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Clarification of the Damage Mechanism of the Long-Period Bridge System Damaged by the 2016 Kumamoto Earthquake

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The purpose of this study is to clarify the damage mechanism of a long-period bridge structure-underground interconnected system, such as the Ohkirihata Bridge damaged in the 2016 Kumamoto earthquake, subjected to the combined loads of long-period pulsive ground motions, surface fault displacements, and ground deformation by slope failures.

Firstly, the site-specific waveforms were estimated by the finite difference method as input ground accelerations to a bridge system model. Secondly, the target bridge system was modeled by three dimensional (3D) finite element (FE) method with 441,196 nodes and 2,437,126 solid elements. The target bridge system is the affected Ohkirihata Bridge which is a 5-span continuous steel girder bridge with the length of 265.4 m long and the width of 12.5 m wide. For the 3D FE model, the reinforced concrete (RC) slabs and steel girders were idealised by equivalent tetrahedral first-order elements with Young's modulus and density of mass of parallel serises of slabs and girders. The five laminated rubber bearings with total rubber thickness of 85 mm -150 mm are placed upon the top of each pier: A1Bs, P1Bs-P4Bs, A2Bs. The layers of natural rubbers and steel plates consisting of a bearing were modeled by hexahedral first-order elements with same plain size and different heights. The RC piers P1-P4 and RC abutments A1, A2 were modeled by tetrahedral first-order elements. The RC foundations of caisson piles supporting P1, P4 and A2 and cast-in-place piles supporting A1, P2 and P3 were modeled by tetrahedral first-order elements. The underground layers consist of nine different soil materials which mechanical properties were identified by soil tests. These shallowed layers were modeled by tetrahedral first-order elements. The minimum sizes of all tetrahedral first-order elements were set as about 1m in average. The stress-strain relationships set for all elements were assumed to behave isotropically in the elastic ranges.

We used a open-source program for large-scale parallel computation of FE analysis: FrontISTR. Linear dynamic analysis was performed by applying the site-specific waveforms of the main ground motions for 10 seconds as equivalent inertial forces to all nodes of the 3D FE model. The Newmark-β method (β=0.25, γ=0.5) was used in the time integration scheme. The time interval was set to be 0.005 sec, and the number of computational steps was 2000. Damping effect on equations of motion was formulated by stiffness-proportional damping matrix. The CG method was applied for the numerical linearization, and the SSOR preprocessing was used. The threshold for determining the convergence about relative error of computed displacement was set at 1.0×10^{-6}.

P3B, P2B, P4B, A2B, and P1B, in that transient order, exceed allowable shear strain of 250% in the 3.9 seconds before the maximum amplitudes of the site-specific EW and NS waveforms, and A1B follows to exceed 250% after the peak. The time-series horizontal motions of the superstructure supported by these bearings until the 3.9 seconds are demonstrated as first, the superstructure moves to the northwest direction between 1.86 s and 2.49 s, next to the southeast direction between 2.49 s and 2.82 s with a slight clockwise rotation around the A1, and further to the north direction between 2.82 s and 3.24 s with a counterclockwise rotation around the A1. The reasons are why the excitation of long-period pulsive ground motions to the primary eigenmode of the target bridge system in the 1.2 s to 1.5 s natural period causes these processes.