16th World Conference on Earthquake, 16WCEE 2017

Santiago Chile, January 9th to 13th 2017

Paper N°1489

Registration Code: S-T1465981736

Seismic Performance of a Steel Transmission Tower during the 2011 Great East Japan Earthquake

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Abstract

The Central Research Institute of Electric Power Industry (CRIEPI) has launched a field observation study on an ultrahigh voltage steel transmission tower approximately 140m in height to evaluate the soundness of aged steel transmission towers under natural environmental conditions, including earthquakes and strong winds. In order to study earthquake and wind resistance we set up a variety of instruments to observe ground motion, acceleration and axial forces The instruments recorded ground motion and the dynamic response of the tower during the 9.0 Mw Tohoku Earthquake. The observation provided rare data on the earthquake resistance capabilities of steel transmission towers.

We examined earthquake-induced dynamic response characteristics based on axial forces and acceleration time histories of members and we performed frequency analyses. Horizontal peak ground surface accelerations of about 0.5G were amplified roughly five or six times at the top of the tower. The amplification was governed by the second stage of the tower, because the ground motion involved relatively higher frequency components. We applied acceleration data taken from the top of the tower and story shear force data from some of the panels to estimate damping ratios for the first and second modes. Using the half-power method, we estimated all of the ratios to be approximately 1%. Some residual axial force extended to the primary members, but peak axial forces remained below the buckling strength for those members. In addition, visual data obtained by remote industrial television cameras showed that the earthquake induced vibrations simultaneously at the redundant members and along the entirety of the tower. This indicates that there was a slight dislocation among members at the connection panels during the earthquake.

Keywords: the 2011 Great East Japan earthquake, transmission steel tower, earthquake observation

1. Introduction

Even though Japan has experienced some of the world's largest earthquakes in recent years, notably the Kobe Earthquake in 1995 and the Tohoku Earthquake in 2011, there has been no record of a steel transmission tower collapsing due to seismic motion. However, a seismic response analysis on one tower based on data from the Kobe Earthquake showed the occurrence of some elastic response to the seismic ground motion. Following the Tohoku Earthquake, much research has been focused on the earthquake resistance of infrastructures, prompting calls for improved methods to evaluate the seismic performance of steel transmission towers. In Japan, where design criteria for steel transmission towers had previously centered on wind load, the dynamic response of towers during an earthquake received limited attention.

In 2010 we set up a system to conduct field observations on an ultrahigh voltage steel transmission tower to study the soundness of aged towers under natural environmental conditions that include earthquakes and strong winds. Using this system, we were able to record the dynamic response of the tower during the Tohoku Earthquake. In this paper, we discuss the dynamic characteristics of the tower based on the observed data.

2. Observation methods

2.1 Subject tower

Picture 1 shows the subject tower which is a tension square type consisting of steel pipe members and with a height of 142 m. Figure 1 identifies the direction of the tower, section numbers and the names of the crossarms and the main posts.

Pic. 1 – Subject tower

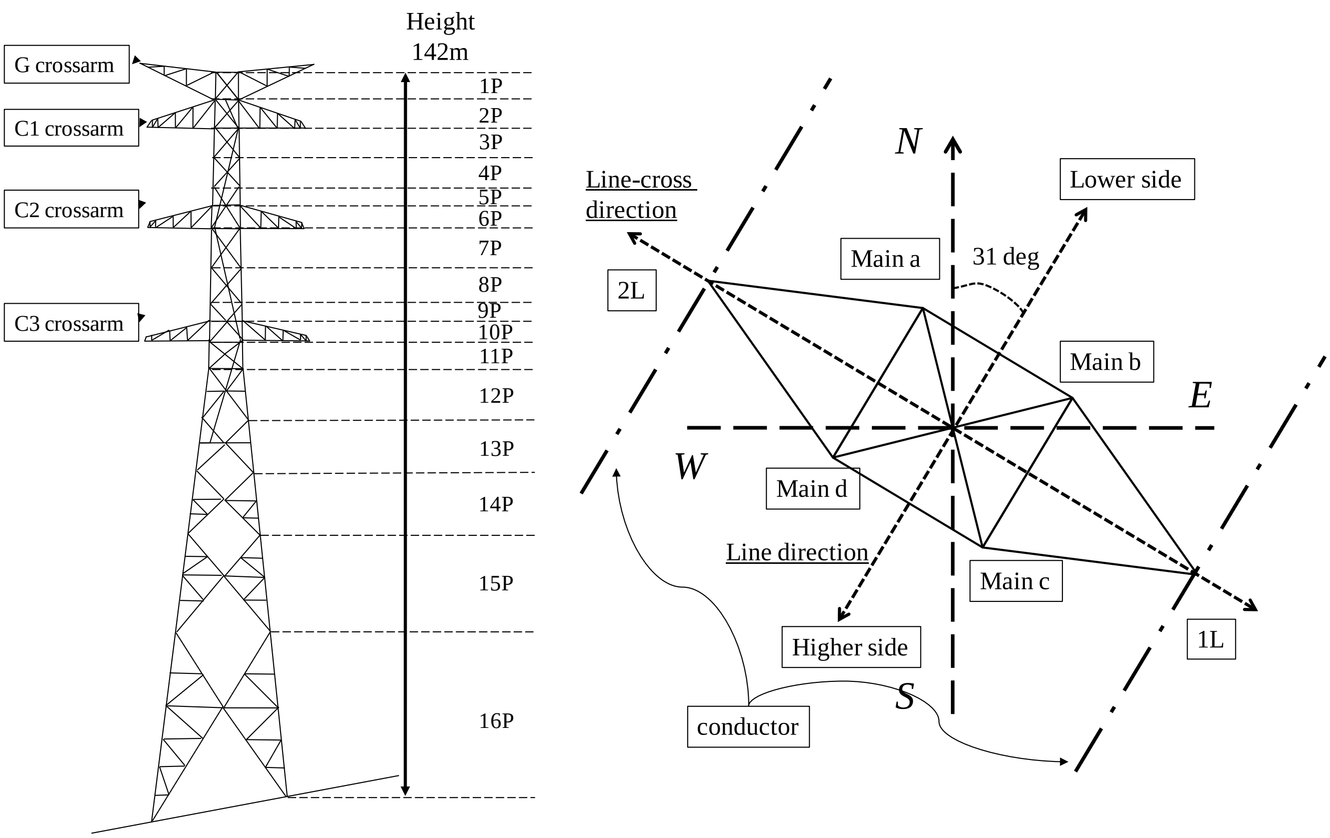


Fig. 1 – part name

2.2 Measurements

In order to investigate the tower's response during an earthquake, we measured factors including acceleration, the force of each member, wind velocity, wind direction and deflection of cables. Figure 2 shows the positions of 83 measurement sensors installed on the tower. We also used three industrial type television cameras to record behavior of the cables and tower members.

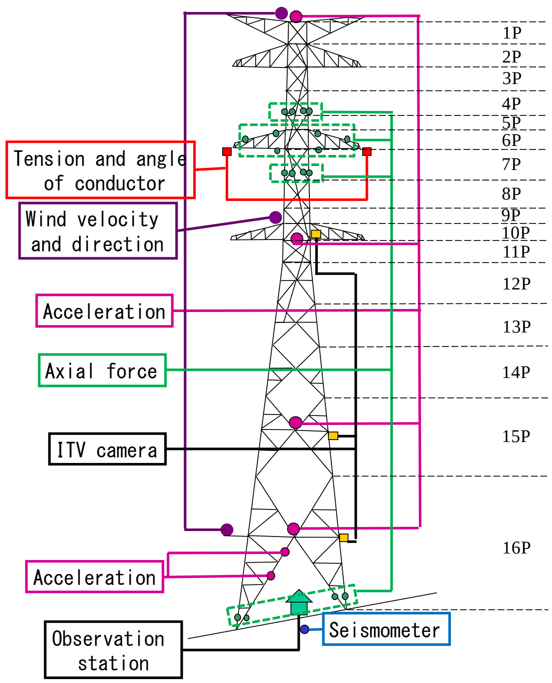
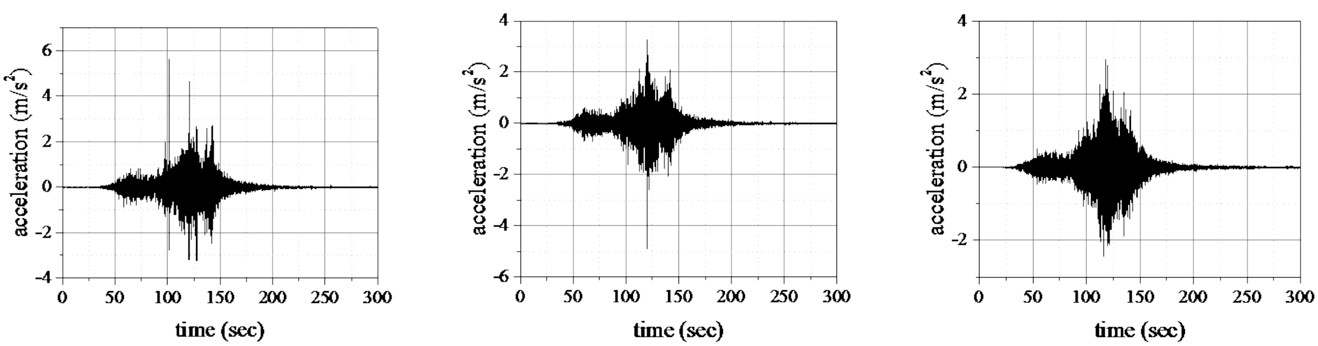


Fig. 2 – Measurement positions

3. Observation data and discussion

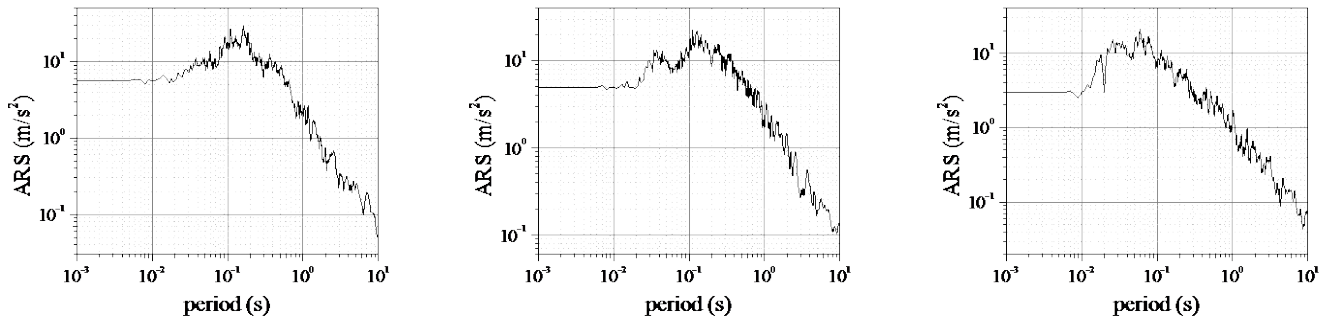
3.1 Ground motion

Figure 3 and Figure 4 show the time histories and the acceleration response spectra of the ground acceleration at the bottom of the tower. The peak ground motion is 6.64 m/s^2 for the north-south direction and 4.92 m/s^2 for the east-west direction. Values for the acceleration response spectrum were greater in the short-period than in the long-period. This trend was also identified as a characteristic of the observed ground motion during the Tohoku Earthquake, designated as the recorded seismic wave at the K-NET Tsukidate Station.



(a) NS (b) EW (c) UD

Fig. 3 – Ground motion during the 2011 Tohoku earthquake

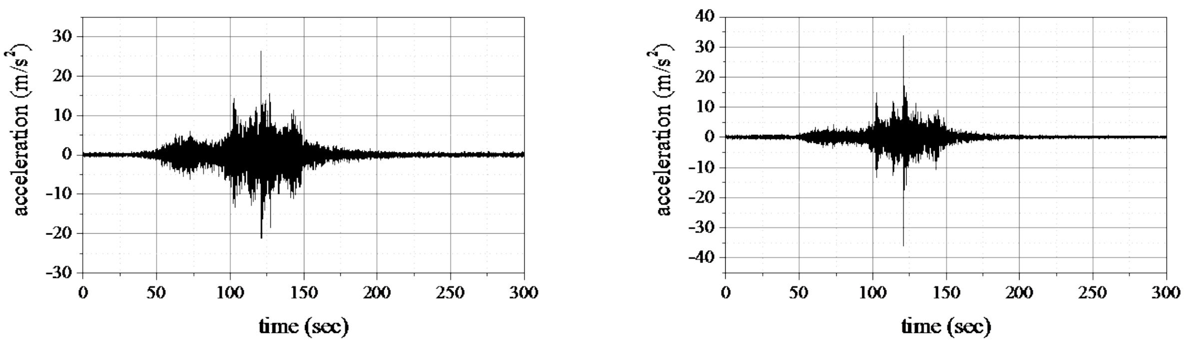


(a) NS (b) EW (c) UD

Fig. 4 – Acceleration response spectrum (ARS) of the ground motion

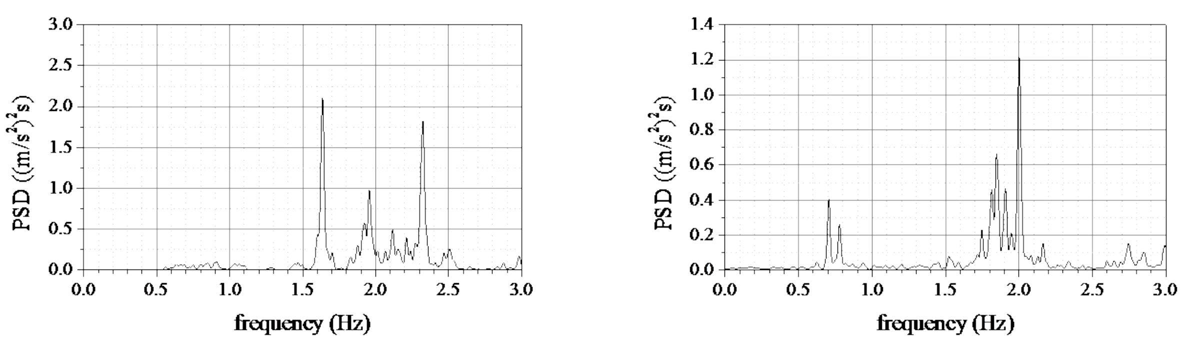
3.2 Tower response

The acceleration time histories and the power spectrum density functions of the line and line-cross directions at the top of the tower are shown in Figure 5 and Figure 6. The maximum acceleration figures for the line and line-cross directions at the top of the tower are 26.4 and 36.1 m/s^2, respectively. In Figure 6, a strong peak is found near 2.0 Hz. It appears that this peak frequency is from the second mode of the tower because the first mode period of a tower is generally equal to 1% of the tower height.



(a) line direction (b) line-cross direction

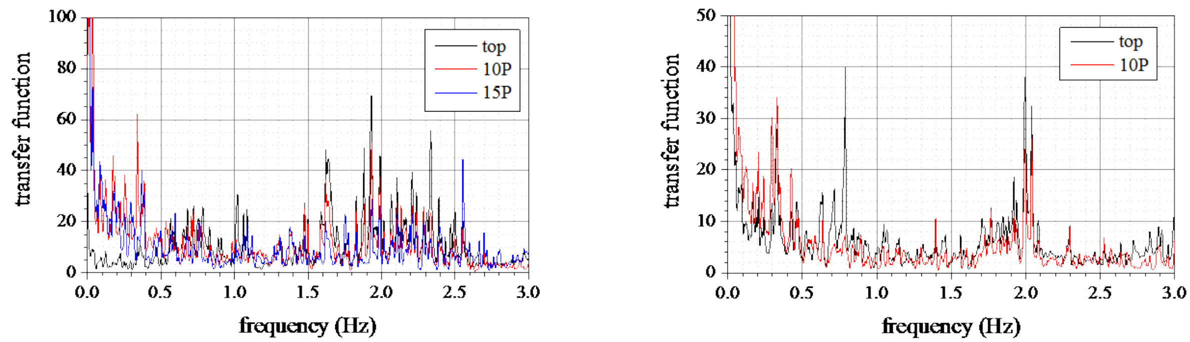
Fig. 5 – Acceleration time histories of the tower top



(a) line direction (b) line-cross direction

Fig. 6 – Power spectral density function of tower top acceleration

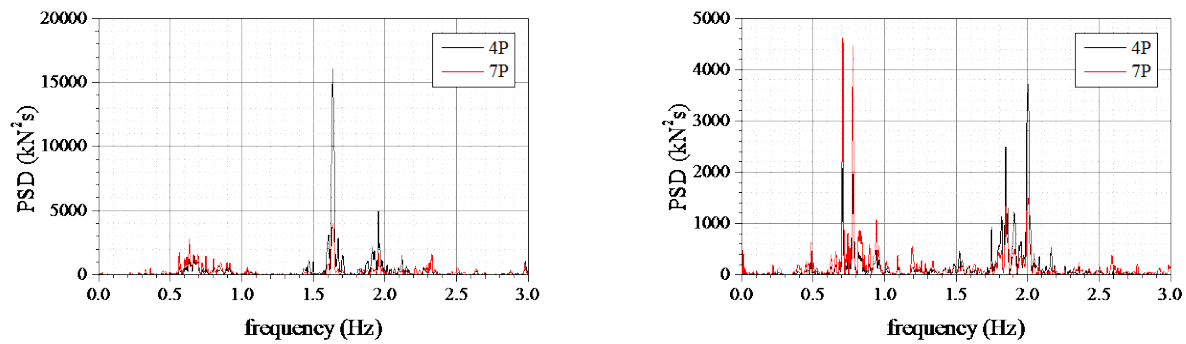
We investigated the dynamic characteristics of the tower by focusing on the transfer function between the top of the tower and the ground motion and the power spectral density function of the story shear force using calculations based on force values for each member. Based on the transfer functions and the power density spectral functions shown in Figure 7 and Figure 8, the 1st mode frequency is 0.63 Hz for the line direction and 0.70 Hz for the line-cross direction. The 2nd mode frequencies are 1.64 Hz and 2.00 Hz, respectively. Figure 9 shows 1st mode and 2nd mode shapes of a finite element model of the target tower. The first natural frequency of the FE model is 0.724 Hz and 0.712 Hz for line direction and line-cross direction, respectively. Because the FE model has no wires, it is generally assumed that wire effects tower 1st mode response of line direction.



(a) line direction

(b) line-cross direction

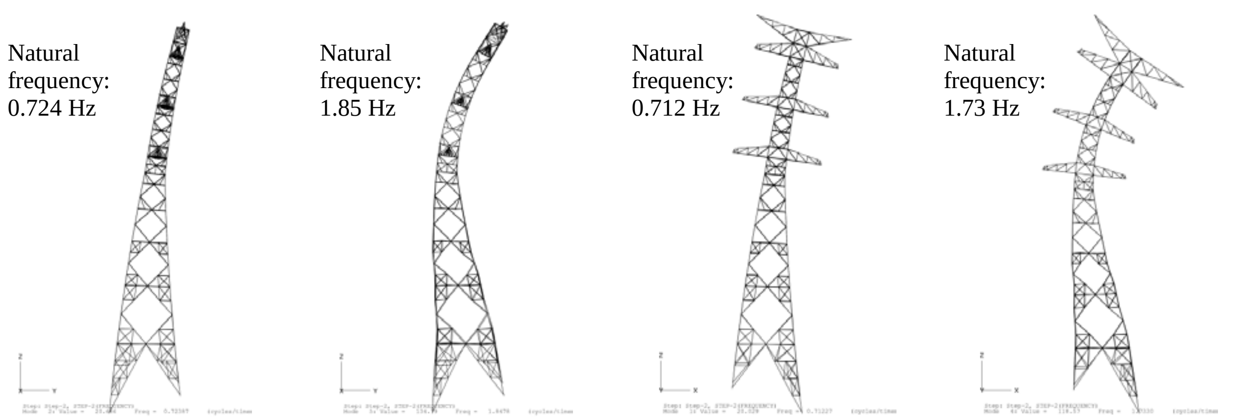
Fig. 7 – Transfer function between tower top response and ground motion



(a) line direction

(b) line-cross direction

Fig. 8 – Power spectral density function



(a) 1st mode (line) (b) 2nd mode (line) (c) 1st mode (line-cross) (d) 2nd mode (line-cross)

Fig. 9 – Mode shapes of target tower

Table 1 shows damping ratios for the line and line-cross direction which we estimated based on the transfer functions and power spectral density functions. The damping ratio for the 1st mode is 1% and is equal to or greater than that of 2nd mode. The story shear force factor for the bottom of the tower is 0.2 and is equal to that of Level 1 earthquake motions as defined in the regulation Design Specification of Highway Bridges in Japan. It is generally assumed that the 1st mode damping ratio of a steel transmission tower with pipe members is equal to 1% during a moderate earthquake, and the observed data corresponds to this value.

Table 1 – Natural frequency and damping ratio

(a) Acceleration

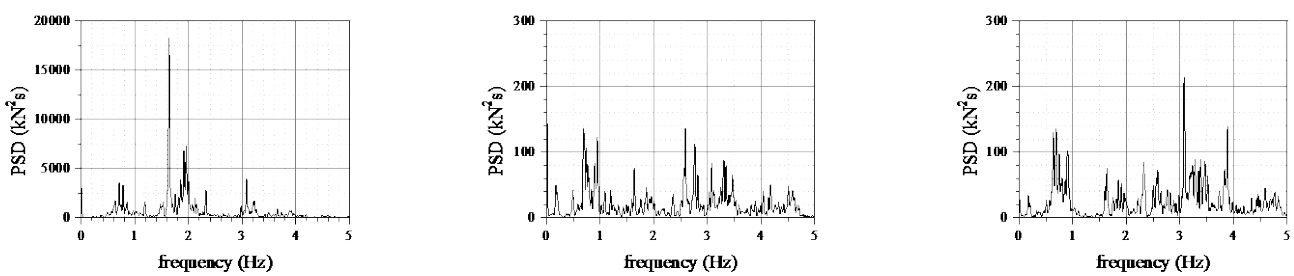
		Line		Line-cross	
		frequency	damping ratio	frequency	damping ratio
1st mode	top	—	—	0.705 Hz	0.00799
	10P	—	—	0.705 Hz	0.00802
2nd mode	top	1.64 Hz	0.00694	2.00 Hz	0.00526
	10P	1.63 Hz	0.00711	2.00 Hz	0.00529

(b) Story shear force

		Line		Line-cross	
		frequency	damping ratio	frequency	damping ratio
1st mode	4P	0.632 Hz	0.00926	0.705 Hz	0.00744
	7P	0.632 Hz	0.00813	0.705 Hz	0.00745
2nd mode	4P	1.64 Hz	0.00674	2.01 Hz	0.00518
	7P	1.64 Hz	0.00688	2.01 Hz	0.00560

3.3 Member response

Figure 10 shows the power spectral density functions of axial forces for the main and bracing members at the 7th panel. These functions have peaks which are near those for the 1st and 2nd modes. In addition, we observed a peak frequency near 0.2 Hz for the power spectral density functions of the bracing members. This low frequency peak is the peak of the cables response because the frequency is less than that for the 1st mode of the tower.



(a) Main member (b) Bracing member (c) Bracing member

Fig. 10 – Power spectral density functions of tower member axial force

Figure 11 shows estimated safety factors based on the maximum axial force for the tower members and the design buckling force. Peak axial forces remained under the buckling strength of the members. We confirmed that the tower response was elastic because of the peak axial forces.

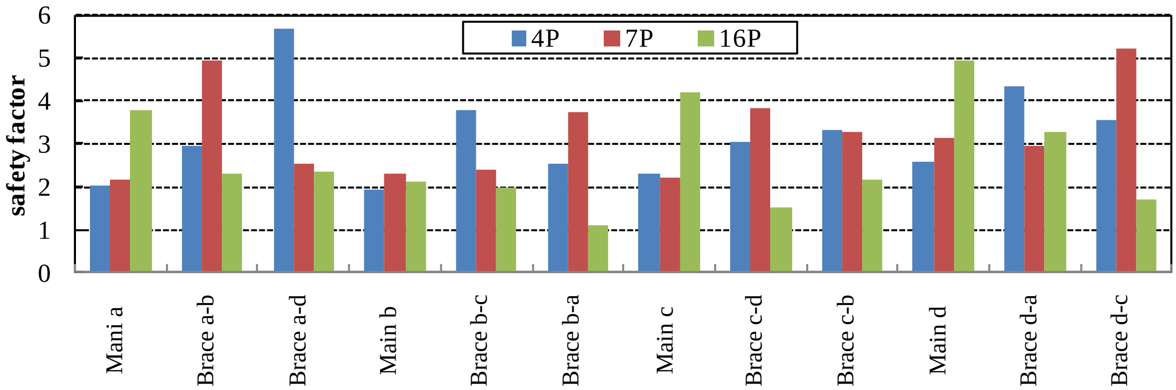
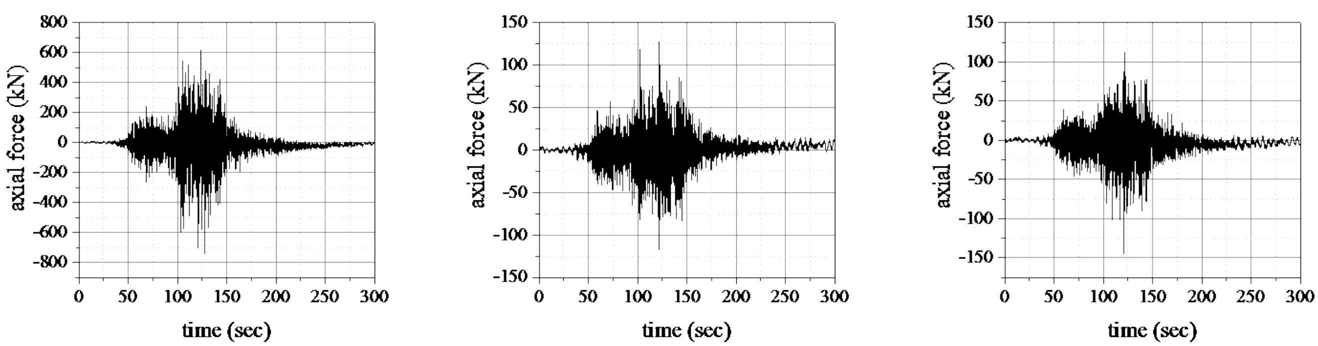


Fig. 11 – Safety factor of tower member during earthquake

Axial force time histories of the main and bracing members at the 7th panel are shown in Figure 12. The center of the axial force response prior to the earthquake is not equal to that recorded immediately following the event. Figure 13 shows the ratio of the difference between the center of axial force prior to and immediately following the earthquake. As a result, the time history for the bracing member shows a difference of axial force before and after the earthquake that is greater than that indicated in the time history for the main member. We attribute this disagreement in the axial force to the slight slippage that occurred at the joints between each member during the earthquake.



(a) NS (b) EW (c) UD

Fig. 12 – Time history of tower member axial force (7th panel)

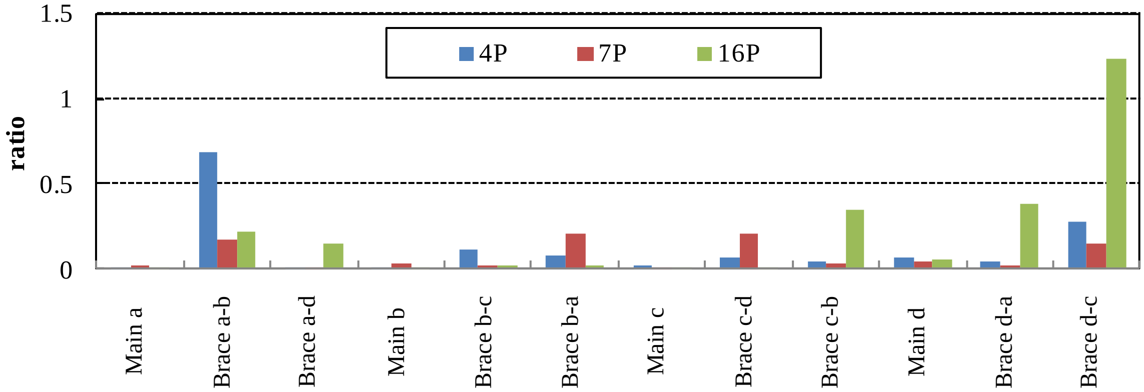


Fig. 13 – Gap of a center of axial force before and after earthquake

4. Conclusions

In this paper, we have described our study on the dynamic response characteristics of ultrahigh voltage steel transmission towers based on field observation and we have been able to make the following conclusions:

(1) Horizontal peak ground surface accelerations of approximately 0.5G were amplified roughly five or six times at the top of the tower. The amplification was governed by the second mode of the tower because the ground motion involved relatively higher frequency components.

(2) Based on the amount of acceleration at the top of the tower and the story shear force, we determined that the first mode natural frequency for the line and line-cross directions was equal to 0.632 Hz and 0.705 Hz, respectively. Moreover, the 1st mode damping ratio for both directions was equal to approximately 0.01.

(3) The earthquake simultaneously induced vibrations at the redundant members and along the entirety of the tower thus causing slight dislocation among members at the connection panels.

5. Acknowledgements

We are particularly grateful for the assistance given by Tokyo Electric Power Company Holdings, Inc. and J-Power Systems Co.

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