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Effects of vertical ground motions on seismic vulnerabilities of a continuous track-bridge system of high-speed railway

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ABSTRACT

A continuous bridge in high-speed railway is close to several known faults in China. Those faults, respectively at different distances from the bridge site, will produce different ground motions with the different ratios of vertical component to the horizontal component at the bridge site. It is necessary to identify the influence of vertical ground motions on the seismic responses and vulnerabilities of the track-bridge system. This paper solved this problem by carrying out an incremental dynamic analysis (IDA) and a further seismic fragility analysis on a widely used continuous bridge in China. The results show that the damage probabilities of most bridge and track components increase along with the increase of vertical part in ground motions. This trend is significant for the sliding layer of track part in the longitudinal direction and the piers of bridge part in any direction, however, insignificant for the bearings of bridge part in any direction. Moreover, this trend is more significant for the track part across the girder gap due to the different seismic responses of adjacent bridges. The seismic design of track-bridge system should rigorously take the vertical part of ground motions into account.

1. Introduction

Chinese high-speed railway is developing rapidly now, and the track structure is usually constructed on the bridge structure to ensure rail smoothness, improve space utilization, avoid other transportation interferences, etc. Some of those high-speed railway bridges are at the site surrounded by several known faults, since a lot of people live in the neighborhood and need the high-speed railway. Those bridges will be shocked by the horizontal ground motions and the vertical counterparts, simultaneously transmitted from different faults at different distances from the bridge site. It is necessary to identify the influence of those vertical ground motions on the seismic responses and vulnerabilities of those track-bridge systems, which determine the structural and traffic safety.

In addition to the major studies on the seismic responses under horizontal ground motions, a very few researches validated that the vertical ground motions led to two types of failure modes in columns at least [1]: (1) shear failures due to the reduction of shear capacity and ductility supply influenced by changing the axial force; (2) compressive failures, such as the buckling of longitudinal reinforcement and the crushing of concrete due to the significant increase of axial force caused by the vertical part of ground motion and the overturning moment. For example, Kunnath et al. [2] conducted a nonlinear time history analyses on a typical highway bridge under the horizontal and vertical ground motions, and found that the vertical part of ground motions caused an obvious increase of axial force in piers and moment in girders. Rahai [3] modeled piers with 3D solid elements, and validated that the shear and axial strains increased significantly due to the vertical ground motions. Veletzos and Restrepo [4] developed a detailed finite element model for the popular precast segmental bridge, and revealed that the vertical ground motion urged the segmental joints to crack and into the nonlinear range. Wilson et al. [5] further put forward that the skew and curved bridges were more vulnerable than the straight bridge when subjected to the vertical ground motions.

The ratio of vertical component to its horizontal counterpart in the designed ground motions is a debatable point in the criteria of many countries and in the studies of many researchers, due to the lack of adequate theoretical and experimental verifications on the vertical ground motion mechanics and its influence on the structural seismic responses. The previous seismic design guidelines in California [6] adopted 25% of the bridge dead load to represent the equivalent vertical load of vertical ground motion, only when the bridge was located

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at a site with a designed horizontal peak ground acceleration (PGA) being greater than 0.6 g. Gülerce and Abrahamson [7] carried out a seismic assessment on a single-bent, two-span highway bridge designed according to the above seismic design guidelines, and found that the probabilities exceeding the elastic limit state of negative bending moment at mid-span are about 40% at rock sites and 70% at soil sites by considering the vertical ground motion, however, less than 10% without considering the vertical ground motion under earthquakes with a horizontal PGA of 0.6 g. Newmark et al. [8] previously proposed that the vertical component was 2/3 of its horizontal counterpart in the designed ground motions of long-span structures, which was subsequently accepted by EUROCODE-8. However, the site measurements of previous extreme earthquakes showed that the PGA of vertical component would be close to and even exceed the horizontal PGA [9,10]. For example, the vertical PGA of Ichinoseki-nishi earthquake reached up to 4 times of gravitational acceleration. Li et al. [11] conducted a statistical analysis on 130 sets of ground motions with an earthquake magnitude range from 4.5 to 8 and a fault distance within 15 km, and revealed that the ratios of vertical component to the horizontal component were much larger than 2/3 within periods being less than 0.1 s and then decreased to be less than 2/3 within long periods for the earthquake spectrum due to the different propagation mechanics of P and S ground motion waves. The ratio of vertical ground motion to its horizontal counterpart decreased with an increase of fault distance [12], and was also affected by the earthquake magnitude and site condition [11].

Different numerical methods were advanced to investigate the effects of the vertical part of ground motions on the structural seismic responses. Collier and Elnashai [12] proposed a simplified procedure to obtain the structural response under the combination of the horizontal ground motion and its vertical counterpart, and used modification factors to reflect the effects of the PGA of horizontal component and vertical component, however, with some limitations. Button et al. [13] put forward a dead load multiplier, determined by the magnitude, fault distance and other parameters, to envelop the responses of six typical highway bridges under different spatial earthquakes, however, with the overestimated or underestimated errors. Warn and Whittaker [14] confirmed that the direct summation of the peak vertical axial forces, respectively induced by the vertical ground motion and overturning moment, overestimated the actual axial force of bearing when compared with a simulation test of a steel truss bridge, because those two peak values hardly occurred simultaneously. Any simplified methods could predict the seismic demands of structures, however, couldn't reflect the fluctuations of moment capacity, shear capacity and ductility supply under the interactions of horizontal and vertical ground motion excitations. Wang et al. [15] used a numerical simulation to obtain the time histories of bending moment capacity and shear capacity, and revealed that those capacities varied with high frequency and considerable amplification when considering the influence of vertical ground motions. It would cause the premature failure or the absolutely different failure modes, and impede the implementation of traditional capacity design method. Hosseinzadeh et al. [16] found that the ductile bending failure mode of piers changed to the brittle shear failure mode due to the significantly fluctuating axial force and even the existence of tension in the piers under the near-field earthquakes. This phenomenon had been observed by Lee and Mosalam [17] when they conducted a shaking table experiment of a scaled bridge model under spatial earthquakes. And the shear-displacement hysteresis loop derived from variable normal force model was extremely unstable and asymmetric since the axial force changed at a much higher frequency than the lateral force [18]. It implied that the time interval between the peak accelerations of horizontal and vertical motions had a significant effect on the shear capacity of piers, and Kim et al. [19] predicted that the occurrence of shear failure was random by considering the vertical part of ground motions. Hence, it would be better to use the rigorous time history analysis method to assess the structural seismic responses under spatial earthquakes.

The new concepts of probabilistic seismic demand model and seismic hazard assessment procedure were developed to consider the strong randomness of ground motions and the variability of structural parameters. Gülerce and Abrahamson [7] used those new concepts to compare the probabilities exceeding the elastic limit state of piers and girders in bridge structures under different ground motions, and validated that an increase of vertical part in ground motions significantly increased those probabilities. The ratio of vertical ground motion to its horizontal counterpart should be considered as the intensity measurements (IM) of ground motions to build the probabilistic seismic demand model [20]. Wang et al. [21] used a vulnerability model to get the conclusion that the vertical ground motion had a considerable impact on the failure probabilities of fixed bearings but had minor influence on that of expansion bearings and piles. And the influence of the vertical part of ground motions on the seismic vulnerabilities of piers depended on the designed axial compression ratio of piers.

Although the above structural seismic responses are sensitive to the vertical part of ground motions, there are few corresponding studies on the seismic responses and vulnerabilities of high-speed railway bridges, especially considering track-bridge interactions [22]. The seismic damage of track structure, such as the continuous ballastless China Railway Track Slab II (CRTSII), should be paid more attention, because it influences both the bridge damage and the traffic safety [23] during and after earthquakes. Therefore, this paper numerically identifies the influence of vertical ground motions on the seismic responses and vulnerabilities of track-bridge system, including a continuous track structure CRTS II and a continuous bridge in a high-speed railway, recommended in Chinese criterion [24–26]. The analysis results can be applied to the improvement of the current criterion in China [27,28] and other countries.

2. Bridge structure

2.1. Bridge introduction

Fig. 1 shows a (48 + 80 + 48) m continuous bridge widely used in Chinese high-speed railway [24–26]. The bottom line of concrete box girder is quadratic parabola, and is supported by eight spherical steel bearings in Fig. 2 on the top of four piers. The heights of those 1#–4# piers are 13, 13, 13 and 12 m, respectively. The longitudinal and transverse lengths of the rectangular sections are 4.2 and 8.6 m, respectively, for the 2# and 3# piers. Likewise, the cross section sizes are 3.4 and 7.6 m, respectively, for the 1# and 4# piers. Moreover, each of 2# and 3# piers is supported by 20 Φ1.5 m circular piles, while each of 1# and 4# piers is founded on 16 Φ1.25 m circular piles.

There is the track structure CRTS II having many components in Fig. 3 on the girder [24–26]. The base plate, being continuous across girder gaps, slides on the girder under earthquakes by using the siding layer with a friction coefficient of 0.2. However, one part of base plate is fixed at the girder point on the top of fixed bearings by shear teeth, which are composed of 3 grooves, 3 side walls and 14 HRB335 shear studs with a diameter of 28 mm. The similarly continuous track plate is connected on the base plate by using the concrete and visco-elastic asphalt (CA) layer with a shear capacity of 415 kN per 6.45 m. However, the parts of track plate at the girder ends are connected on the base plate by the shear bars, which are composed of 2 rows and 8 HRB500 steel bars with a diameter of 28 mm. The track plate is transversely and vertically restricted by many lateral blocks on the girder, with a hard foam material and a rubber material in the restriction gaps. The rails are connected on the track plate by using a kind of WJ-8C fastener with a longitudinal failure force of 15.0 kN.

2.2. Finite element model (FEM)

Fig. 4 shows a FEM model of the above track-bridge system, built by