DYNAMIC RESPONSE ANALYSIS OF BRIDGE WITH LEAD RUBBER BEARING SYSTEM UNDER 2011 OFF THE PACIFIC COAST OF TOHOKU EARTHQUAKE GROUND MOTIONS

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1. INTRODUCTION

A large scale earthquake with a magnitude of 9.0 (Mw) severely struck the north-eastern region of Japan on 11th March, 2011. The maximum peak ground acceleration (PGA) recorded from this earthquake reached 2700 gal (NS component) and the ground motion lasted around 300 seconds. This earthquake caused a dramatic tsunami and brought about massive damage to the infrastructure of the region, including road and railway traffic networks. The characteristics of the ground motions caused by the 2011 Off the Pacific Coast of Tohoku Earthquake (2011 Tohoku Earthquake) were significantly different from those specified in the current design specifications. Furthermore, there were some reports of damage to the bridges in the regions affected by the force of the 2011 Tohoku Earthquake.

In this work, the characteristics of ground motion were investigated and the seismic performance of a typical bridge with a Lead Rubber Bearing (LRB) system was verified by dynamic analysis method. As a result, the ground motions caused by this earthquake affected the seismic performance of bridges with LRB systems.

2. GROUND MOTIONS

According to the surface ground motion records of K-net and KiK-net, approximately 100 observation sites recorded that the combined PGAs (components of NS, EW and UD) were bigger than 400 gal. Among them, there were 52 observation sites where the surface ground were classified into Type I ground in seismic design. The response acceleration spectra, with a damping ratio of 5% of the main ground motions and Level 2 ground motions (Type1 and Type2) recommended by the current highway bridge seismic design specifications (seismic design ground motion) on type1 ground were shown in Fig. 1. The maximum response acceleration was 12884.7 gal at the period of 0.24 s calculated by I-01N (MYG004-175 km from the epicenter - NS component), while that calculated by the seismic design ground motion I-2-3 (Type 2) was 1832.6 gal at the same period. Their ratio was about 1.0: 7.0. At the period of 0.40 s, the response acceleration of I-03E (TCG014-297 km from the epicenter - EW component) was 5126.7 gal, while that of the seismic design ground motion I-2-1 was 2010.5 gal. Their ratio was around 1.0: 2.5. At the period of 0.54 s, the response acceleration of I-10E (TCGH13-282 km from the epicenter - EW component) was 2502.4 gal, while that of the seismic design ground motion I-2-1 was 2015.4 gal. Their ratio was about 1.0: 1.2. With the increase of the period, the response accelerations of the surface ground motions on type1 ground became smaller than that of the seismic design ground motions when the period was longer than 0.60 s, namely, comparing with the seismic design ground motions, the surface ground motions caused by the 2011 Tohoku Earthquake dominated at the periods shorter than 0.6 s. However, I-20E (IBR002 EW component) and I-06E (MYGH10) dominated around 1.0 s and 3.0s, respectively. Among the surface ground motions recorded, I-01N, I-10E and I-20E were assumed to mostly affect the seismic performance of the prototype bridge whose fundamental period was about from 0.9 s to 1.6 s, therefore, these ground motion components were selected as the ground motion input waves in this study as shown in Fig. 2. Furthermore, in order to compare with the results from the seismic design ground motions, I-1-3 and I-2-2 were also considered.

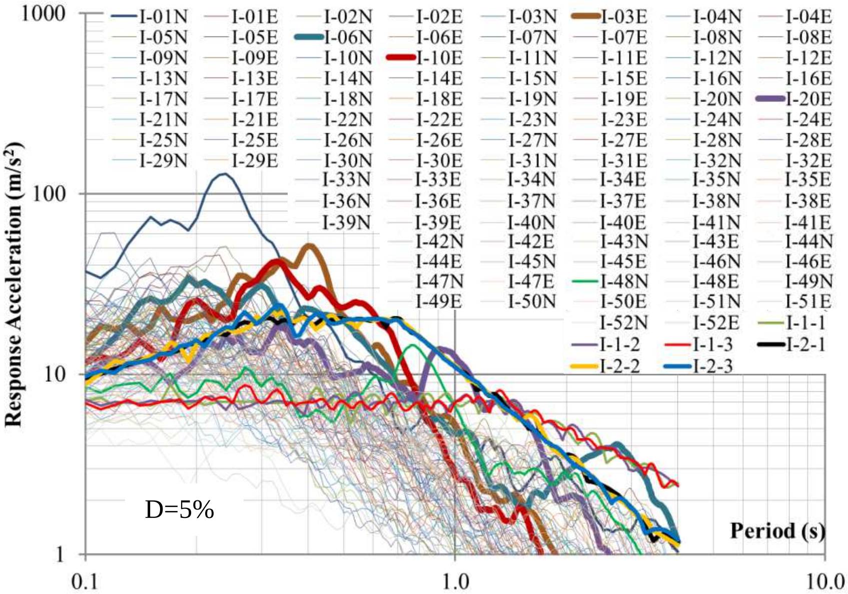

Fig. 1: Response Acceleration Spectrum of Main Ground Motions

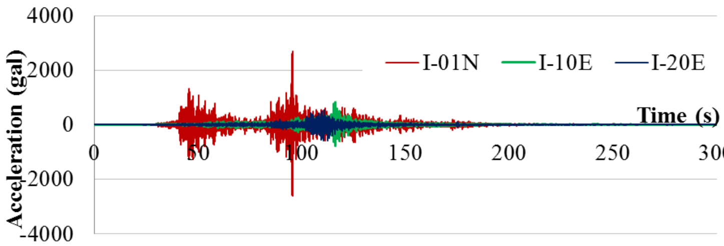

Fig. 2: Input Ground Motions

3. PROTOTYPE BRIDGE

As shown in Fig. 3, a typical highway bridge with LRB system demonstrated in “Materials for Seismic Design of Highway Bridges” was considered in this work that was calculated based on the highway

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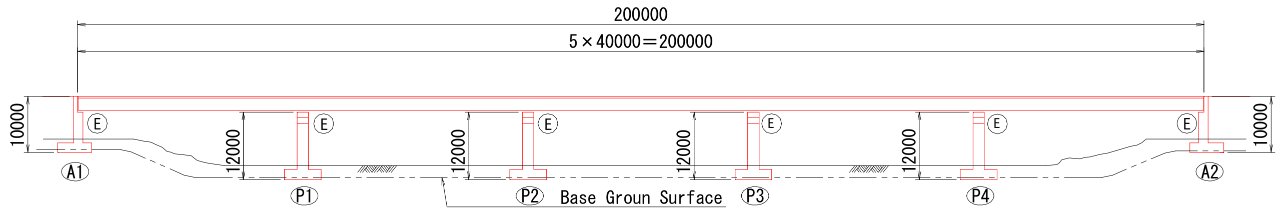

Fig. 3: Prototype Bridge

bridge design specifications from 1996. The bridge was a 5-span continuous steel plate girder bridge with a total length of 200 m. The superstructure was supported elastically both in longitudinal and transversal directions on all substructures except in transversal direction on both abutments and its weight was 31400 kN. The girder was fixed at the ends in transversal direction. The height of the pier columns was 10.0 m including the overhanging. The substructures were built of reinforced concrete. The rebar arrangement was shown as Fig. 4. The compressive strength of the concrete was 21 MPa and the yield strength of the rebar was 295 MPa. The LRB for abutment had an effective plane dimension of 450 mm in longitudinal and transversal direction width, and 160 mm (10 mm@16 layers) in depth, and that for the pier had an effective plane of 600 mm in width and 154 mm (14 mm@11 layers) in depth. There were 4 lead plugs with a diameter of 65 mm and 85 mm in a bearing for abutment and pier respectively.

4. ANALYSIS RESULTS AND CONSIDERATIONS

The response was calculated by nonlinear dynamic analysis method. As an example, the results of longitudinal direction were introduced. ***Response of superstructure:*** Fig. 5 shows the displacement of the superstructure calculated by the 2011 Tohoku Earthquake waves, the maximum displacements caused by I-01N, I-10E, I-20E, I-1-3 and I-2-2 were 0.120 m, 0.072 m, 0.170 m, 0.121 m and 0.175 m, respectively. I-20E caused the largest displacement among the 2011 Tohoku Earthquake waves, that was 140% and 97% of the ones cased caused by I-1-3 and I-2-2, respectively. ***Response of LRB***: Fig. 6 shows the P-δ hysteretic loop of P1 LRB. The displacements caused by I-1-3 and I-2-2 were 0.089 m and 0.168 m, respectively. I-20E caused the largest displacement, and that was 0.153 m. It was 179% and 91% of that caused by the I-1-3 and I-2-2, respectively. ***Response of plastic hinge***: the hysteretic loop of the M-θ of plastic hinge of P1 was shown in Fig. 7. The maximum rotational angles were 2.72 mrad, 0.65 mrad, 0.72 mrad, 0.61 mrad and 0.85 mrad calculated by I-01N, I-10E, I-20E, I-1-3 and I-2-2, respectively. Among the three 2011 Tohoku Earthquake waves, the I-01N caused the largest rotational angle, which was 446% and 320% of the ones caused by I-1-3 and I-2-2, respectively. Except the I-01N, all the other input waves kept the plastic hinge within crack state. The plastic hinge entered its yield state under the action of I-01N.

5. CONCLUSION

(1) the ground motions recorded on Type 1 ground had greater peak accelerations, and the response spectra of almost all of the records were distinguished when the period was shorter than 0.6 s, but there were a few ground motions whose response spectra were distinguished around 1.0 s, 3.0 s comparing with the seismic design waves; (2) the ground motions on Type1 ground calculated larger rotational angle of the plastic hinge of LRB bridge, that affected the seismic performance of LRB bridge.

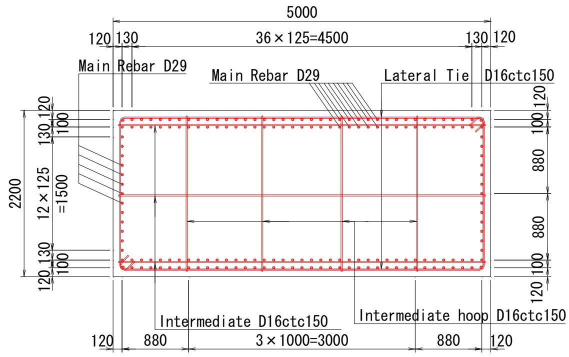

Fig. 4: Rebar Arrangement of the Pier Column

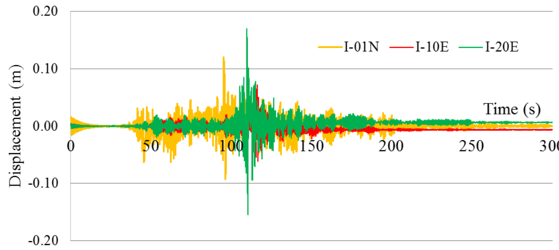

Fig. 5: Response Displacement of Superstructure

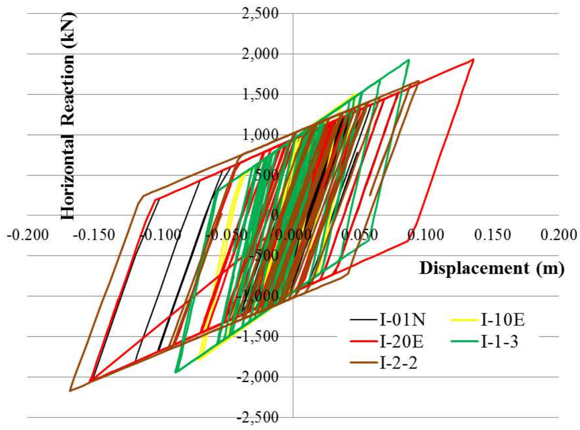

Fig. 6: P-δ Hysteretic Loop of P1 LRB

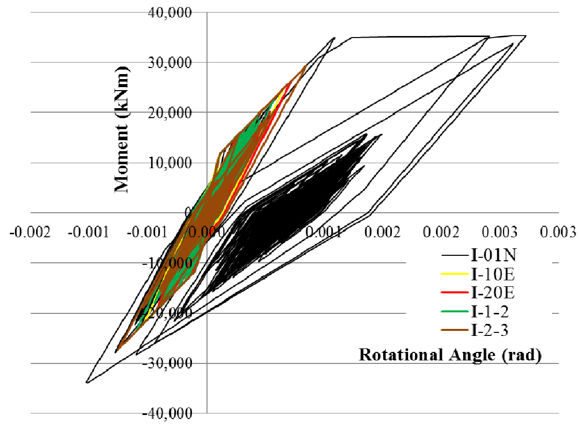

Fig. 7: M-θ Hysteretic Loop of P1 Plastic Hinge

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