NONLINEAR SEISMIC RESPONSE ANALYSIS OF HIGH-SPEED RAILWAY VIADUCTS CONSIDERING TRAIN LOAD

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ABSTRACT

Japan is located in an earthquake-prone region. Therefore, the earthquake-proof capacity of the Shinkansen system is always a concern considering the extremely high-speed of the bullet trains. It is important to examine the seismic performance of the existing bridge structures to ensure the safety of both the bridges and running trains. In the Seismic Design Code for Railway Structures in Japan, the live load of trains is treated as additional mass to the bridge structure, which needs further discussions to ensure both the safety and economical demands. In this paper, as a preliminary step of this research, linear and nonlinear seismic response analyses of a typical portal rigid frame reinforced concrete Shinkansen viaduct are analyzed and evaluated by using a general program. The bridge seismic response of the case considering the train as additional mass is compared with that of the case considering no train load to investigate the influence of train mass on the nonlinear response of the bridge. In addition, the influence of different ground properties on the bridge seismic response is also examined.

Keywords: Shinkansen viaducts, seismic design, nonlinear seismic response analysis, train load.

1. INTRODUCTION

Japan's high-speed railway system, the Shinkansen, serves a vital role in the transportation network that connects its major cities. It has contributed importantly to the economic and social development of Japanese society since 1964. To this day, the transportation capacities of the high-speed railway system have been continually augmented through the use of new train models and improved equipment along with the increased number of trains. The high-speed railway's lines usually pass directly over densely populated urban areas, where the railway structure mainly comprises elevated bridges. On the other hand, Japan is located in an earthquake-prone region. Therefore, the earthquake-proof capacity of Shinkansen system is always a concern considering the extremely high-speed of the bullet trains. The Kobe earthquake, which occurred on Jan. 17, 1995, brought severe damage in Japan's Hyogo Prefecture. The traffic networks of Kansai area were entirely

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paralyzed. The railway viaducts were also collapsed by the earthquake. The bridge structures, especially the piers, were damaged severely in a wide area. Then, during the Chuetsu earthquake on Oct. 23, 2004, which was the strongest since the 1995 Kobe earthquake, the Shinkansen viaducts sustained severe damages and the first derailment accident of the bullet train occurred. Although fortunately no human lives were lost, the importance of earthquake-proof capacity of the Shinkansen system was recognized anew after this disaster. It is important to examine the seismic performance of the existing bridge structures to ensure the safety of both the bridges and running trains.

In the Seismic Design Code for Railway Structures in Japan (RTRI 1999), the live load of trains is merely attached as an additional mass to the bridge structure. However, the seismic response of the bridges under dynamic running train loads is very complicated (Kawatani et al. 2009), which needs further investigations to improve the design code to meet both safety and economical demands. On the other hand, very few researches have been conducted especially on the nonlinear seismic response analysis of railway bridges considering dynamic train loads, because of the extremely difficulties of the problem (Matsumoto et al. 2007). Therefore, the final goal of this research is to establish a numerical approach to simulate the nonlinear seismic response of the running train-bridge coupled system.

In this paper, as a preliminary step towards the above final goal, linear and nonlinear seismic response analyses of general portal rigid frame reinforced concrete Shinkansen viaducts are analyzed and evaluated by using a general program named MIDAS. The bridge seismic response of the case considering the train as additional mass is compared with that of the case considering no train load to investigate the influence of train mass on the nonlinear response of the bridge. In addition, the influence of different ground properties on the bridge seismic response is also examined. The seismic performance of the bridge is investigated by examining the cross-sectional forces of the pier compared with the strength limits.

2. SEISMIC RESPONSE ANALYSIS

Modeling the viaducts as three-dimensional (3D) finite elements, linear and nonlinear dynamic responses of the bridges are analyzed by using a general program named MIDAS, in which Newmark's β direct numerical integration method adopting 1/4 as the value of β is used to solve the dynamic differential equations.

2.1. Bridge model

A typical high-speed railway reinforced concrete viaduct in the form of a rigid portal frame shown in Figure 1 is adopted in this analysis. The viaducts are built with 24-meter bridge blocks. Each block consists of three 6 m-length spans and two 3 m cantilever parts, so called hanging parts, at each end. In this analysis, one block (24 m) of bridge is adopted as the analytical model. Figure 2 shows that the three-block bridge is modeled as 3D beam elements with 6 DOF at each node. The lumped mass system is adopted for the bridge beam elements. One car of the train is presumed to stand on the middle of the bridge. Then, it is converted into mass and attached to the structural nodes at the wheel positions. The bridge pier bottoms are set to be fixed. The damping constant of 0.05 is assumed for the first and second natural modes of the structure. For examining the seismic performance of the pier, the sectional view of the pier is shown in Figure 3. There are 12 axial reinforcing bars in the cross-section. Eigenvalue analysis of the bridge model is performed and the natural periods are shown in Table 1.

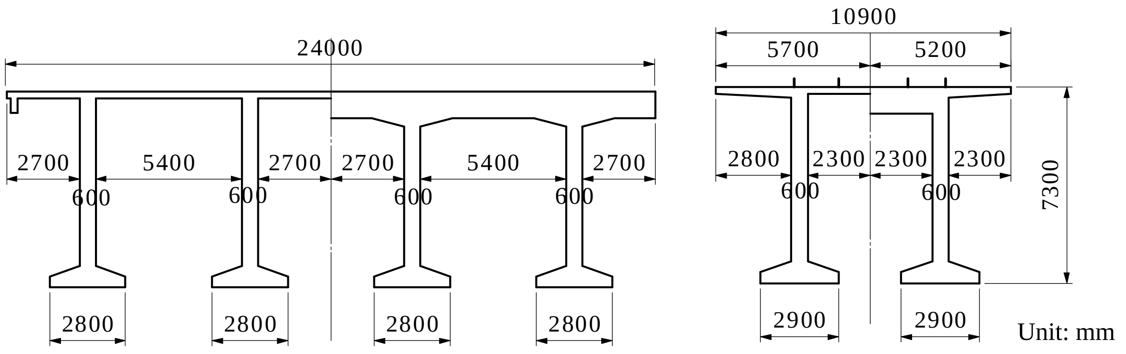
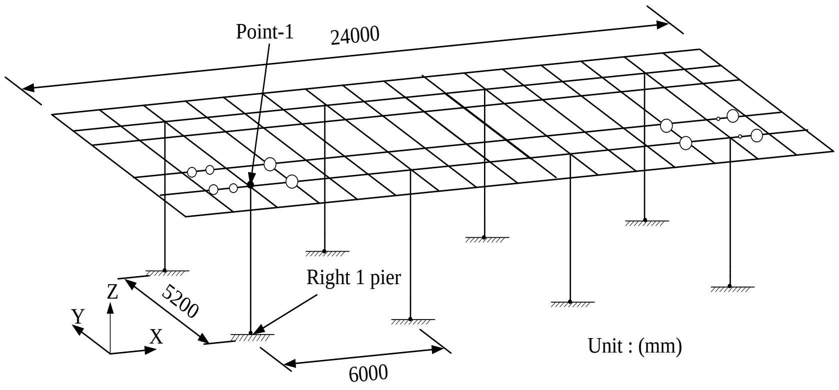


Figure 1: Dimension of the bridge.



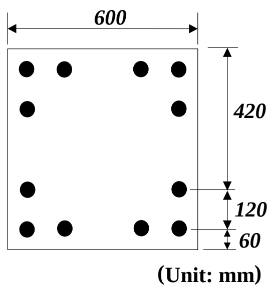


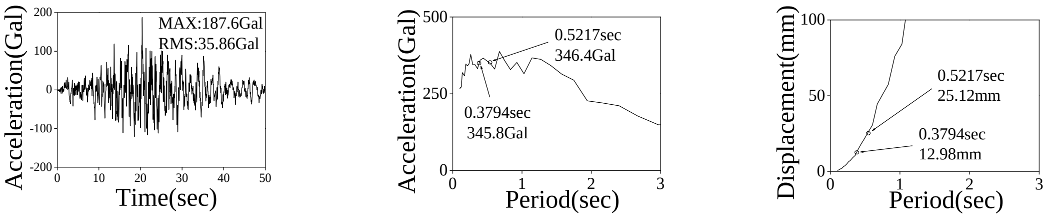
Figure 2: 3D FE model of the bridge. **Figure 3: Cross-sectional view of pier.**

Table 1: Natural periods of the bridge

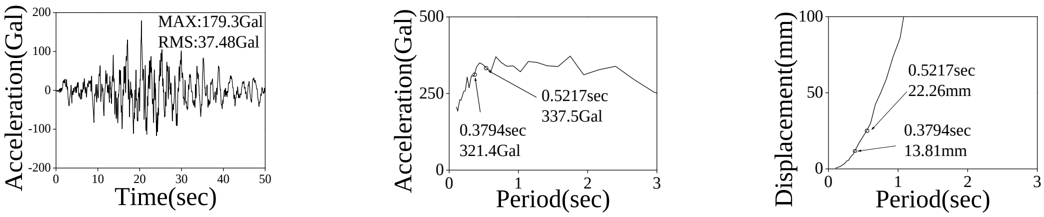
Mode	with no train load	with the train as additional mass	Mode shape
	period (sec)	period (sec)	
1st	0.3943	0.6024	x (the first logitudinal)
2nd	0.3794	0.5217	y (the first transverse)
3rd	0.3755	0.5073	x
4th	0.07365	0.1022	z (the first vertical)
5th	0.07259	0.1010	z

2.2. Adopted ground motions

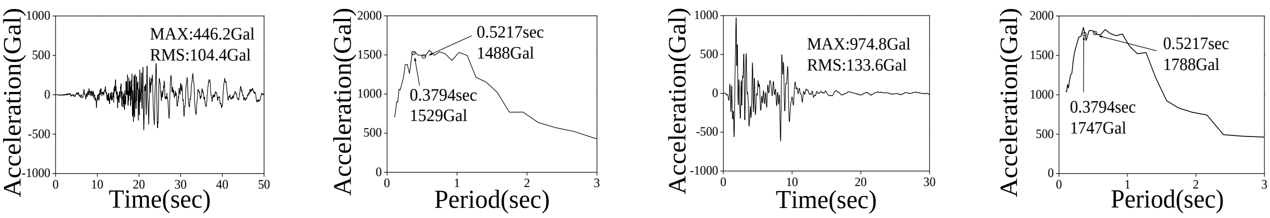
To investigate the seismic performance, the Level 1 and Level 2 ground motions defined in the Seismic Design Code for Railway Structures in Japan (RTRI 1999) are adopted as the seismic load. Figure 4 indicates the horizontal component simultaneously with its seismic response spectrum of the L1 and L2 ground motions designated in the design code. The vertical component of the design ground motion is not considered in this analysis. In the seismic response spectra, the period of 0.3794 second denotes the predominant horizontal natural period of the bridge considering no train load, while the period of 0.5217 second indicate that of the bridge considering the train as additional mass, respectively. Durations of 50 seconds for L1 and L2 Spc I ground motions and 30 seconds for L2 Spc II ground motion are adopted in the analysis, considering their peak values. For the viaducts adopted in this analysis, the ground properties are assumed as G3 and G6 soil, which have the natural periods of 0.25-0.5 second and 1.0-1.5 second in surface ground and are defined as normal and soft soil in the seismic design code. The ground motions used in this analysis are corresponding to the ground properties of G3 and G6 soil.



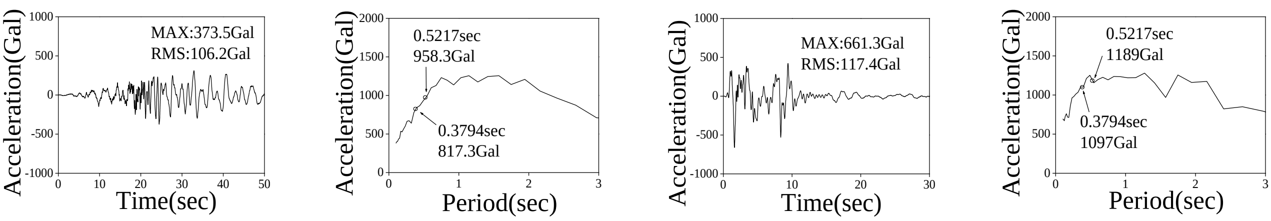
(a) Level 1_G3



(b) Level 1_G6



(c) Level 2_Spc Ⅰ _G3 (d) Level 2_Spc Ⅰ _G6



(e) Level 2_Spc Ⅱ _G3 (f) Level 2_Spc Ⅱ _G6

Figure 4: Ground motions (Time history and seismic response spectra).

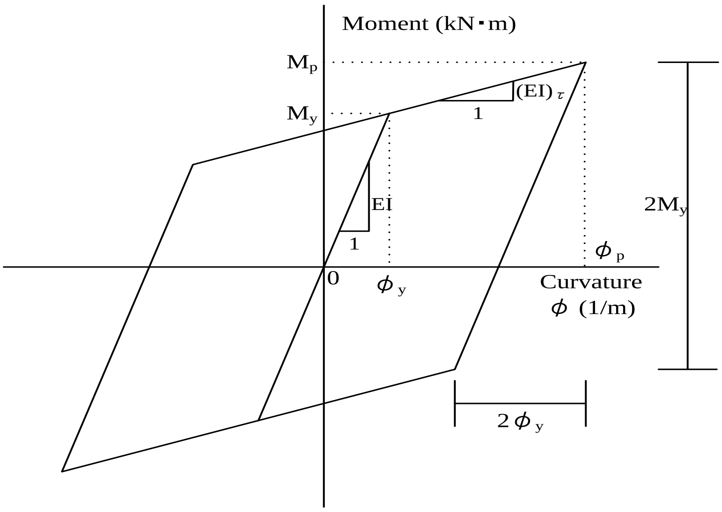


Figure 5: Hysteretic model of M- ϕ by-linear.

2.3. Restoring force model

In the nonlinear seismic response analysis, restoring force based on bi-linear hysteretic constitutive law as shown in Figure 5 is adopted for the element of RC bridge pier. The value of yield moment is set previously referring to previous studies (Kawatani et al. 2007), while the ultimate moment is not set in the current model.

3. NUMERICAL RESULTS

The examinations of seismic performance using the analytical models and the ground motions described above are carried out. To examine the effect of the train as additional mass on the seismic response of the bridge, two analytical cases are chosen for the seismic analyses. For both cases, only horizontal component of six ground motions are applied.

Case-1: Considering no train load.

Cass-2: Considering the train as additional mass to the bridge.

The effect of the train on the seismic response and performance of the bridge will be shown through these two cases.

3.1. Influence of train mass

Corresponding to the two cases described above, horizontal accelerations and displacements of Point-1 of the bridge indicated in Figure 2 are shown in Figure 6. Under L1 ground motions, the accelerations of Case-1: No train are almost the same as those of Case-2: Train as mass and the displacements of Case-2 are larger than those of Case-1. This indicates that the acceleration and the displacement responses show good agreement with the features of the seismic response spectra, which results from the variation of natural periods due to the train mass. And the reduction effect of

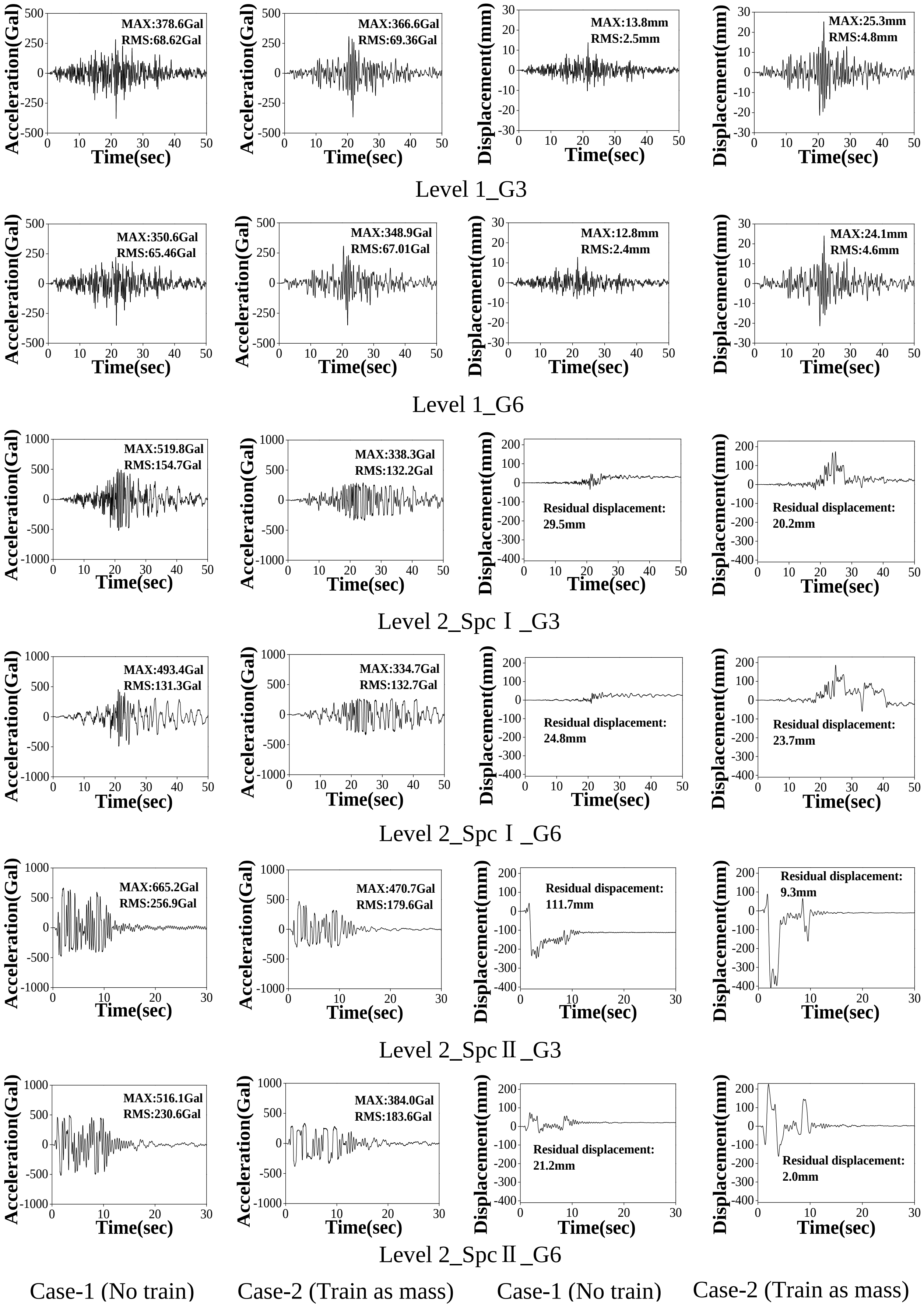


Figure 6: Seismic responses of bridge (horizontal acceleration and displacement).

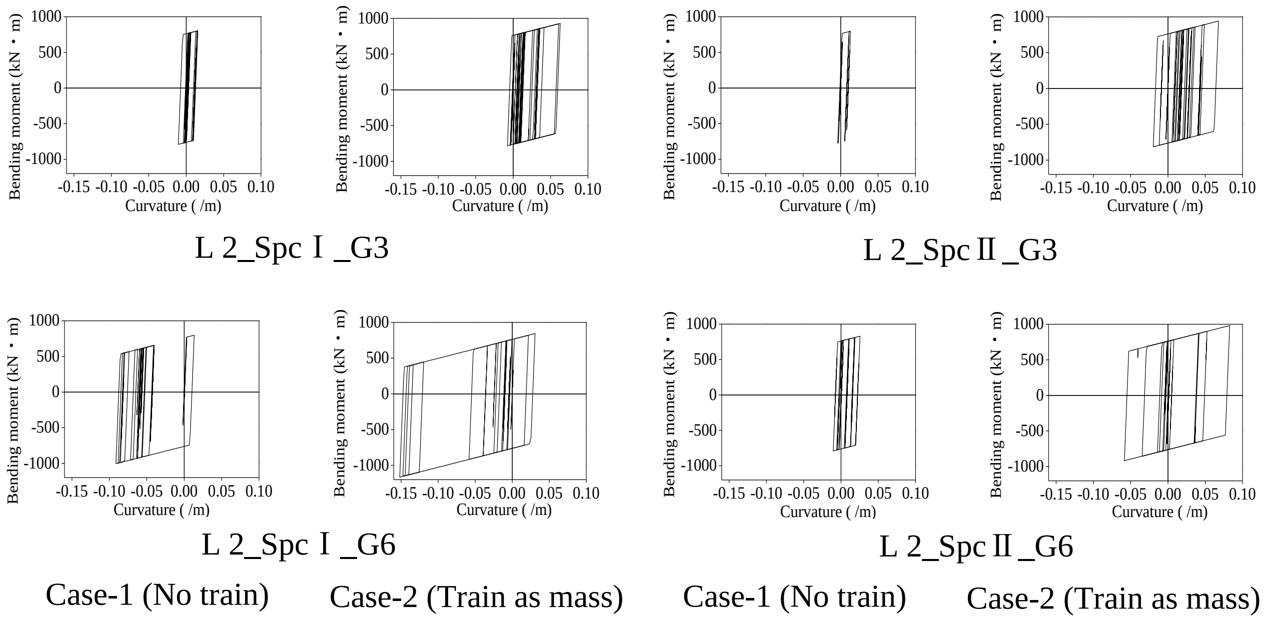


Figure 7: M- ϕ hysteresis of Right 1 pier

the train as additional mass on the acceleration response of the bridge can be confirmed by comparing the acceleration results of Case-1 and Case-2 under L2 ground motions. The acceleration responses of Case-1 display lager the maximum values than those of the Case-2. This is because, considering the train mass, the hysteresis loops at Right 1 pier bottom shown in Figure 7 are large and dissipation energy increase. However, in the displacement responses of L2 ground motions, the final residual displacements of Case-2 is smaller than those of Case-1, whereas the hysteresis loops of Case-2 are much larger than those of Case-1 in Figure 7. The piers may be damaged severer due to the larger hysteresis loops. This phenomenon indicates that the restoring force of the pier become stronger due to considering the train as additional mass to the bridge during the earthquake, which may lead to an underestimation of the plastic deformation.

3.2. Influence of ground motions

The acceleration and the displacement responses of G3 are smaller than those of G6 expect for the displacement responses of Case-2 subjected to L2_Spc1. This is because the two ground motion characteristics are different. The natural period of G3 soil is quite different from that of G6 soil and the seismic response spectra of G3 soil tends to be larger than that of G6 soil.

4. CONCLUSIONS

In this paper, linear and nonlinear seismic response analyses of a typical portal rigid frame reinforced concrete Shinkansen viaduct are analyzed and evaluated by using a general program. The seismic performance of the elevated bridge is evaluated for the cases of considering no train and considering the train as additional mass. In the analytical results, under L1 ground motions, the change of the structural natural periods by considering the train mass affects in particular the displacement response. Under L2 ground motions, analytical results show the reduction effect due to the train as additional mass on the seismic response, while the damage state of the piers may be underestimated. And the seismic responses of the bridge are also affected by the difference of the ground properties.

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