Analysis of Tower-Girder Transfer Pounding on Yokohama-Bay Cable-Stayed Bridge during the 2011 Great East Japan Earthquake

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Summary

This paper presents an analysis on the transverse pounding based on seismic records of the Yokohama-Bay Bridge during the March 11, 2011 earthquake. Pounding mechanism involves wind-shoe, side bearing, and tower-link. The pounding mechanism is modelled by a simplified structural model using contact spring to simulate the contact force between tower and girder at wind-shoe. The model is verified by comparing the simulated and identified time-history responses, spectra, natural frequencies and shapes of the transverse modes. Using the verified model, the amount of pounding force is quantified and the effect of pounding force on wind-shoe is investigated in more detailed. The study also investigates transverse pounding force effect on the structure in general for earthquakes larger than the 2011 Great-East Japan earthquake.

Keywords: cable-stayed bridge, seismic response, transverse pounding.

Yokohama-Bay Bridge links Tokyo to Yokohama Harbor area as a part of Tokyo-Yokohama Bay shore expressway. The bridge is a continuous double-deck three-span cable-stayed with a total span of 860m (200m-460m-200m). The girder consists of steel truss-box with double-deck: the upper deck for six-lane Yokohama Expressway Bay shore route and the lower deck for the two-lane national route. The bridge has two H-shaped towers 172 m tall and 29.25 m wide welded as monolithic section. The bridge was opened in September 1989 and in 2005 a seismic retrofit program for safety assurance of level-2 earthquake according to Japan's bridge seismic code was conducted. On March 11, 2011, The Great East Japan (Tohoku) earthquake hit northeastern Japan with magnitude of Mw 9.0, notably the largest earthquake in Japan's modern history. Intensity 5+ (PGA 1.4-2.5 m/s2) out of the maximum scale of 7 JMA's seismic intensity was recorded on the bridge site. Seismic responses during the earthquake show the dominance of transverse vibration on girder and tower with the maximum girder displacement of 62 cm.

Transverse pounding between tower or pier and girder was not considered in design. In design, the tower-girder and pier-girder transverse connections were modelled as connected springs and the effect of transverse gaps were neglected. In this analysis tower-girder and pier-girder lateral connections are modelled by contact springs. The bridge is modelled by a simplified flexural element for tower, pier and girder. As shown in Fig.1, the transverse pounding was caused mainly by transverse motion in the first transverse frequency. Therefore, in the analysis we focus on the motion in transverse direction only. In the pounding model, contribution of the cable to the response is assumed to be significant in vertical and longitudinal direction, whereas in transverse direction it is assumed to be small. Mass and stiffness of girder, pier and tower were computed using a reference model provided by design documents (Fig.2). Locations of lumped mass in the model were selected so that they correspond to the locations of vibration sensors installed on the actual bridge. In that way, the simulated responses can be compared directly to the recorded ones. In the model, pounding is modelled as two-side contact problem between the nodes that correspond to tower and girder at the connecting points (i.e. location of wind shoes). Variable (δ) defines two possible conditions of the nodes, namely, during non-contact condition ($\delta \neq 0$) and during contact condition $\delta = 0$.

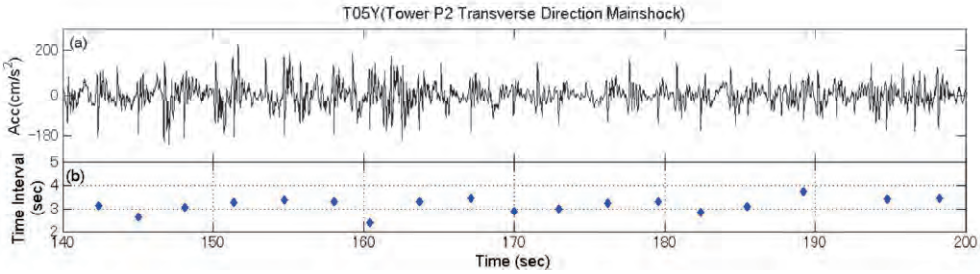

Fig.1. Close-up look at the tower transverse (in-plane) acceleration at the tower-to-girder connections shows(a) periodic impulse response for and (b)Time interval between consecutive impulses during

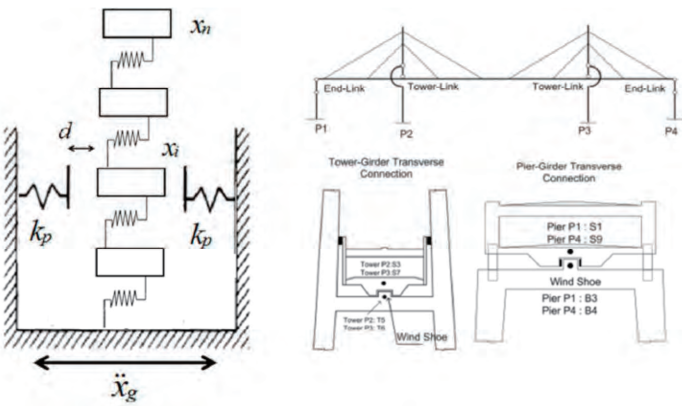

Fig.2. Schematic figure of pounding model at tower-girder and pier-girder connections

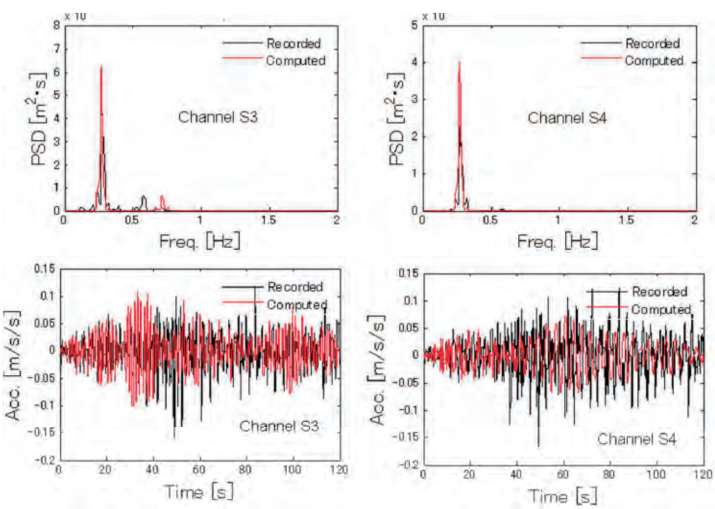

Fig.3. Comparisons between model and measured acceleration responses and spectra for two locations on the wind shoes on P2 (S3) and P3(S4)

Comparison of simulated and measured responses are shown in Fig. 3 for spectra of accelerations recorded on sensor (S3 and S4) that are located on the wind shoes of tower P2 and P3, respectively. The figures show that the model can capture the dominant peaks associated with the bridge first transverse mode. In general, the model can represent vibration characteristics in transverse direction very well as shown by the results in natural frequency, spectra, root-mean squared of time histories and mode-shape of the first transverse mode. The discrepancies are thought to be related to the limitation of the model in representing the contribution of stiffness from the cables since the effects of cables in transverse direction is neglected, hence the model are in general was more flexible. Nevertheless, since overall behaviour of transverse vibration is dominated by the first transverse mode and the pounding occurrence is strongly related to the first transverse mode, the model is considered to be reasonably adequate to represent the transverse behaviour and transverse pounding of the bridge.