Bridge Damage Caused by the 2011 Great East Japan Earthquake

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Summary

This paper presents damage of bridges during the 2011 Great East Japan earthquake. Effectiveness of recent development of seismic design and implementation of seismic retrofit is evaluated based on comparison of the damage during the 2011 Great East Japan earthquake and the 1978 Miyagi-ken-oki earthquake. Tsunami-induced damage is also presented for bridges along the Pacific Coast. Typical feature of tsunami-induced damage is presented based on a field investigation.

Keywords: Great East Japan (Tohoku) earthquake, seismic damage, bridges, ground motions, tsunami, seismic design codes.

1. Introduction

The Great East Japan earthquake (M_w9.0) occurred at 14:46 (local time) on March 11, 2011 along the Japan Trough in the Pacific Ocean. This was the largest earthquake which ever historically occurred around Japan. The fault zone extended 450km and 200km in the north-south and west-east directions, respectively, as shown in Fig. 1. Extensive damage occurred in a wide region in the east Japan.

A number of strong motion accelerations were recorded in the damaged areas by the National Institute of Earth Science and Disaster Prevention (NIED) and Japan Meteorological Agency (JMA). Most accelerations were recorded at stiff sites as it was the purpose of NIED and JMA to record base-rock accelerations. Fig. 2 shows typical acceleration records along the Pacific Coast. Ground accelerations continued over 200s, and they had at least two wave groups reflecting the fault rupture process. The highest peak ground acceleration of 27.0 m/s^2 was recorded at Tsukidate City. However the high acceleration was resulted from a single pulse with 0.2 second period, therefore response acceleration at 1.0s was only 5.1m/s^2 as shown in Fig. 3. Consequently damage of bridges and buildings was very minor in Tsukidate City. This clearly shows that only PGA cannot be a reliable index for seismic design.

At soft soil sites in the north Miyagi-ken and south Iwate-ken (refer to Fig. 1), ground accelerations included longer period components leading to higher response accelerations at 0.5-1.5s. For example, response acceleration was over 16 m/s^2 at 0.8 s in Osaki as shown in Fig. 4.

Seismic design code of bridges was extensively enhanced since 1990 (JRA 1990, 1995, 1996, 2002, 2012) [1,2,3]. Only a combination of a static elastic analysis assuming 0.2-0.3 seismic coefficient and an allowable stress design approach (seismic coefficient method) was used until 1990 (JRA 1964, 1971, 1980). The static elastic method is still in use today but a combination of an inelastic static analysis and Type I and II design ground motions as shown in Fig. 5 has been the main stream approach in the post-1990 codes. Thus the seismic demand was much enhanced in the post-1990 design codes.

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