SEISMIC RESPONSE CHARACTERISTICS OF A LONG-PERIOD AND LONG-SPAN BRIDGE BASED ON RECORDED WAVEFORMS

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Abstract

For the Tsurumi Tsubasa Bridge, which is a long-span, cable-stayed bridge on the Tokyo metropolitan expressway, this study revealed lower-level linear dynamic responses by performing seismic response analyses based on a three-dimensional frame structural model exposed to multi-input ground excitations, coupled with analysis of the observation records from the aftershock of the 2011 off the Pacific coast of Tohoku earthquake. Modal parameters and vibration modes of the bridge from the modal analysis were identified. By comparing the analytical responses with the observed ones in the bridge, the reproducibility of the seismic records was discussed, and the validity of the dynamic analytical model was verified.

The Tsurumi Tsubasa Bridge is a three-span, continuous steel cable-stayed bridge located in Tokyo Bay. The total length is 1020.0 m, the centre span is 510.0 m and each side span is 255.0 m. On this bridge, seismic observation was carried out by setting 25 accelerometers and 2 displacement transducers measuring 50 acceleration and displacement components. The sampling frequency of the observed waveform was set to 100 Hz, and the measurement duration was 174.50 s. By using beam elements, nonlinear beam elements, truss elements, linear spring elements, nonlinear spring elements and nonlinear viscous elements, a three-dimensional, nonlinear, frame structural model of the bridge was assembled. A Rayleigh damping matrix was assumed, and the ground motion was implemented in the model as a multi-variant input at each foundation. The Newmark β method, with β = 0.25, was used for numerical integration at an interval of 0.01 s.

The result of the modal analysis in this study was basically consistent with those in previous studies, and it was greatly dependent on the modelling of elastic restraint cables. For vertical (UD) girder motion, the first symmetric and asymmetric bending modes (T_1 = 4.811 s and T_3 = 3.455 s), the first torsional mode (T_5 = 2.001 s) and the second symmetric and asymmetric bending modes (T_6 = 1.953 s and T_7 = 1.658 s) were dominant. For transverse (TR) girder motion, the first lateral symmetric bending mode (T_2 = 3.925 s) and the first bending modes of the two main towers (T_{10} = 1.431 s and T_{11} = 1.359 s) were significant. The first lateral symmetric bending mode had a relatively larger difference of natural period than that in previous studies because it depended on the modelling of the boundary conditions around the vertical axes of the main towers. For longitudinal (LG) girder motion, the first swing mode (T_4 = 2.831 s) was significant.

On the basis of the dynamic analysis, seismic response characteristics of the main towers and girders were examined to determine whether a given vibration mode was excited in the aftershock. Numerical modelling provided high reproducibility in the comparison of Fourier spectra for the analytical and observed aftershock waveforms. From this comparison, it was estimated that for girder longitudinal motion, the lack of substantial excitement of the first swing mode by this aftershock was due to relatively lower long-period motions.

Keywords: long-period bridge, long-period ground motion, dynamic analysis, seismic response characteristics

1. Introduction

Observational records of the aftershocks of the 2011 off the Pacific coast of Tohoku, earthquake Japan, were obtained at the Tsurumi Tsubasa Bridge in Tokyo Bay at 2:56:18 am on 13 March 2011. In a previous study, Yamaguchi *et al*. [1] verified the dynamic characteristics of the Tsurumi Tsubasa Bridge by carrying out a series of vibration tests and analysing the data. Yamamoto *et al*. [2] used observational records of the 2004 Niigata-ken Chuetsu earthquake to clarify the nonlinear seismic behaviours and natural vibration characteristics of the Yokohama Bay Bridge, the Rainbow Bridge and the Tsurumi Tsubasa Bridge, which are all long-period, long-span bridge structures. Furthermore, Siringoringo and Fujino [3],[4] identified the structural parameters of the Yokohama Bay Bridge and the Katsushika Harp Bridge using observational seismic records.

In this study, we perform dynamic analysis using a three-dimensional nonlinear frame model of the Tsurumi Tsubasa Bridge. This analysis verifies the reproducibility and validity of the analysed seismic responses with respect to the observed data.

2. Target bridges and seismic records

The Tsurumi Tsubasa Bridge is a three-span, continuous steel cable-stayed bridge located in Tokyo Bay (Fig. 1). The bridge length is 1020.0 m, the central span is 510.0 m, and each side span is 255.0 m. The base of the main tower is a steel-frame reinforced concrete structure, and the end piers are reinforced concrete structures. Vertical bearings, horizontal bearings, elastic restraint cables, stoppers and vane-type oil dampers are installed in the main tower. Pendel bearings and horizontal bearings are installed in an end pier. The vertical bearing is movable in both the longitudinal direction and the transverse direction. The horizontal bearings in the main tower and end pier are also free in the longitudinal direction but fixed in the transverse direction. The pendel bearings support motion in the vertical direction and the longitudinal direction. An elastic restraint cable is used to connect the main tower with the main girder in the longitudinal direction.

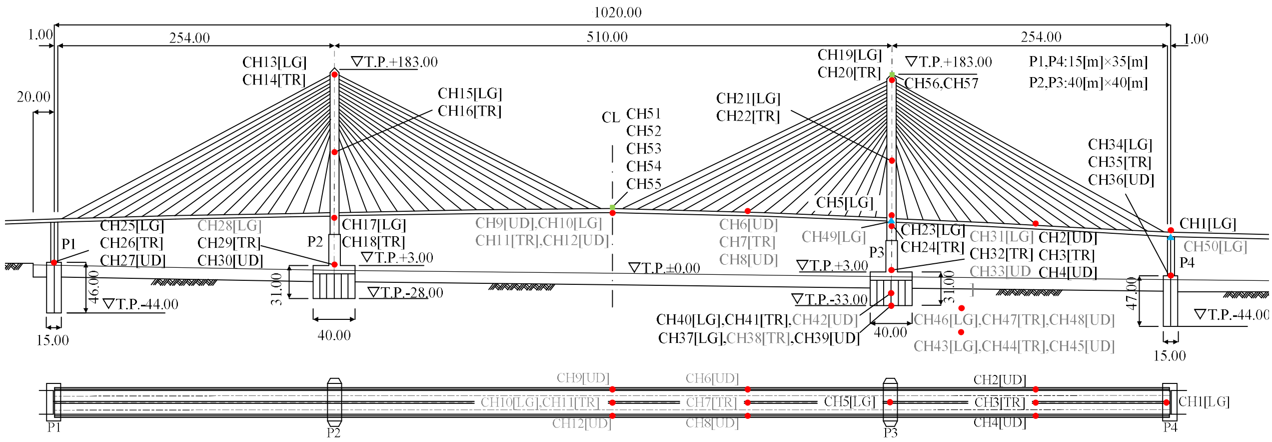


Fig. 1 – Tsurumi Tsubasa Bridge

Seismic observation was performed using accelerometers and displacement transducers at 27 locations on 50 components over a duration of 174.5 s. The sampling frequency of the waveform was 100 Hz. The acceleration waveform at the base of the P3 main tower showed a long-period component of 2.5 s or more with an amplitude of 0.03 m/s^2 according to Fourier spectrum analysis.

3. Analytical modelling

On the basis of a previous study [2] that carried out dynamic analyses of the Tsurumi Tsubasa Bridge based on observed data from the 2004 Niigata-ken Chuetsu earthquake and the bridge's construction material [5], a three-dimensional nonlinear frame model was constructed as shown in Fig. 2. Linear beam elements were used to model the main girder, the connections in the girder fixed by an elastic restraint cable and by a main cable, the cross beam of the main girder and the cross beam of the main tower supporting the main girder. Nonlinear beam elements were used to model the end pier and the lower part of the main tower under the main girder, down to the foundation. The main cable and the elastic restraint cable were modelled as truss elements, with the initial tensions set based on the study in [5]. The vertical and pendel bearings were also modelled as truss elements. Horizontal bearings were modelled as linear spring elements, and stoppers were modelled as nonlinear spring elements. Sway-rocking coupled motion in the foundation was modelled as a nonlinear spring element. Vane dampers were modelled as nonlinear viscous elements.

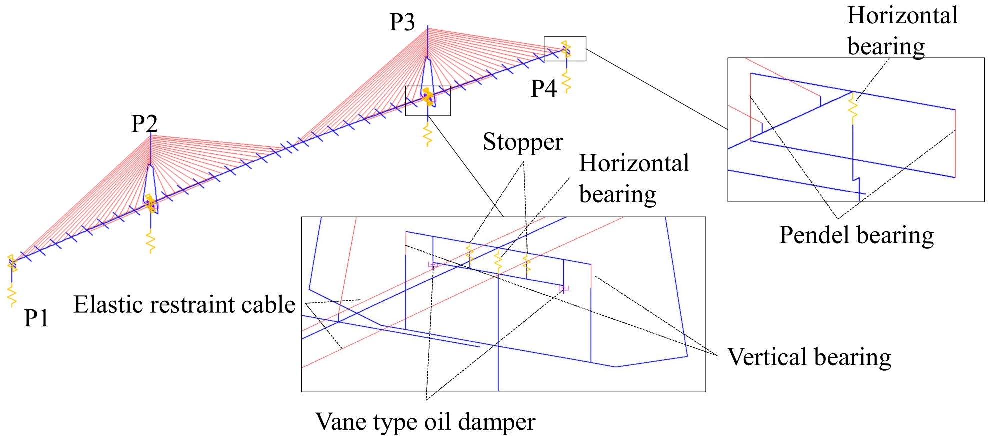


Fig. 2 – A three-dimensional nonlinear frame model of the Tsurumi Tsubasa Bridge

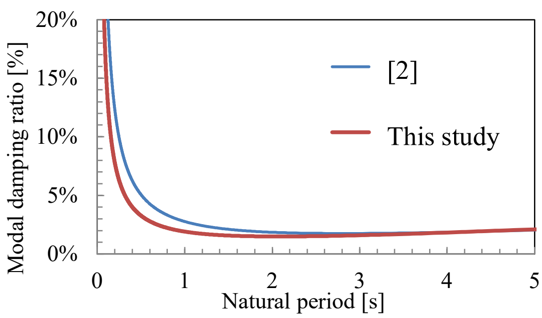


Fig. 3 – Modal damping ratio for Rayleigh damping

As for multi-input ground excitations, the P1 input ground motions were given by CH25 (LG direction), CH26 (TR direction) and CH27 (UD direction), and the P2 input ground motions were given by CH25 (LG direction), CH29 (TR direction) and CH30 (UD direction). The P3 ground motions were inputted by CH37 (LG direction), CH39 (UD direction) and CH41 (TR direction), whereas the P4 ground motions were inputted by CH34 (LG direction), CH35 (TR direction) and CH36 (UD direction). The Newmark β method, with $\beta = 0.25$, was used for numerical integration, with an interval of 0.01 s. The damping matrix in the equation of motion was assumed to be Rayleigh damping as shown in Fig. 3.

4. Modal analysis results

The results from the modal analysis were compared with those obtained in previous studies [1],[2],[6] and with the results computed by Metropolitan Expressway Company Limited (MEX) after seismic retrofitting as shown in Fig. 4. Fig. 5 shows the dominant natural vibration modes. In brief, the results in the present study show good agreement with these other results, as shown in Fig. 4.

In the UD direction of the main girder motion, the dominant modes were the first symmetric and asymmetric bending modes (T_1 = 4.811 s and T_3 = 3.455 s), the first torsional mode (T_5 = 2.001 s) and the second symmetric and asymmetric bending modes (T_6 = 1.953 s and T_7 = 1.658 s). Moreover, Fig. 5 shows the vibration of the main tower in the LG direction due to UD vibration of the main girder.

In the TR motion direction, the significant modes were the first lateral symmetric bending mode of the girder (T_2 = 3.925 s) and the first bending modes of the two main towers (T_{10} = 1.431 s and T_{11} = 1.359 s). In comparison with previous studies, the largest difference in natural period was found in the first lateral symmetric bending mode, since it depended on the modelling of the boundary conditions around the vertical axes of the main towers. In addition, Fig. 5 shows the vibration of the main tower in the TR direction due to TR vibration of the main girder.

In the LG direction of girder motion, the first swing mode of the girder (T_4 = 2.831 s) was significant. Since this mode is difficult to observe and analyse during construction, it is important to analyse this mode using actual seismic observation records. In the present model, this mode depended strongly on the modelling of elastic restraint cables.

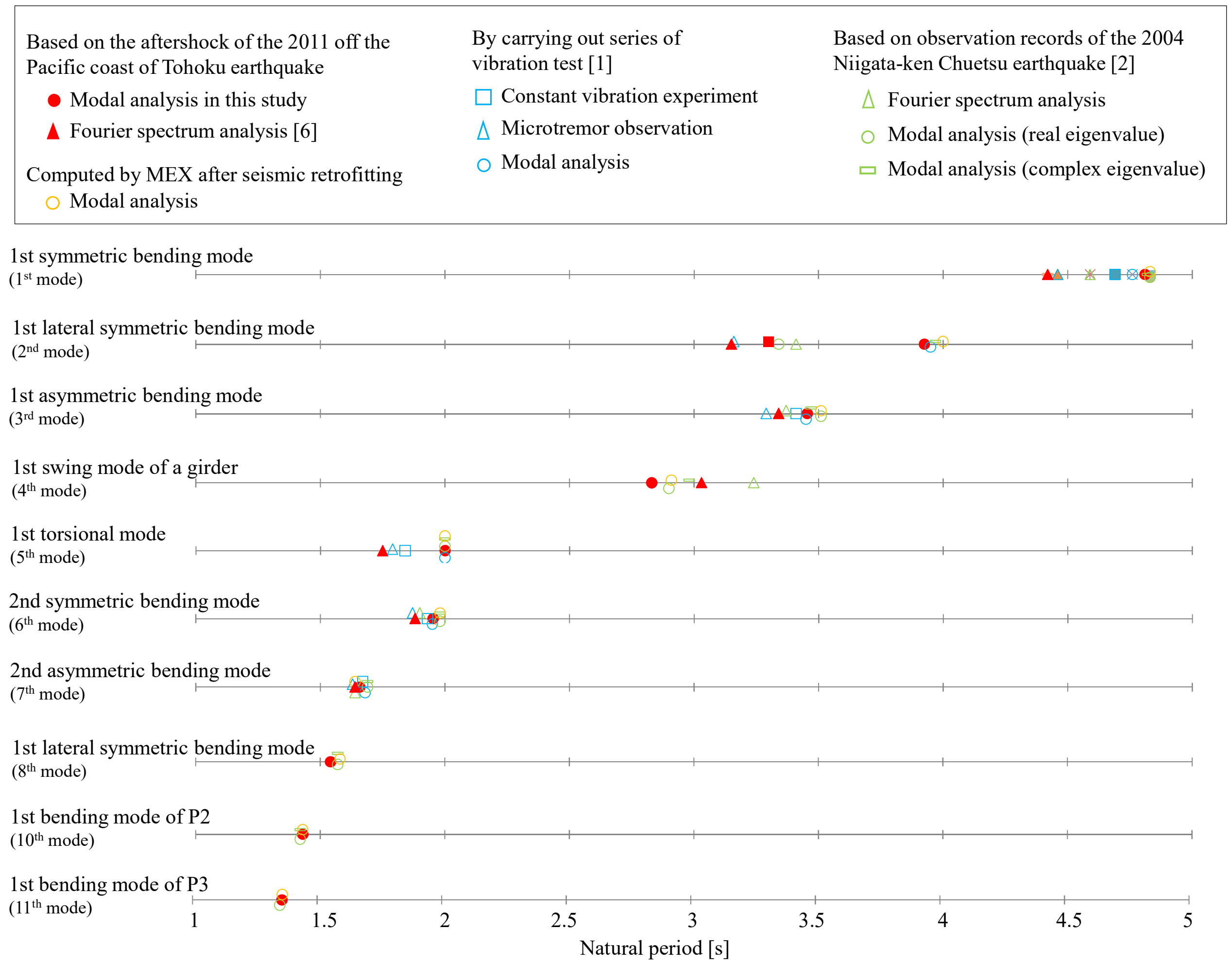


Fig. 4 – Comparison of natural periods

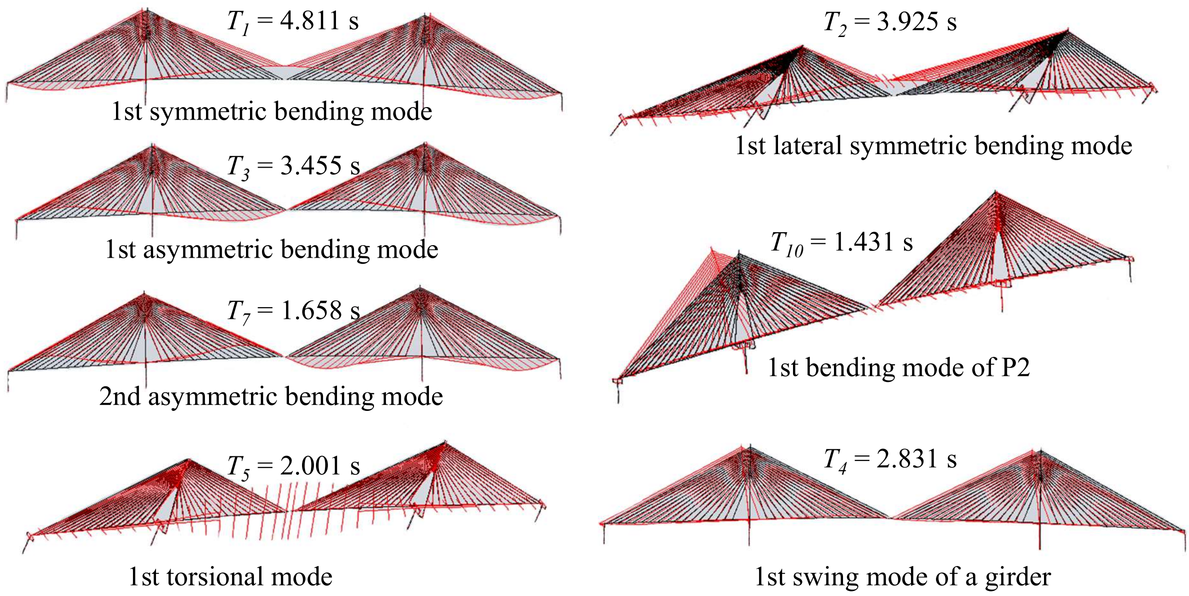


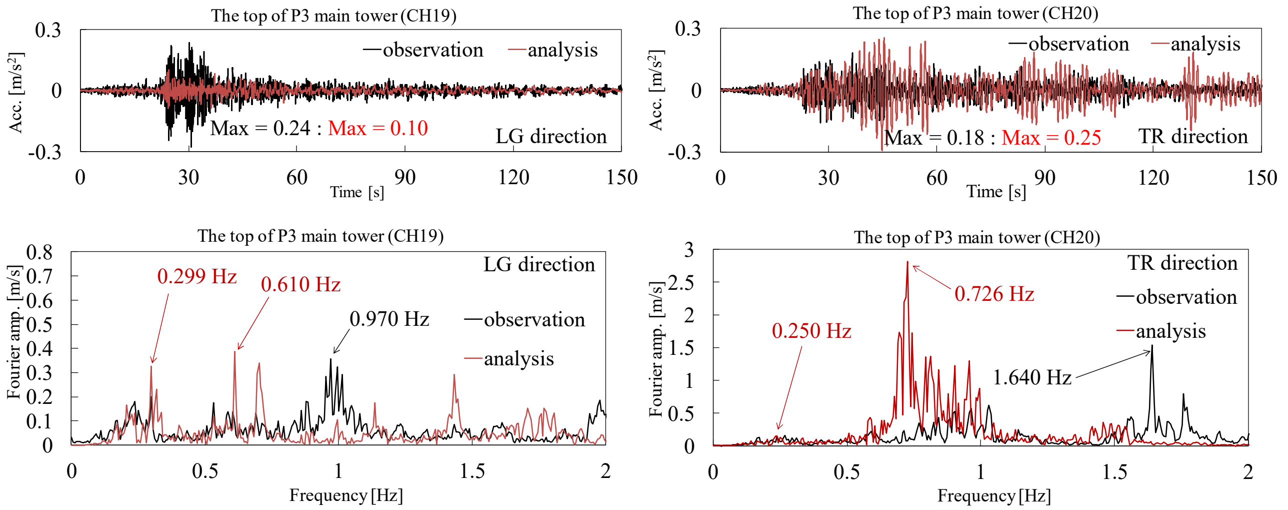
Fig. 5 – Natural vibration mode types

5. Seismic response characteristics

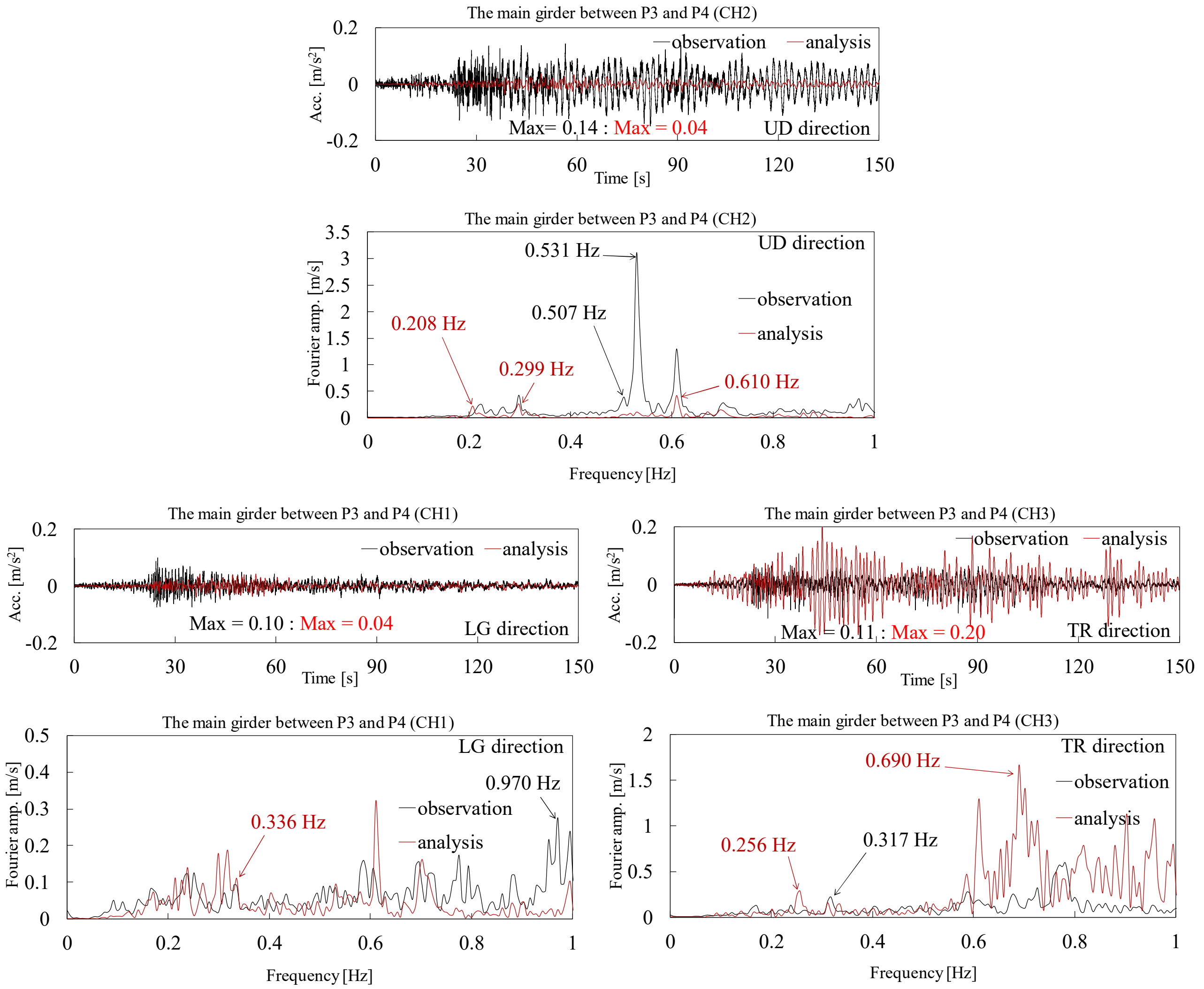
Fig. 6 compares the dynamic analysis results with the observational records for the top of the P3 main tower and for the main girder between P3 and P4.

The maximum value of the analysed data in the LG direction for the top of the P3 main tower was 0.10 m/s^2, which is less than half that of CH19 (LG direction). By contrast, the maximum value of the analysed data in the TR direction for the top of the P3 main tower was 0.25 m/s^2, which is nearly equal to that of CH20 (TR direction). In the Fourier analysis of the analysed data for the top of the P3 main tower in the LG direction, three peaks appear in the frequency range ~0.2–0.35 Hz, corresponding to the natural frequencies of the first symmetric bending mode (0.208 Hz, T_1 = 4.811 s), the first asymmetric bending mode (0.289 Hz, T_3 = 3.455 s) and the first swing mode of a girder (0.353 Hz, T_4 = 2.831 s) according to eigenvalue analysis. In addition, two peaks appear in the frequency range ~0.5–0.6 Hz, corresponding to the natural frequencies of the second symmetric bending mode (0.512 Hz, T_6 = 1.953 s) and the second asymmetric bending mode (0.603 Hz, T_7 = 1.658 s) according to eigenvalue analysis. Moreover, the frequencies showing the peaks show good agreement between analytical and observational values. In the Fourier analysis of the analysed data for the top of the P3 main tower in the TR direction, one peak appears near 0.25 Hz, corresponding to the natural frequency of the first lateral symmetric bending mode of a girder (0.255 Hz, T_2 = 3.925 s) according to eigenvalue analysis. Furthermore, the first bending modes of the P2 and P3 main towers were observed near 0.7 Hz according to eigenvalue analysis.

The maximum value of the analysed data in the LG direction for the main girder between P3 and P4 was 0.04 m/s^2, which is about half than that of CH1 (LG direction). In the Fourier analysis of the data, one peak appears near 0.35 Hz, corresponding to the natural frequency of the first swing mode of a girder (0.353 Hz, T_4 = 2.831 s) according to eigenvalue analysis, in good agreement with observational data. It is presumed that stimulation of the first swing mode of the girder by this ground motion was small. On the other hand, the maximum value of the analysed data in the TR direction at the main girder between P3 and P4 shows a larger value of 0.2 m/s^2, which is about twice that of CH3 (TR direction). In the Fourier analysis of the data, ~0.25 Hz peaks correspond to the natural frequency of the first lateral symmetric bending mode of a girder (0.255 Hz, T_2 = 3.925 s) according to eigenvalue analysis, but the peak appears at low frequencies compared with the observed peak (0.317 Hz). Furthermore, the first bending modes of the P2 and P3 main towers were observed near 0.7 Hz according to eigenvalue analysis.



a. The top of the P3 main tower



b. The main girder between P3 and P4

Fig. 6 – Comparison of observation records and dynamic analysis results

For the most important UD direction data at the main girder between P3 and P4, the maximum value is 0.04 m/s^2, which is ~30% smaller than that of CH2 (UD direction). The Fourier amplitudes show two peaks in the frequency range ~0.2–0.3 Hz, corresponding to the natural frequency of the first symmetric bending mode (0.208 Hz, T_1 = 4.811 s) and the first asymmetric bending mode (0.289 Hz, T_3 = 3.455 s) according to eigenvalue analysis, indicating good agreement with observational values. In the eigenvalue analysis, the first torsional mode (0.500 Hz, T_5 = 2.001 s) and the second symmetric bending mode (0.512 Hz, T_6 = 1.953 s) were not excited, and one peak appears near 0.6 Hz, corresponding to the natural frequency of the second asymmetric bending mode (0.603 Hz, T_7 = 1.658 s).

6. Summary

A three-dimensional nonlinear frame model of the Tsurumi Tsubasa Bridge was constructed for dynamic analysis, whose results were compared with those of seismic records of aftershocks of the 2011 off the Pacific coast of Tohoku earthquake . It was clarified based on the modal analysis that the dominant vibration mode in the LG direction was the first swing mode of a girder (0.353 Hz, T_4 = 2.831 s). In the TR direction, the dominant vibration mode is the first lateral symmetric bending mode of a girder (0.255 Hz, T_2 = 3.925 s), and in the UD direction, the dominant vibration modes were the first symmetric bending mode (0.208 Hz, T_1 = 4.811 s) and the first torsional mode (0.500 Hz, T_5 = 2.001 s). The seismic response characteristics of the nonlinear frame model of the Tsurumi Tsubasa Bridge gave good agreement with the results from observed time series waveforms and Fourier amplitudes.

7. Acknowledgements

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8. References

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