communication tower changes with the increase of the height of the main building. The curves are shown in Figure 4 and Figure 5. It should be noticed that the zero floor means the towers fixed on the ground.

Table 1. The natural periods (s) of the first 9 orders in different structures.

structure	1st order	2nd order	3rd order	4th order	5th order	6th order	7th order	8th order	9th order
40m high tower	0.55145	0.55145	0.12604	0.12604	0.07433	0.05600	0.05600	0.05071	0.03962
70m high tower	1.30011	1.30011	0.30766	0.30568	0.13033	0.13033	0.11382	0.09312	0.07509
1-story frame	0.14261	0.14172	0.13810	0.13756	0.13656	0.12317	0.12013	0.12010	0.11697
2-story frame	0.23859	0.23401	0.22289	0.21932	0.21445	0.18803	0.18754	0.18284	0.15874
3-story frame	0.33949	0.32913	0.30897	0.29893	0.28436	0.24050	0.23845	0.22690	0.18364
4-story frame	0.44274	0.42664	0.38801	0.37278	0.34260	0.27685	0.27292	0.25702	0.19841
5-story frame	0.54765	0.52638	0.45700	0.43808	0.38866	0.30182	0.29621	0.27743	0.20759
6-story frame	0.65396	0.62766	0.51581	0.49426	0.42437	0.31925	0.31227	0.29152	0.21706
7-story frame	0.76160	0.73011	0.56534	0.54188	0.45193	0.33171	0.32365	0.30149	0.25698
8-story frame	0.87061	0.83356	0.60681	0.58193	0.47331	0.34084	0.33194	0.30876	0.29715
9-story frame	0.98106	0.93796	0.64146	0.61550	0.49005	0.34770	0.33814	0.33774	0.31419
10-story frame	1.09304	1.04330	0.67044	0.64366	0.50329	0.37859	0.35296	0.35113	0.34289
11-story frame	1.20666	1.14961	0.69474	0.66731	0.51390	0.41966	0.38890	0.37618	0.35708
12-story frame	1.33203	1.25691	0.71520	0.68725	0.52249	0.46091	0.42705	0.41178	0.37706
13-story frame	1.44073	1.36650	0.73323	0.70480	0.53001	0.50284	0.46609	0.44808	0.40377
14-story frame	1.56007	1.47596	0.74797	0.71917	0.54446	0.53584	0.50511	0.48425	0.42884
15-story frame	1.68152	1.58649	0.76057	0.73145	0.58645	0.54458	0.54075	0.52066	0.45264
16-story frame	1.80519	1.69810	0.77142	0.74199	0.62819	0.58451	0.55760	0.54462	0.47518
17-story frame	1.93117	1.81081	0.78081	0.75110	0.67031	0.62492	0.59448	0.54819	0.49649
18-story frame	2.05960	1.92463	0.78898	0.75899	0.71258	0.66582	0.63170	0.55119	0.51660
19-story frame	2.18837	2.03770	0.79539	0.76516	0.75428	0.70650	0.66852	0.55325	0.53504
20-story frame	2.32416	2.15560	0.80254	0.79767	0.77189	0.74911	0.70679	0.55613	0.55326

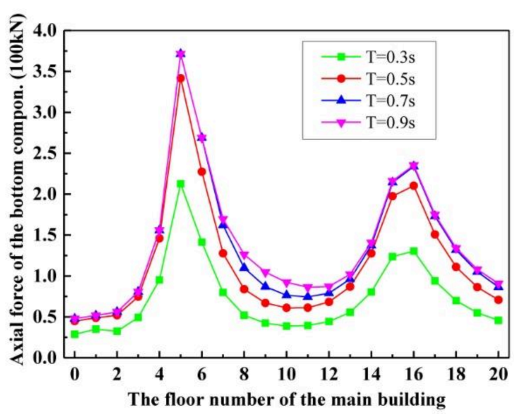


Figure 4. Axial force at the bottom component of the tower with height of 40m.

As shown in Figure 4 and Figure 5, there are several extreme points in the curves. When the 40 m communication tower erected on the 5-story and 16-story buildings, the extreme value appears. When the 70 m communication tower erected on the 3-story and 12-story buildings, the extreme value appears. The axial force of the bottom component reaches the maximum value when the basic natural period of the communication tower is close to the main structure. Compared with communication tower fixed on the ground, the axial force is magnified by about 8 times, which is shown in Table 2. Figure 4 and Figure 5 also show that the whiplash effect not only occurs when the characteristic periods of the first order mode of the communication tower and the main building are close, but also occurs when the periods of high order modes of those are similar. The seismic response of the 40 m communication tower on a 16-story main building has been amplified by nearly 5 times of that on the ground. The seismic response of the 70 m communication tower on a 3-story main building has been amplified by nearly 1.6 times of that on the ground.

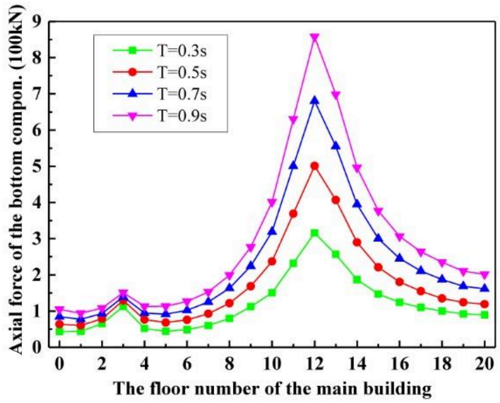


Figure 5. Axial force at the bottom component of the tower with height of 70m

Table 2. Axial forces and amplification factors at the bottom brace of the tower structures when the first order natural periods are similar.

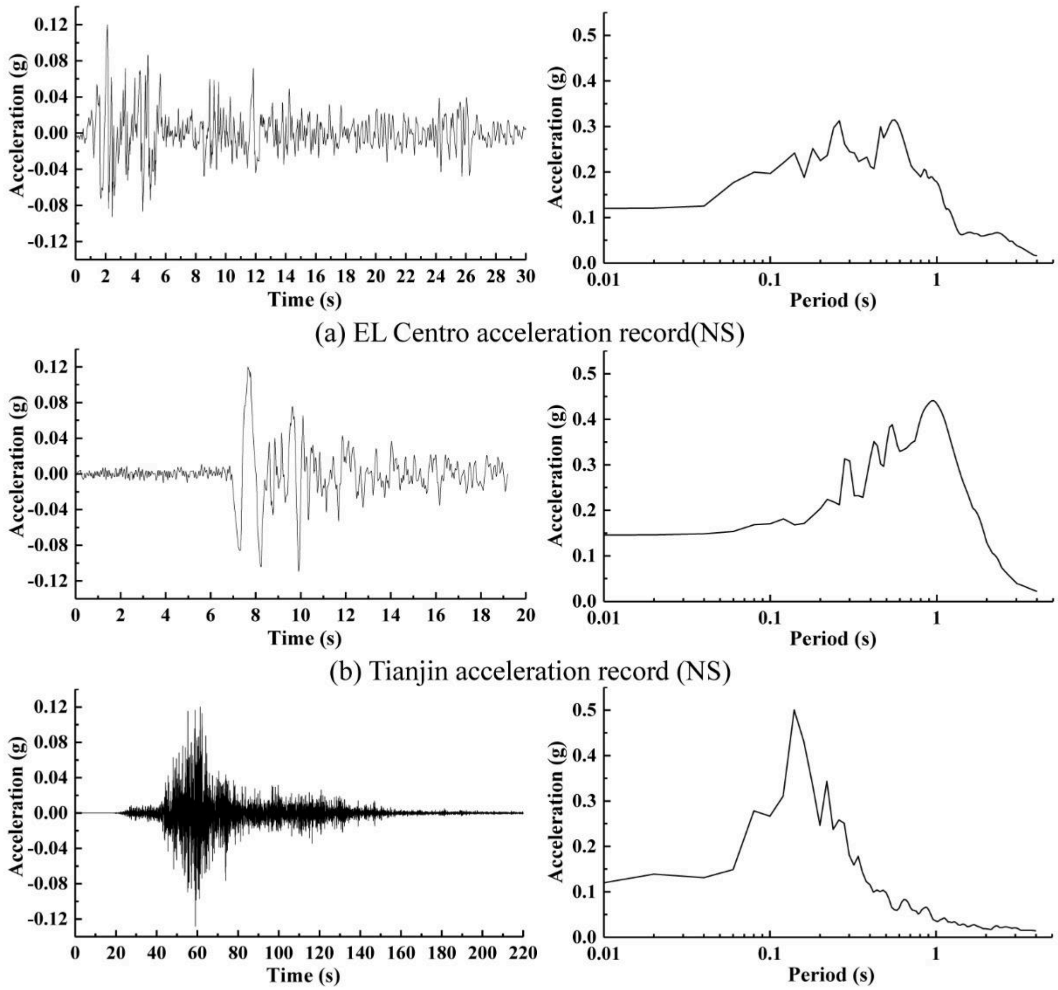
Characteristic period of site soil (T/s)	40 m communication tower			70 m communication tower		
	F_0(kN)	F_m(kN)	F_m/F_0	F_0(kN)	F_m(kN)	F_m/F_0
0.3	28.7	212.7	7.4	43.4	315.9	7.3
0.5	44.9	341.6	7.6	63.6	500.1	7.9
0.7	47.8	371.3	7.8	84.5	680.4	8.0
0.9	47.8	371.3	7.8	104.6	858.1	8.2

Note: 1. F_0 is the axial force at the bottom brace of the tower when the tower is fixed on the ground; 2. F_m is the axial force at the bottom brace of the tower when the characteristic first order natural periods of communication tower and the main building are close.

Dynamic analysis: The linear time history analysis method is adopted for the dynamic analysis. Three earthquake records including Jiangyou acceleration record (EW) in the 2008 Wenchuan Ms 8.0 earthquake, EL Centro acceleration record (NS), Tianjin acceleration record (NS) in the 1976 Tangshan earthquake were adopted as the earthquake excitation inputs. In order to implement the comparison, the peak acceleration of those three earthquake records were all scaled to 0.12g. The scaled records and their corresponding response spectra are shown in Figure 6. The direction of earthquake excitation input is the short axis direction of the main structure.

The axial forces of the bottom component of the tower are shown in Figure 7 and Figure 8. The variation trend of axial force is similar to the results of mode decomposition method. The position of the extreme point of the axial force and the position of the maximum point is related to the basic natural periods of the structures.

When the characteristic site period is smaller than the characteristic periods of the first order mode of the communication towers, the seismic effect of communication tower will increase greatly with the increase of the characteristic site periods. For example, compared to the 70 m tower in Tianjin (NS) and Jiangyou (EW), the axial force of the bottom member of the tower of the former is about 3 times of the latter. When the characteristic site period is close to or larger than the communication tower vibration period, the seismic effect of the communication tower will still increase with the characteristic site periods, but it increases slower than before.



(a) EL Centro acceleration record(NS)

(b) Tianjin acceleration record (NS)

(c) Jiangyou acceleration record (EW)

Figure 6. Three ground motion acceleration records and their response spectra.

CONCLUSION

Whiplash effects appear when the natural period of the communication tower is close to that of the main building. The most sever whiplash effect happens when the natural periods of the tower and the first order mode of main structure are close to each other. Compared with communication tower fixed on the ground, the seismic responses of towers are magnified by about 8 times when the natural periods of the first order mode are close. When the natural periods of high order mode are close, whiplash effect will also occur, but the values of amplification factor of the whiplash effect are smaller. The result of time history analysis shows that the seismic whiplash effect of the communication towers increases with the characteristic periods of site soil. So communication towers should not be built on the main structure which has similar natural vibration period, and it’s better to choose the site which has a short characteristic period.

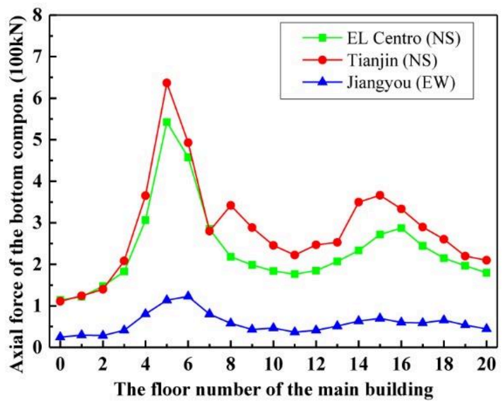


Figure 7. Axial force at the bottom component of the tower with height of 40m.

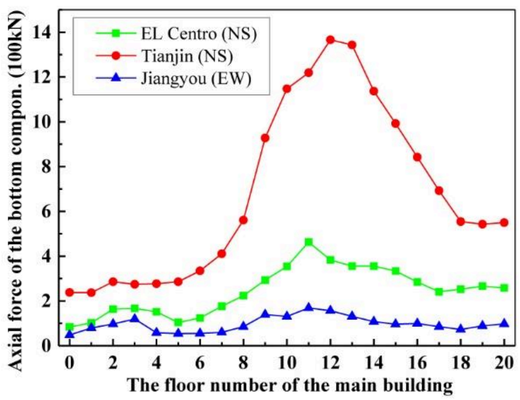


Figure 8. Axial force at the bottom component of the tower with height of 70m

ACKNOWLEDGEMENTS

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Study on the Application of Base Isolation for the Ultra Large Cooling Tower Structure

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ABSTRACT

Base isolation technology has been widely used in areas of buildings and bridges, while no application is found in cooling towers. In present paper, base isolation design using LRB is applied to an ultra large cooling tower structure with the height of 220 m. Seven earthquake acceleration time history records inputted in three orthogonal directions are used in the analysis of the seismic response of the cooling tower structures with and without base isolation. Considering the travelling wave effect, three earthquake acceleration time history records inputted in single direction are used in the analysis of the seismic response of cooling tower structures with and without base isolation under inconsistent seismic displacement excitations. It is found that the base isolation technic diminishes the seismic action on the ultra large cooling tower structure significantly and reduces the base shear force by 74% in rare earthquakes and protect the cooling tower from the adverse travelling wave effect.

INTRODUCTION

The highest cooling tower in the world with 200 m height is located at Niederaussem Power Station in Germany at present (YU, 2016). The height of ultra large cooling towers under construction is over 200m in China. Great efforts have been made in the research on wind resistance of cooling tower structures. However, the seismic action may be in dominant in high seismic intensity areas, on which much less studies is carried out. WOLF (1980) assumed the supporting columns and their foundation which were the most vulnerable part of the structure influenced the seismic response of cooling towers decisively, and slipping and lift-off those took place in the entire foundation in extreme earthquakes played a less important role. Gupta (1976) proposed the response spectrum method which provided the maximum practical use and then the design only for the horizontal component was adequate. Nasir (2002) discovered that the first five circumferential modes and the first lateral mode all lay within the critical band of dominant periods of most earthquakes, which should be considered in seismic design of hyperbolic shells. Sabouri-Ghomi (2006) concluded that the columns of typical cooling towers were influenced greatly by earthquake action and would be rendered unstable and collapsed under severe earthquakes. As the base isolation technic can reduce the seismic action on structures, it is applied to the ultra large cooling tower and the seismic isolation effect is studied in present paper.

BASE ISOLATION DESIGN FOR AN ULTRA LARGE COOLING TOWER STRUCTURE

As shown in Fig.1, the cooling tower is 220m high with the diameter of 185m at the elevation of 0m. The towering shell is supported by 58 columns in the shape of X. The shell and columns are built of C45 and C50 reinforced concrete separately according to Chinese codes. The thickness of the shell ranges from 0.648m on top to 1.7 m at the bottom. The section of the column is 0.9m by 1.6m, and the section of the ring foundation is 2m by 14m. Schematic design of the seismic isolation is employed for this ultra large cooling tower. The column piers are connected by the ring plate which is the same size as the ring foundation. 232 lead rubber bearings with the diameter of 1200mm are placed under the ring plate, which means there are two LRBs below each column pier and two LRBs beside it along the radial direction of the ring foundation (Fig.3 and Fig.4). The period corresponding to the maximum mass participating factor of the non-isolated cooling tower is 0.83s, while that of the isolated tower is 2.97s when the shear strain of the LRBs is 100%.

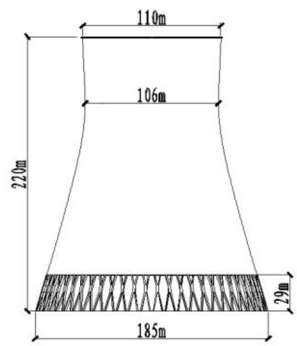

Figure 1. Elevation view

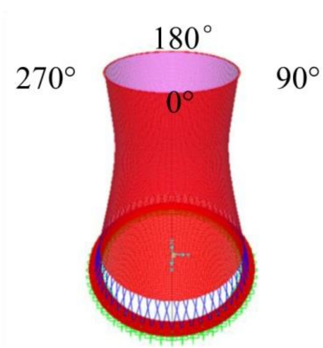

Figure 2. Finite element model

ANALYSIS OF STRUCTURAL RESPONSE UNDER CONSISTENT SEISMIC EXCITATION

To verify the effeteness of the seismic isolation technique, the cooling towers with and without base isolation are analyzed using time history analysis method under consistent seismic acceleration excitations. The finite element model is established, which is shown in Fig.2. The towering shell is simulated by using layered shell element, and the ring plate is simulated using solid element. In order to compare the analysis results of the cooling towers with and without base isolation quantitatively, only the nonlinearity of seismic isolators is considered. The superstructure is analyzed using linear elastic method without the analysis of the plasticity and damage.

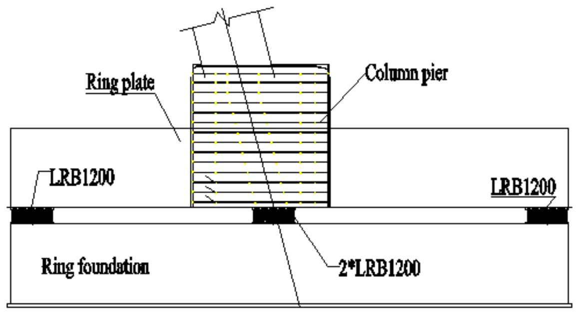

Figure 3. Section view

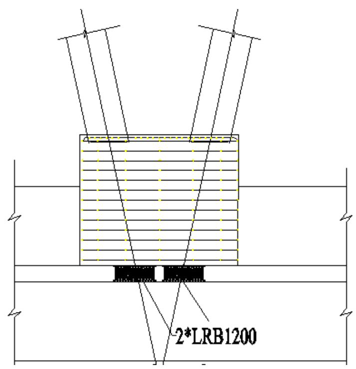

Figure 4. Side view

According to Chinese code for seismic design, the PGAs of the earthquake input of the finite element model in terms of the frequent earthquakes, fortification earthquakes, and rare earthquakes corresponding to 8-degree (0.2g) fortification areas are 70Gal, 200Gal and 400Gal separately. Five natural earthquake records and two artificial ground motions are adopted. The two artificial ground motions are in accordance with the design spectrum using two natural seismic records. The information of the seven acceleration records is listed in Table 1, and the seismic response spectrums and design spectrum are shown in Fig. 5.

Table 1. Seismic acceleration records

Case	Seismic accelaration records	Component
ACC1	1999, Chi-Chi earthquake, TAP051	West
ACC2	1979, Imperial Valley, CA, Meloland Overpass FF	0°
ACC3	1992, LANDERS-JUNE 28, YERMO-FIRE STATION	270°
ACC4	1992, LANDERS-JUNE 28, YERMO-FIRE STATION	360°
ACC5	1999, Chi-Chi earthquake, TCU070	West
ACC6	1995, Kobe, Osaka (Matched)	0°
ACC7	1940, Imperial Valley, El Centro (Matched)	NS

The maximum radial acceleration relative to the ring foundation and maximum radial displacement relative to the ring plate of the cooling tower is observed in four directions, which are along the meridian of 0°, 90°, 180° and 270°. And the top point, the throat point and the bottom point of the towering shell in each direction are taken as the observation points. The average values of maximum seismic acceleration and displacement response obtained from different time history analyses of cooling towers with and without base isolation in terms of frequent earthquakes, fortification earthquakes and rare earthquakes are compared as shown in Figs. 6, 7 and 8. In the figures, “I” denotes the cooling tower with base isolation, and “N” denotes the non-isolated cooling tower. The value of the maximum radial acceleration of the observation point relative to the ring foundation divided by PGA is defined as the acceleration amplification factor.

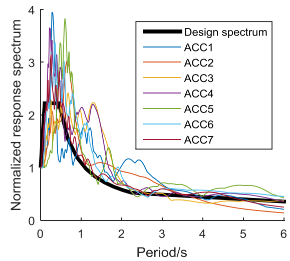

Figure 5. Seismic response spectrum

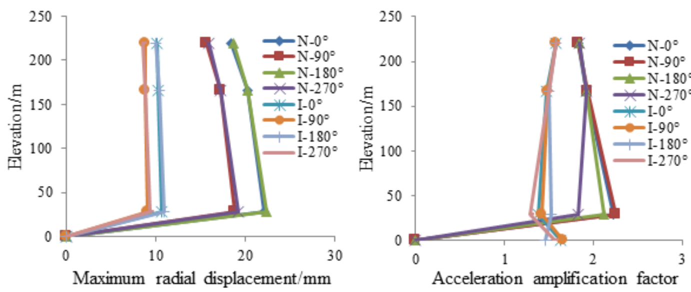

Figure 6. Peak displacement and acceleration amplification factor (PGA=70Gal)

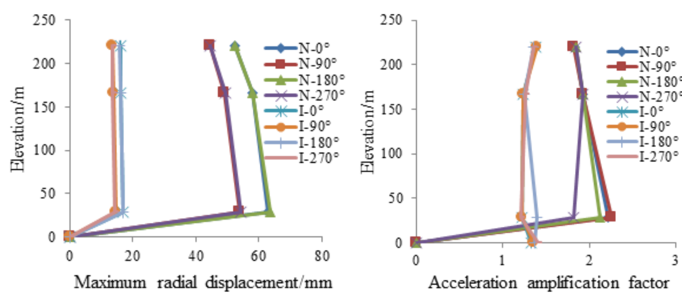

Figure 7. Peak displacement and acceleration amplification factor (PGA=200Gal)

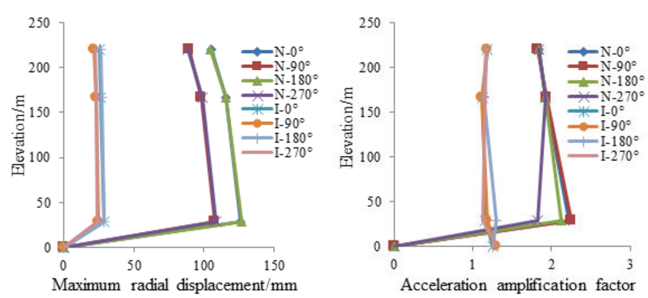

Figure 8. Peak displacement and acceleration amplification factor (PGA=400Gal)

It can be seen from the results that the bottom of the towering shell suffers from more sever seismic acceleration and displacement response than the other parts. The isolation technic reduces the seismic response of the towering shell obviously. The maximum displacement of the shell bottom decreases by 52% for frequent earthquakes, 74% for fortification earthquakes and 77% for rare earthquakes. And its maximum acceleration decreases by 27% for frequent earthquakes, 45% for fortification earthquakes and 48% for rare earthquakes.

The maximum principal tensile and compressive stress of the towering shell are shown in Fig.9 which reflects the remarkable reduction after employing the base isolation technic. The maximum principal tensile stress of the tower shell decreases by 27% for frequent earthquakes, 76% for fortification earthquakes and 86% for rare earthquakes. And its maximum principal compressive stress decreases by 25% for frequent earthquakes, 53% for fortification earthquakes and 66% for rare earthquakes.

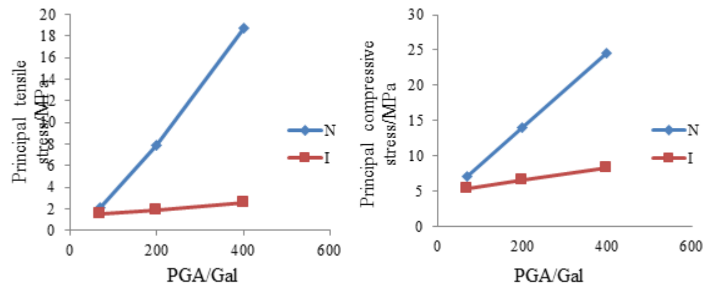

Figure 9. Maximum principal tensile and compressive stress of the towering shell

The average values of maximum internal force of columns obtained from different time history analyses are shown in Figs.10 and 11. The maximum axial compressive force of columns decreases by 27% for frequent earthquakes, 54% for fortification earthquakes and 65% for rare earthquakes when the base isolation technic is adopted. And the maximum shear force along the height of column section decreases by 22% for frequent earthquakes, 38% for fortification earthquakes and 54% for rare earthquakes.

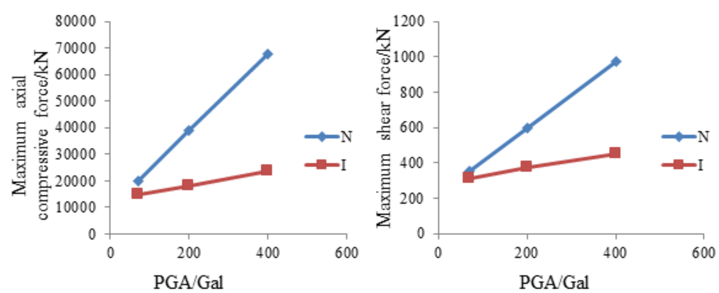

Figure 10. Maximum axial compressive force and shear force of columns

The maximum torque of columns decreases by 6% for frequent earthquakes, 52% for fortification earthquakes and 65% for rare earthquakes compared with those of the non-isolated cooling tower. And the maximum moment of the column about its principle bending axis decreases by 22% for frequent earthquakes, 63% for fortification earthquakes and 64% for rare earthquakes.

Fig. 12 shows that the base isolation technic reduces the base shear force of the ultra large cooling tower by 25% for frequent earthquakes, 72% for fortification earthquakes and 74% for rare earthquakes, which diminishes the transmission of seismic action from the foundation to the superstructure remarkably.