wall and frame column are 0·59 and 0·72, respectively, and the maximum stress ratio of steel element is 0·48. There is no over-bar information, and no shear wall or column subjected to tension in the first floor. Each performance level under frequent earthquake for the whole structure and elements is satisfied. As to medium earthquake, the maximum axial compression ratios of shear wall and frame column are 0·76 and 0·85, respectively. Shear walls and columns of top strengthened region are still in elastic state, and the others' shear capacity is in elastic state. Parts of shear walls which are in unyielding state have bending over-bar information. The maximum stress ratio of steel element is 0·58, and there are parts of frame beams which have over-bar information and no shear wall or column subjected to tension in the first floor. Each performance level under medium earthquake for the whole structure and elements is satisfied.

3.2 *Overall stability analysis*

OpenSEES, a research program used in seismic analysis developed by the Pacific Earthquake Engineering Research Center (PEER), is adopted in geometric nonlinear analysis of the structure. Overall

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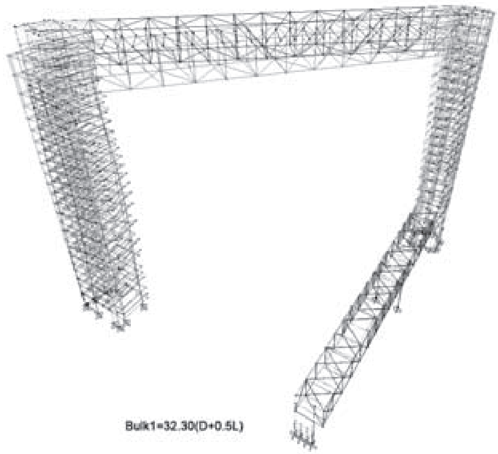
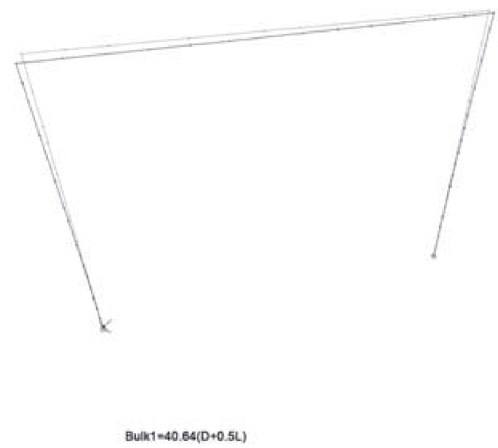


(a)



(b)

Figure 6. Linear bulking modal of the (a) origin model and (b) simplified model

stability analysis could be a complex process, so the analysis procedure is adopted as follows: (a) a simplified gate-type model in SAP2000 according to the equivalent principle of stiffness, mass, geometry and restrains. Modify the model to make the mechanical properties match well with the origin model, so the stability characteristic of the origin model can be reflected well by the simplified model. (b) Geometric nonlinear analyses under different conditions are applied by OpenSEES on the simplified model, to make sure the overall stability of the structure is satisfied. The linear buckling modal results of the origin and simplified model are shown in Figure 6. Conditions and results of geometric nonlinear analysis are shown in Table 5.

Constant load is initial load, and increment load is the second applied load, which is increasing until bulking occurred. Load coefficient–top displacement curve is shown in Figure 7. The overall stability is decided by out-plane stability of the structure, and the stability coefficient can be taken as 32·2, so the overall stability can be satisfied.

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Table 5. Geometric nonlinear analysis

Condition	Constant load	Increment load	Bulking load coefficient (stability coefficient)
1	0	$D + 0{\cdot}5L$	32·27
2	0	$D + 0{\cdot}5L + W_Y$	32·16
3	W_Y	$D + 0{\cdot}5L$	32·18
4	0	$D + 0{\cdot}5L + W_X$	32·23
5	W_X	$D + 0{\cdot}5L$	32·26

D, dead load; L, live load; W_X, wind load at direction X; W_Y, wind load at direction Y.

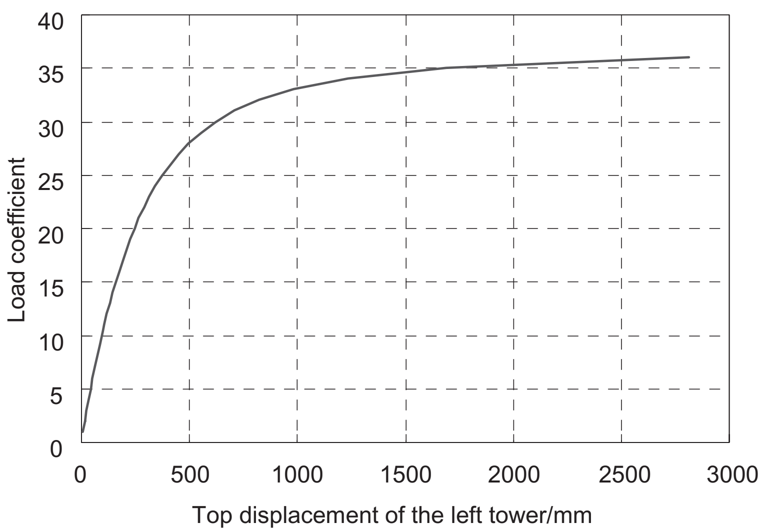

Figure 7. Load coefficient–top displacement curve of the left tower

3.3 Push-over analysis

PERFORM-3D, an inelastic analysis program based on fibre model, is adopted in push-over analysis for each tower. Take no consideration of vicious dampers, and the top and bottom gallery (vertical loads of the galleries are still preserved) in the push-over analysis of single tower. Inverse triangle load distribution is applied in the analysis, and the curves of top displacement–base shear are shown in Figure 8.

It is a fact that the structure is mainly composed of two towers and a long-span truss at the top; the analysis of single tower could be a reference. As to the left tower, the base shear result of push-over analysis is 12 700 and 10 301 kN for elastic analysis under rare earthquake; as to the right tower, the base shear result of push-over analysis is about 20 000 and 9801 kN for elastic analysis under rare earthquake. Based on comparison of the base shear results mentioned above, it can be concluded in macro-level that the two towers, which both have a sufficient lateral bearing capacity and structural lateral demand under rare earthquake, can be satisfied.

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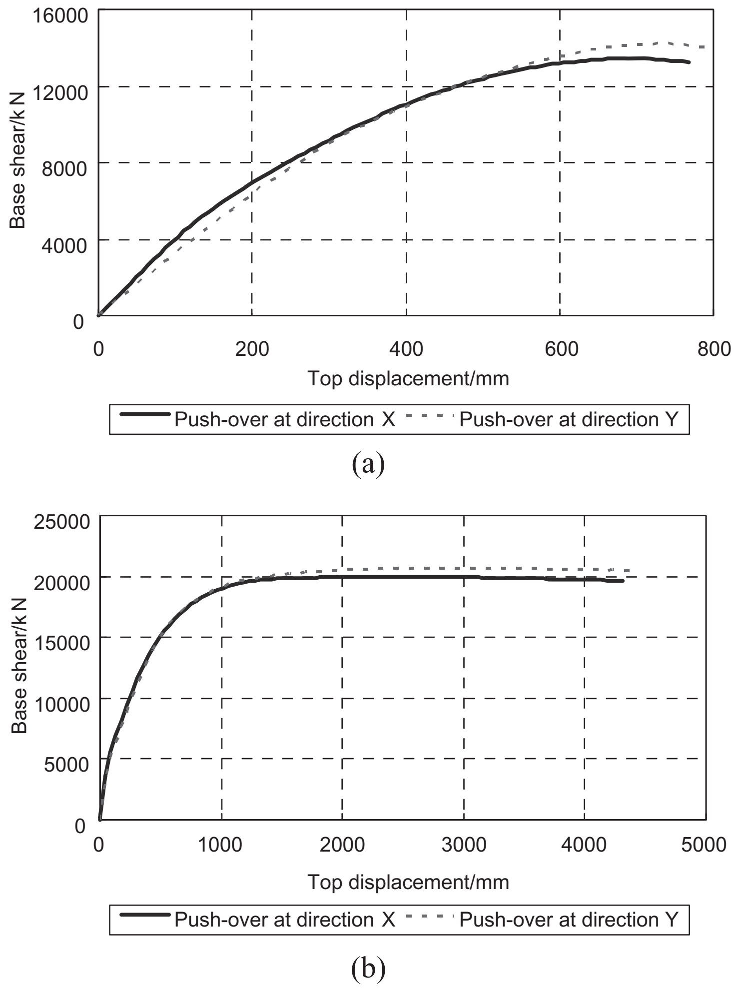

Figure 8. Top displacement–base shear curve: (a) the left tower and (b) the right tower

3.4 Wind-induced vibration time history analysis

Wind-induced vibration time history analysis based on wind pressure time history measured in wind tunnel test is presented in this paper, and the basic procedures are as follows (Chen *et al*., 2009): (a) elastic model, which has linear elastic materials and a small deformation, is modelled in finite element analysis program. Vicious damper set in the structure is taken into account, and its nonlinear behaviour is defined. (b) Determination of wind load time history data. Wind pressure time history data of measuring points could be transformed into wind load time history data of structure node in combination with its tributary area. Wind-induced vibration time history analysis can be realized after wind load time history data imported into the analysis program. (c) Determination of parameters in analysis. Direct integration method or fast nonlinear analysis method can be applied, and time step and cumulative time are also determined. (d) Time history analysis and statistical analysis of results. Structure evaluation of wind-induced vibration can be realized based on storey drifts, storey shear, energy dissipation, vibrating acceleration and so on.

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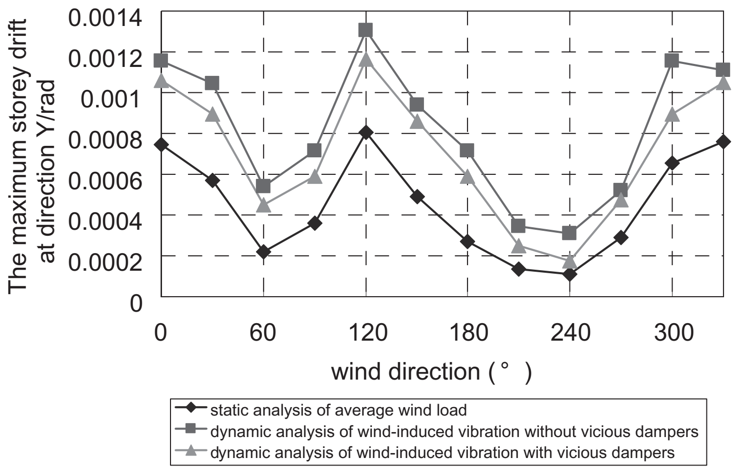

Figure 9. Comparison of maximum storey drifts under different wind directions

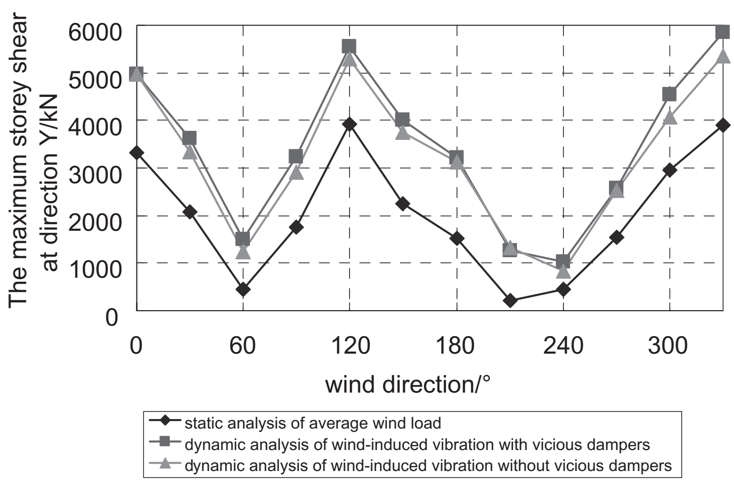

Figure 10. Comparison of maximum storey shears under different wind directions

Viscous dampers are mainly used to control vibration induced by fluctuating wind. There are three work conditions analysed in this paper: static analysis of average wind load, and dynamic analysis of wind-induced vibration with and without vicious dampers. Analysis results show that the maximum storey drift appears under wind direction 120°, as shown in Figure 9. The maximum storey shear appears under wind direction 330°, along direction *Y*, as shown in Figure 10. Due to vicious dampers, both storey drifts and storey shears can be decreased. Energy dissipated by vicious dampers increased smoothly during the vibration, as shown in Figure 11. The ratio of energy dissipated by vicious dampers and total energy imported is about 70%.

The maximum acceleration of the top gallery appears at the mid-span, and the horizontal maximum acceleration is much higher than the vertical result. Vicious dampers are mainly set in direction *Y* of the structure, and acceleration in direction *Y* can be well controlled, a decrease of about 90%. In

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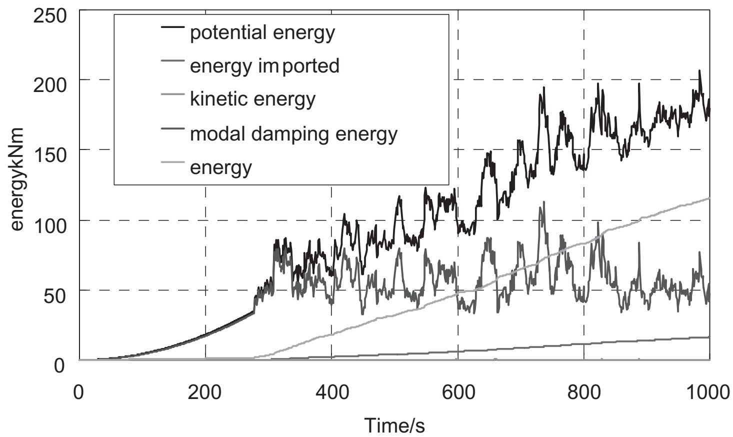

Figure 11. Structural energy dispassion curve of dynamic wind load under direction 0°

direction *X*, structural acceleration response can also be decreased by 50%. Comfortableness of wind-induced vibration is satisfied.

3.5 Dynamic inelastic analysis

Influence of structural ultra-limits on structure seismic performance is generally studied by dynamic inelastic analysis. Seismic response under rare earthquake of the whole structure and the top gallery, and key elements such as vertical elements at the bottom and elements of top strengthened region, coupling beams, etc. were studied. Part of the structure has gotten into inelastic state, and PERFORM-3D, which is an inelastic analysis program based on fibre model, is adopted.

A visualized modelling pre-processing program of PERFORM-3D named ETP V1.1 (Han *et al.*, 2010) by which reinforcement of structure members can be easily imported is adopted. With this program, information of geometric, load, node, section, mass and restraint of the model, which is modelled with much more convenience in ETABS can be imported to PERFORM-3D, by which modelling nonlinear analysis model in PERFORM-3D can be improved both in efficiency and accuracy. Analysis of elastic model in ETABS under the same condition is carried out to have a judgement of reliability and comparison of inelastic deformation.

In the condition numbered GM2Y, as shown in Figure 12(a), due to vicious dampers, the top displacement of the left tower in inelastic analysis is close to the result in elastic analysis, which indicates that the structure does not have an obvious inelastic damage. The top displacement results of the left tower under condition GM3Y are shown in Figure 12(b). It can be seen that during the first 6 s, the results of two models are close, which indicate that the structure is still in elastic state. After 6 s, the top displacement results of two models are separated, which indicate that the structure has an obvious inelastic damage. It can be known from Figure 12(b) that the distance between peak value is increasing with the passage of time, which shows that the structural natural period becomes longer because of stiffness degradation due to structural inelastic damage.

The structural general responses under rare earthquake are listed in Table 6. The maximum storey drift is 1/398 in the model with dampers, and the seismic code requirement can be satisfied. Based on comparison of the results gained with and without dampers, it can be concluded that setting of dampers has an obvious improvement on the structural seismic performance.

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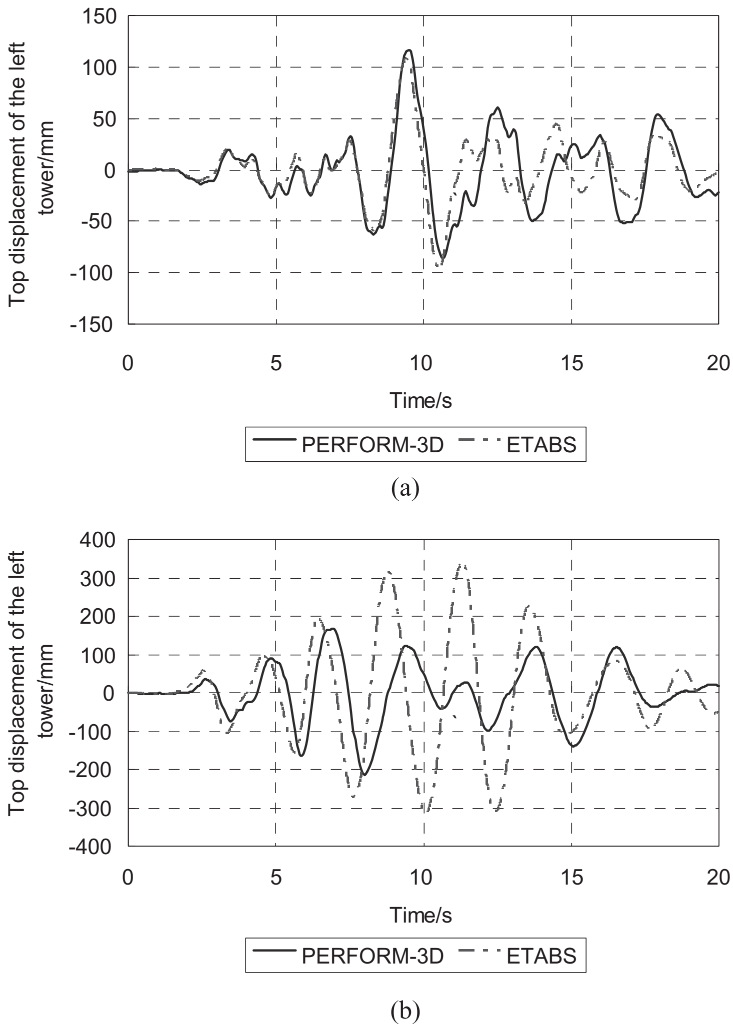

Figure 12. The left tower's top displacement time history of elastic and inelastic analysis: (a) GM2Y and (b) GM3Y

Table 6. General results under rare earthquake

Direction	*Y*	
With or without dampers	Yes	No
The maximum storey drift (rad)	1/398	1/285
The maximum top displacement (mm)	147·92	175·01
Base shear (kN)	13 504	17 815
Base moment (kN·m)	701 967	811 950

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Based on the maximum internal force of elements recorded, expected performance levels of different structure elements, such as bending and shear capacity of shear walls, columns and beams within and without the top strengthened region, are verified.

To those elements which could be yielded, a deformation check is carried out. Take condition GM3Y for an example, deformation performance of shear walls is shown in Figure 13(a), and from the results it can be known that the bending weak parts of shear walls are mainly focused at the bottom of the tower. Shear walls at the ground floor are in LS level, and the weakest one of them with L shape has already reached CP level. Bending deformation performance of shear walls are in state of limit-yielded generally.

Frame columns are in OP state; steel braces connected with towers are in LS state; steel columns and braces in the bottom gallery are in IO state, as shown in Figure 13(b). The weak part of frame beams is focused at the lower part of the left tower, the 3rd~12th storey, as shown in Figure 13(c). The maximum bending deformation of frame beams is close to limit value of LS state, and bending deformation performance of frame beams is in limit-failure state. It can be concluded that deformation performance of these yielded elements met the needs of performance objectives under rare earthquake generally.

It can be seen from analysis results that structure seismic responses of the top strengthened region and the storey below are within performance level set before. Damage degree of key parts of connection, strengthened floors at connection of the top gallery and the towers and shear walls in tension of the towers included are within acceptable level. There is no element over-stressing or instability over-stressing of the top steel truss, and demands of stability and internal force of unyielded condition under rare earthquake are satisfied.

Energy dissipation of the whole structure (take condition GM3Y, for an example) is shown in Figure 14.

It can be seen from Figure 14 that, as to take no consideration of dampers in the analysis, the ratio of dissipated inelastic energy to the whole energy dissipation is 8·5%; as to take consideration of dampers, the ratio of dissipated inelastic energy to the whole energy dissipation is decreased to 3·0%, and the ratio of energy dissipation by dampers to the whole energy dissipation is 23%, by which vibration control effect of vicious dampers can be reflected.

3.6 Temperature difference analysis

The maximum axial force result of elements of the top steel truss induced by temperature difference is 754 kN, which accounts for 8·5% of the results under condition $1{\cdot}0D + 1{\cdot}0L$, and it has no obvious influence on the truss elements. The force of the top steel truss to the tower is 870 kN, which has no control effect of the main tower.

3.7 Floor vibration and stress analysis

The top gallery has a long-span of 120 m, and its transformed distributed load can be 183 kN/m ($D + L$). Due to its long-span and light weight, comfort problem of vertical vibration under human normal activities would be caused. Floor vibration could be caused by people's walking, dancing, sport movement or operation of machine and vehicle equipment. There is little research and no relevant design guideline in our country. The method suggested in reference is adopted in this paper (Applied Technology Council, 1999). The results show that the maximum vertical acceleration of floor vibration is less than 0·015 g, by which comfort problem of floor vibration can be satisfied.

Mode superposition response spectrum method (static and take no consideration of dampers), which is partial to safety, is adopted to calculate stress of floors at the connection of the top gallery and the

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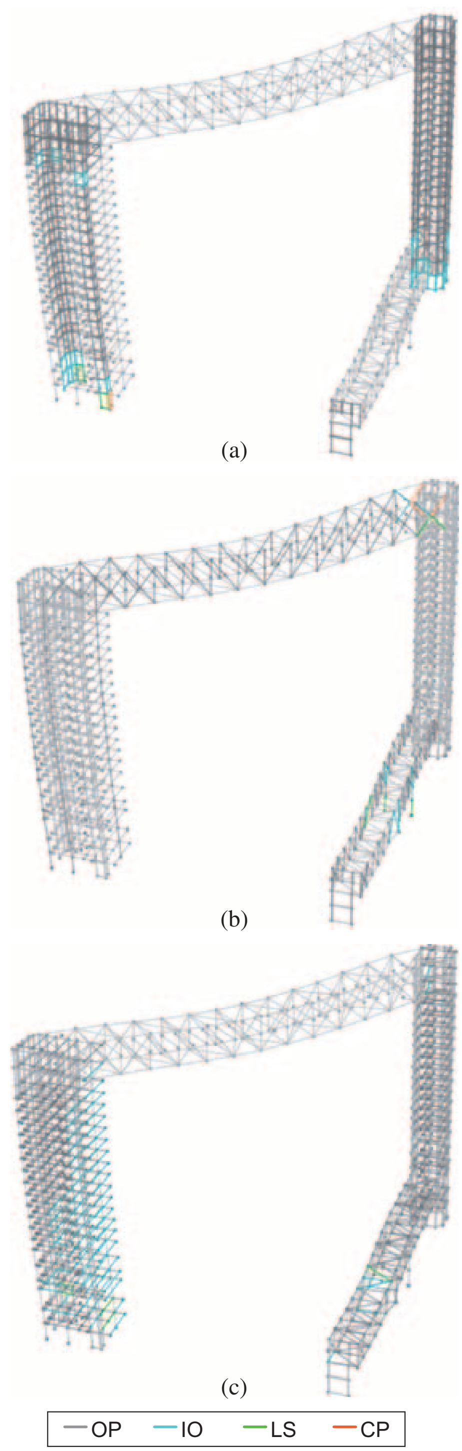

Figure 13. Performance of elements under condition GM3Y: (a) shear walls, (b) frame columns and steel brace and (c) frame beams and steel beams

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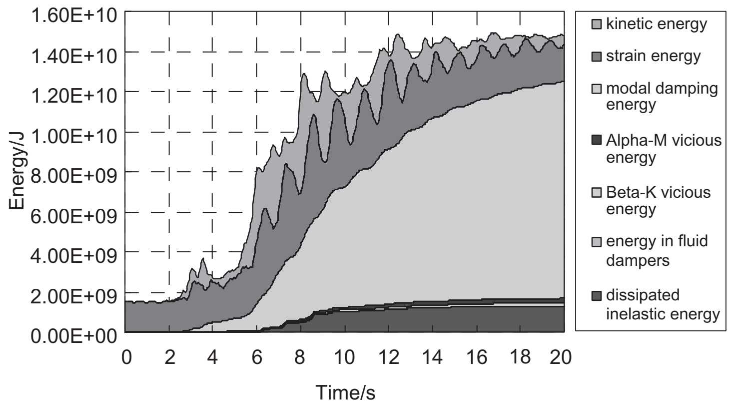
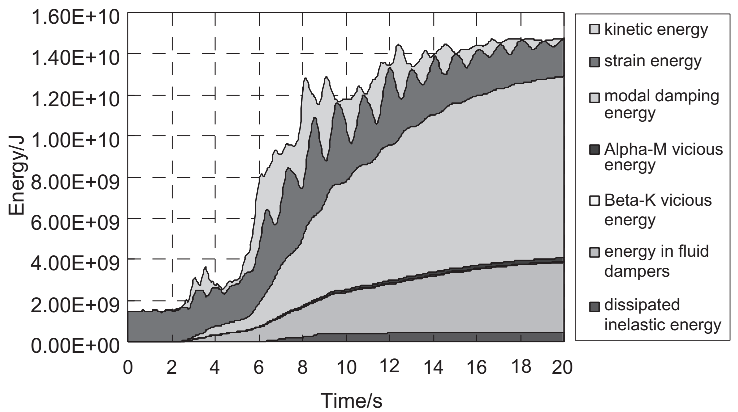


(a)



(b)

Figure 14. Energy dissipation: (a) take no consideration of dampers, and (b) take consideration of dampers

main towers. Summation of the floor's shear force can be gained by section cutting of the most favourable position in the model, and it would be checked by an equation (E.12) suggested in appendix E of the seismic code.

3.8 *Deflection analysis under normal service condition*

The top gallery is 120 m long, 10·5 m wide, 9·5 m high and has a span–height ratio of 12·6. Its transformed distributed load is 183 kN/m ($D + L$). The deflection of the top gallery under this standard combination is 1/870, taking no consideration of construction procedure.

During the lifting process of the top gallery, it can be taken as simply supported and its deflection is 1/760. Therefore, before the lifting, an inverted arch of 1/760 is set for the top gallery, and then it can be considered that there is no deflection during the lifting. When the lifting is finished, deflection of the top gallery which has taken construction into account can be decreased to 1/1290, a result of additional dead load and other loads. This meets the needs of normal service condition.

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4. CONCLUSION

Long-span connected tall building structure adopted in the Zhongzhou Phase α project is a new and special structure system. It is more complicated in analysis and design than normal structures. Based on the research of this project, conclusions could be gained as follows:

(1) A simplified method is adopted in structural nonlinear analysis. Buckling load coefficient of the structure can be gained, and the overall stability of the structure can be verified effectively.
(2) Based on elastic analysis results of the structure under frequent and medium earthquake, it can be seen that limit values required in the codes are satisfied such as storey drift, top displacement and so on. Elements of the structure have a sufficient bearing capacity, and the steel elements have no buckling. The structure meets the needs of performance objectives through the elastic and unyielded analysis under medium earthquake.
(3) Based on the base shear results of push-over analysis of single tower and elastic analysis under rare earthquake, lateral performance of the main towers can be proven.
(4) From comparison of elastic and inelastic analyses results, it can be known that the results are reliable, and structural nonlinear damage can be reduced by vicious dampers set in the structure. In combination with the results of deformation and inter-force, satisfaction of deformation performance objectives of the structure and its elements can be proven. The seismic performance of the structure has a slight improvement than the requirement in the seismic load: 'no damage under frequent earthquake, repairable under medium earthquake, no collapsing under rare earthquake'.
(5) Based on comparison of analysis results with and without dampers, the obvious improvement in seismic performance of vicious dampers on the structure is proven.
(6) By wind-induced vibration time history analysis based on wind pressure time history measured in wind tunnel test, demands of bearing capacity under return period of 100 years, structural stiffness under return period of 50 years and comfortableness under return period of 10 years are proven to be satisfied. Dampers set in the structure have a nice control of acceleration. Because of the little ratio of fluctuating wind to total wind action, deformation and internal force cannot be obviously reduced.
(7) Temperature difference analysis shows that influence of different temperature on the structure is negligible, and temperature difference is not a control factor. Floor vibration and deflection analysis of the top gallery prove that the structure satisfy the requirement of normal service condition.

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