DYNAMIC CHARACTERISTICS OF A LONG-PERIOD, CABLE-STAYED BRIDGE INFRASTRUCTURE SUBJECTED TO LONG-PERIOD GROUND MOTIONS

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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 [1] in Tokyo Bay at 2:56:18 am on 13 March 2011. In a previous study, Yamaguchi *et al.* [2] verified the dynamic characteristics of the Tsurumi Tsubasa Bridge by carrying out a series of vibration tests and analyzing the data. Yamamoto *et al.* [3] used observational records of the 2004 Niigata-ken Chuetsu earthquake to clarify the nonlinear seismic behaviors 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 [4,5] identified the structural parameters of the Yokohama Bay Bridge and the Katsushika Harp Bridge using observational seismic records. Shoji *et al.* [6] verified the modal characteristics of the two towers in the Tsurumi Tsubasa Bridge by the Fourier spectrum analysis using observational records of the aftershocks of the 2011 off the Pacific coast of Tohoku earthquake.

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 analyzed seismic responses with respect to the observed data [7]. Specifically, this study discusses the coupling vibration between different modes of the bridge when the stiffness of the dominant structural parameter of the bridge is varied for a long-period structural system composed of linear and nonlinear elements [8], subjected to long-period ground motions estimated from those of the Great Kanto megathrust earthquakes [9].

2. TARGET BRIDGE AND SEISMIC RECORDS

The Tsurumi Tsubasa Bridge is a three-span, continuous steel cable-stayed bridge located in Tokyo Bay. 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 (LG) direction and the transverse (TR) direction. The horizontal bearings in the main tower and end pier are also free in the LG direction but fixed in the TR direction. The pendel bearings support motion in the vertical (UD) direction and the LG direction. An elastic restraint cable is used to connect the main tower with the main girder in the LG direction.

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 the bridge's construction material [1] and a previous study [3], a three-dimensional nonlinear frame model was constructed. 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 the bridge's construction material [1]. 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.

The damping matrix in the equation of motion was assumed to be Rayleigh damping. The ground motion was implemented in the model as a multi-variant input at each foundation. The Newmark β method, with $\beta = 0.25$, was used for numerical integration, with an interval of 0.01 s.

4. MODAL ANALYSIS RESULTS

The results from the modal analysis were compared with those obtained in previous studies [2,3,6] and with the results computed by Metropolitan Expressway Company Limited (MEX) after seismic retrofitting.

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 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 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 LG girder motion, the first swing mode (T_4 = 2.831 s) was significant.

5. SEISMIC REPONSE CHARACTERISTICS

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 of the 2011 off the Pacific coast of Tohoku earthquake at 2:56:18 am on 13 March 2011.

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.

6. NONLINEAR RESPONSE INTERACTIONS

The associated interactions of the nonlinear structural elements and the resulting sensitivity of the main linear structural elements of the bridge to long-period ground motions estimated from those of the anticipated Great Kanto megathrust earthquakes were examined.

When the axial stiffness of the elastic restraint cable which is the dominant structural parameter for the coupling vibration characteristics of the bridge was -50%, the first swing mode of a girder approached the first lateral symmetric bending mode. This resulted in a decreased amplitude in the LG direction and an increased amplitude in the UD direction of the first swing mode of the girder. The response around the TR axis of the P3 main tower pier increased. Response in the LG direction and around the LG axis of the foundation-to-ground springs, which also have different material nonlinearities, increased. The damping force of the vane damper, which is a nonlinear element with nonlinear viscosity, decreased. The maximum displacement of the stopper, which is a nonlinear element with phenomenological nonlinearity like pounding between the elements, increased. These interactions increase the response in the UD direction on the P3-P4 deck and in the LG and UD directions on the centre span deck. The inside cable is more related to the response of the centre span deck, while the outside cable is more related to the response of the deck from P3-P4.

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