NUMERICAL ANALYSIS AND REINFORCEMENT EFFECT EVALUATION ON SEISMIC RESPONSE OF SHINKANSEN VIADUCTS UNDER DYNAMIC TRAIN LOAD

X. HE[1*], Y. KATASE[2†], H. HISASHI[2], M. KAWATANI[3], T. HAYASHIKAWA[1], C.W. KIM[4], and T. MATSUMOTO[1]

[1]*Faculty of Engineering, Hokkaido University*
[2]*Graduate School of Engineering, Hokkaido University*
[3]*Graduate School of Engineering, Kobe University*
[4]*Graduate School of Engineering, Kyoto University*

ABSTRACT

This study is intended to numerically evaluate the effect of proposed countermeasures to reduce the seismic response of Shinkansen viaducts. In this study, assuming that the structural behavior remains in elastic domain under moderate earthquakes, an analytical approach to simulate the seismic response of the Shinkansen bridge-train interaction system is developed. A bullet train car model idealized as a sprung-mass dynamic system, which can reflect the motions of the car in both vertical and horizontal directions, is established. The viaduct is idealized as a 3D finite element model. Differential equations of the bridge system are derived taking advantage of modal analysis technique. Newmark's β method for direct numerical integration is adopted to solve the coupled differential equations of the bridge-train interaction system. Then the seismic responses of the bridge under bullet trains are simulated. Countermeasures to reinforce the bridge structure against seismic loads are proposed and numerically examined in this paper.

Keywords: Seismic analysis, Train-bridge interaction, Seismic response reduction, Shinkansen

1. INTRODUCTION

The earthquake-proof capacity of the high-speed railway system, so called Shinkansen, has been always a concern considering the extremely high-speed of the bullet trains, since Japan is located in one of the most earthquake-prone regions in the world. 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

* Corresponding author: Email: xingwen_he@eng.hokudai.ac.jp

† Presenter: Email: yositugu.katase@gmail.com

was recognized anew after this disaster. In particular, for the existing viaducts, it is important to developed effective countermeasures to ensure the structural safety and running safety of the train.

On the other hand, in the Japanese seismic design code for railway structures (RTRI 1999) the trains are still treated as subordinate variable loads, virtually additional mass, to the bridge structure but not dynamic systems, because of extreme complexities of the bridge-train interaction under earthquakes. However, obviously it is not completely rational to treat the train merely as an additional mass to the bridge structure because the train itself is a complicated dynamic system. In the design practice based on performance-based design, to satisfy both safety and economy demands, the dynamic effect of trains on the bridge seismic response should be further investigated. He et al. (He et al. 2005; 2009; 2011) has attempted recently to evaluate the seismic response of the Shinkansen bridge-train interaction system, and further efforts are demanded.

In this study, assuming that the structural behavior remains in elastic domain under a moderate earthquake, an analytical approach to simulate the seismic response of the Shinkansen bridge-train interaction system is developed. A bullet train car model idealized as a sprung-mass dynamic system, which can reflect the motions of the car in both vertical and horizontal directions, is developed. The viaduct is idealized as a 3D finite element model. Differential equations of the bridge system are derived taking advantage of modal analysis technique. Newmark's β method for direct numerical integration is adopted to solve the coupled differential equations of the bridge-train interaction system. Then the seismic responses of the bridge and the train are simulated. The dynamic effect of the train on the seismic response of the bridge is examined. The countermeasures to reinforce the bridge structure against seismic loads are also proposed, whose effects are also numerical examined in this paper.

2. ANALYTICAL MODELS AND ASSOCIATED PROPERTIES

The analytical models of the train and the structure as well as the seismic loads used for the case-studies in this paper are described as follows, together with their associated properties.

2.1. Train model

The bullet train is idealized as a sprung-mass dynamic system, assuming that the car body and the bogies are rigid bodies and connected with each other three-dimensionally by linear springs and dampers. The *j*th car model of the bullet train is shown in Figure 1. In this model, the sway, bouncing, pitching, rolling and yawing motions of the car body, and the sway, parallel hop, axle windup, axle tramp and yawing motions of the front and rear bogies are taken into account, which leads to a 15-degree-of-freedoms (DOFs) system. The variants used in the train model with 15-DOFs are shown in Table 1.

A bullet train composed of 16 cars is adopted in the analysis based on the actual operational condition. Each car is treated as independent dynamic system without modeling the coupling device, considering the analytical conditions that the train is running on a straight line and the inertia force

due to the ground motion is simultaneously acting on the cars without phase difference. The distance between the centers of two adjacent cars is 25 meters. The train dynamic properties are shown in Table 2.

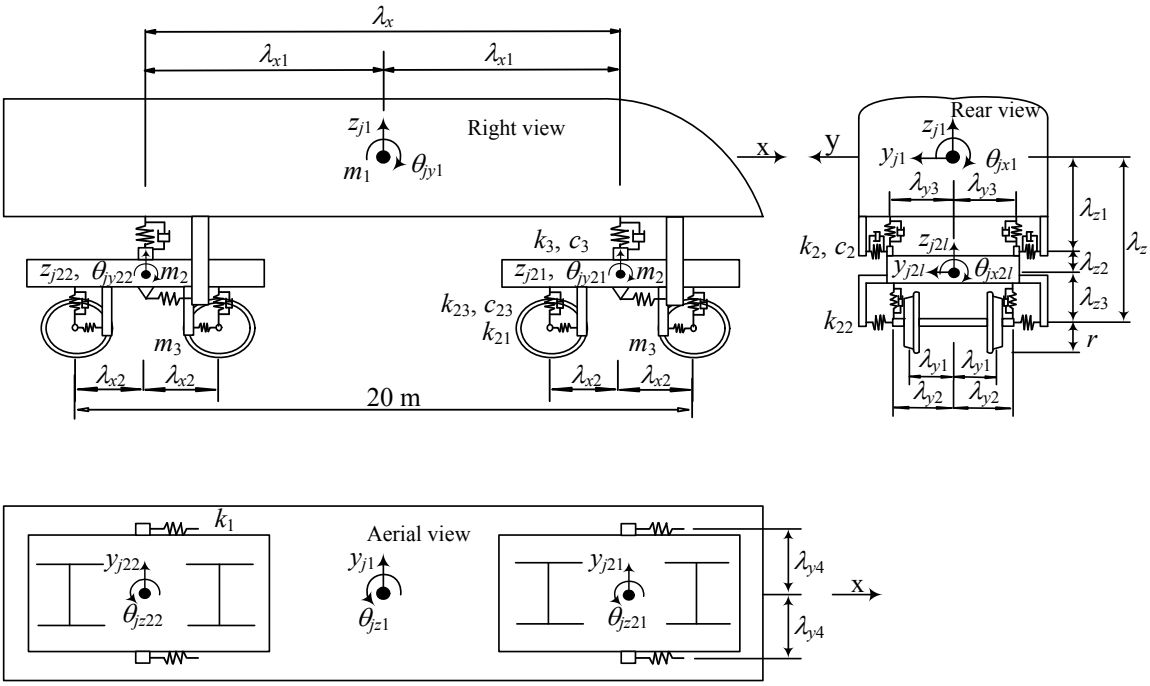


Figure 1: Sprung-mass dynamic car model with 15 DOFs

Table 1: Variants employed in train model

Definition (j^{th} car)	Notation
Lateral translation of car body	y_{j1}
Sway of front bogie	y_{j21}
Sway of rear bogie	y_{j22}
Bouncing of car body	z_{j1}
Parallel hop of front bogie	z_{j21}
Parallel hop of rear bogie	z_{j22}
Rolling of car body	θ_{jx1}
Axle tramp of front bogie	θ_{jx21}
Axle tramp of rear bogie	θ_{jx22}
Pitching of car body	θ_{jy1}
Windup of front bogie	θ_{jy21}
Windup of rear bogie	θ_{jy22}
Yawing of car body	θ_{jz1}
Yawing of front bogie	θ_{jz21}
Yawing of rear bogie	θ_{jz22}

Table 2: Properties of bullet train

Definition	Notation	Value
Weight of car body	w_1	321.6 kN
Weight of bogie	w_2	25.9 kN
Weight of wheel	w_3	8.8 kN
Mass moment of inertia of car body	I_{x1}	49.2 kN·s^2·m
	I_{y1}	2512.6 kN·s^2·m
	I_{z1}	2512.6 kN·s^2·m
Mass moment of inertia of bogie	I_{x2}	2.9 kN·s^2·m
	I_{y2}	4.1 kN·s^2·m
	I_{z2}	4.1 kN·s^2·m
Spring constant	k_1	5000 kN/m
	k_2	176.4 kN/m
	k_3	443 kN/m
	k_{21}	17500 kN/m
	k_{22}	4704 kN/m
	k_{23}	1210 kN/m
Damping coefficient	c_2	39.2 kN·s/m
	c_3	21.6 kN·s/m
	c_{23}	19.6 kN·s /m

2.2. Bridge and rail models

A typical Shinkansen RC viaduct of portal rigid frame structure as shown in Figure 2, on which the field measurements were carried out, is employed for seismic analyses. The viaduct is composed of bridge blocks of 24 m long connected by rail structure and ballast at adjacent ends. Each block consists of three 6 m-long center spans and two 3 m cantilever girders (hanging parts) at each end. Figure 3 shows that the one-block bridge is modeled as 3D beam elements with six-DOF at each node. The lumped mass system is adopted for the beam elements. Mass of the ballast is also incorporated into the structural elements. The total weight of one block is about 6500 kN. Spring elements are set at the pier bottoms to simulate the effect of ground springs. The ground springs are calculated including the elastic effects of the footing and pile structures as well as the surrounding soil. The ground spring constants are shown in Table 3, based on field measurements. Rayleigh damping (Agabein 1971) is adopted for the structural model. According to the past field test results, the damping constant of 0.03 is assumed for the first and second natural modes of the structure.

The rail structure is also modeled as 3D beam elements. Spring elements are also defined here to simulate the elastic effect of sleepers and ballast at the positions of sleepers. Properties of the rail and the spring constant of the track are shown in Table 4. The irregularities in both vertical and horizontal directions of the rail surface are considered in the analysis. The measured values of the irregularity (Kawatani et al. 2004) in the vertical direction are used and the irregularity in the horizontal direction is assumed to be half the values of the vertical ones based on empirical results.

2.3. Reinforcement structures

Two countermeasures to reinforce the bridge structure against seismic loads are proposed in this study. To ensure the safety of both the running bullet train and the bridge structure, it is considered effective to increase the horizontal stiffness of the bridge. Therefore, in the proposed countermeasures, steel struts are adopted to enhance the bridge piers in horizontal direction, as shown in Figure 4 (Reinforcement model 1) and Figure 5 (Reinforcement model 2). The reinforcing members are H steels whose stiffness is about half of the pier's stiffness. In Reinforcement model 1, one end of the strut is connected to the pier top and the other end to the pier bottom, while in Reinforcement model 2 the strut is connected to the top and the middle point of the pier. The joints between the steel struts and the piers are rigid connections. The struts are intersecting with each other and rigidly jointed at the intersected point. The difference between the two models is whether to completely block the under-bridge space or not.

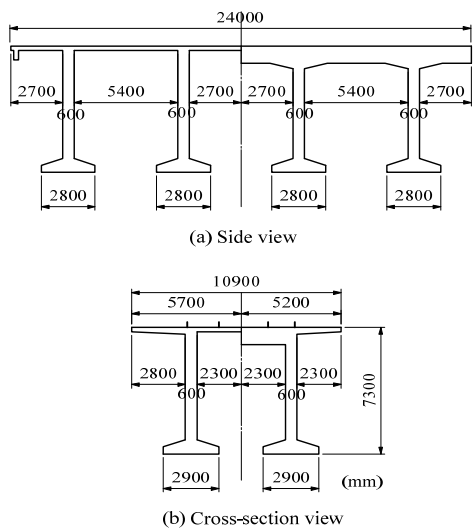


Figure 2: Shinkansen RC viaduct

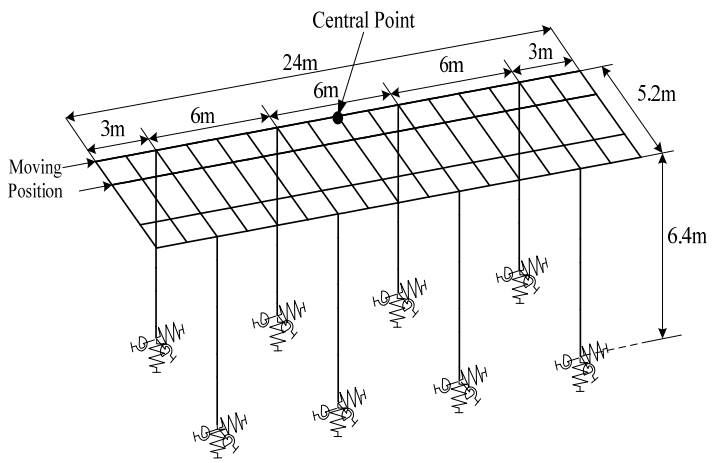


Figure 3: standard bridge model

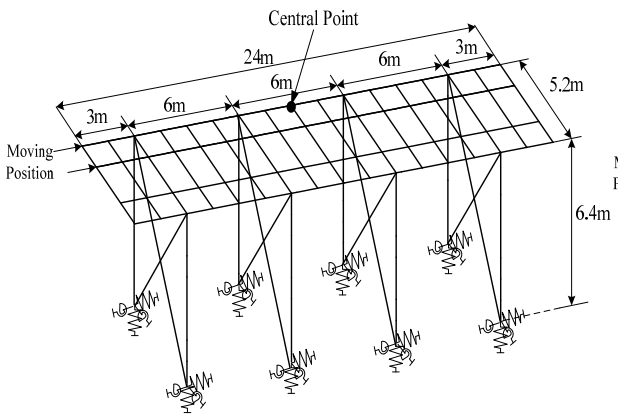


Figure 4: Reinforcement model 1

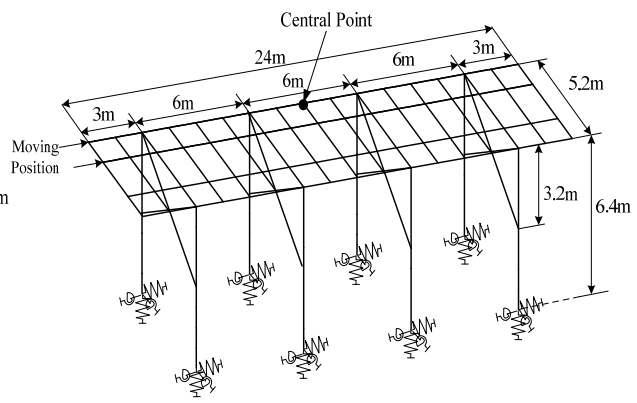


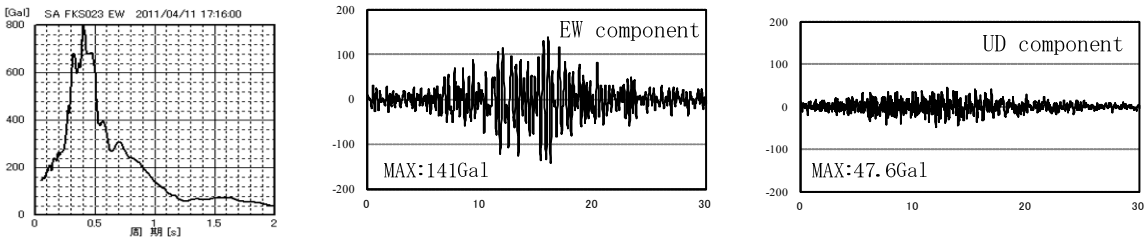
Figure 5: Reinforcement model 2

Table 3: Ground spring constants

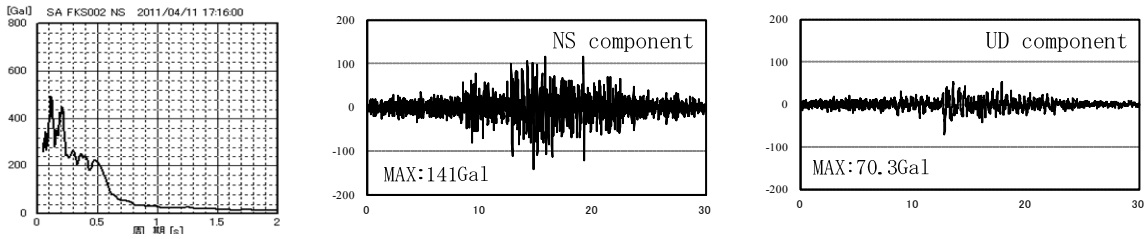
Sort of spring	Longitudinal	Transverse
Vertical spring of pile top (kN/m)	3.86×10^{6}	
Rotating spring of pile top (kN・m/rad)	3.64×10^{6}	2.42×10^{6}
Horizontal spring of footing (kN/m)	4.84×10^{3}	4.72×10^{3}
Horizontal spring of pile top (kN/m)	8.22×10^{4}	8.08×10^{4}

Table 4: Properties of the rail

Area (m^2)	7.75×10^{-3}
Mass (t/m)	0.0608
Moment of inertia (m^4)	3.09×10^{-5}
Spring constant of track (MN/m)	70



Ground motion 1 (Aiziwakamatsu, Fukushima on April 10, 2011)



Ground motion 2 (Harikawa, Date, Fukushima on April 10, 2011)

Figure 6: Ground motions (Time history and seismic response spectrum)

2.4. Seismic loads

To investigate the seismic response of the bridge-train interaction system, two actual recorded moderate ground motions with different frequency components downloaded from the Kyoshin Network (K-NET) of National Research Institute for Earth Science and Disaster Prevention in Japan (NIED 2011), which are the records of from aftershock of the 2011 off the Pacific coast of Tohoku Earthquake, are adopted for the analyses. The acceleration waveforms as well as the response spectra of the horizontal components are shown in Figure 6. Ground motion 1 was recorded at Aiziwakamatsu, Fukushima on April 10, 2011, and Ground motion 2 at Harikawa, Date, Fukushima on April 10, 2011, respectively. The designations of EW, NS and UD in the figures respectively indicate data channels of East-West, North-South and Up-Down directions.

3. DEVELOPED NUMEICAL PROCEDURE

In this research, using the above numerical models, the bridge-train interaction is formulated based on D'Alembert's Principle and FEM, and a computer program is developed. The dynamic differential equations of the bridge are derived using modal analysis technique (Kim et al. 2005). The earthquake excitation is considered as inertial force acting simultaneously on all masses of the bridge and train models.

Newmark's β step-by-stet numerical integration method (Newmark 1959) is employed to solve the coupled differential equations of the bridge-train interaction system. The parameter β, which controls the variation of acceleration within the time step, is set as 1/4. The rate of convergence is set as 0.001 for the acceleration response to ensure the analytical accuracy. The structure is assumed to remain in elastic behavior during moderate earthquakes. The bridge-train interaction algorithm is verified by comparing the analytical results with experimental ones. The accuracy of the seismic analysis algorithm is validated through comparison with a general program. The detailed formulization of the bridge-train interaction system and in-formation about the validation process of the analytical procedure are described in the Reference (He et al. 2011).

4. SEISMIC RESPONSE EVALUATION

4.1. Analytical cases

Employing the seismic analytical approach described above, the dynamic response analyses using the analytical models and the seismic loads described previously are carried out. To examine the dynamic effect of the train on the seismic response of the bridge, the following four analytical cases are devised for the seismic analyses. For all cases, both EW and UD components of the four ground motions are applied.

Case-1: Without considering the train load. The seismic response of the 1-block bridge itself will be calculated.

Case-2: Considering the train as additional mass to the bridge. For the three-block bridge shown in Fig. 3, three cars of the train are assumed to exist on the center of the bridge. Then, they are converted into mass and attached to the structural nodes at the wheel positions.

Case-3: Treating the train as a dynamic system standing on the bridge. As in Case-2, three cars are assumed to stand on the three-block bridge. The dynamic interaction between the train and the bridge is considered.

Case-4: The train is running through the bridge with the operational speed of 270 km/h. In this case, because the speed is very high, an actual train composed of 16 cars runs through the three-block bridge model in a very short time. To fully express the bridge-train interaction, the train is assumed to comprise an infinite number of cars that it can keep running on the bridge during the

earthquake. To evaluate the response of the train, only the response of the car moving on the middle block will be used, thus the response time history will be the combination of those of all cars.

4.2. Analysis results

Corresponding to the four cases described above, horizontal acceleration responses of the Central point of the bridge under the aforedescribed two ground motions with maximum and RMS values are shown in Figure 7 and Figure 8.

For the standard model, the magnitudes of the acceleration response are different for the two ground motions as well as for the four analytical cases. Ground motion 1 results much larger accelerations compared with Ground motion 2. The reason is considered as that the acceleration response spectra of Ground motion 1 are quite lager at around 0.467sec, which is natural period of the bridge, compared with that of Ground motion 2. The horizontal amplitudes of Ground motions 1 and 2 are similar, while the seismic responses of the bridge are significantly different. From this, it can be known that the response spectrum characteristics of the ground motion affect the seismic response significantly, rather than the magnitude of the amplitude. This phenomenon indicates that even for rather moderate earthquakes, according to the relation between the predominant frequency components and the natural period of the bridge, the seismic response may be unexpectedly intense.

Comparing the responses of the Cases 1 and 2, it can be said that considering the train load as additional mass to the bridge is almost on the safe side of design compared with the case of bridge only. However, comparing Cases 2 and 4, although in most occasions considering the train as additional mass can give safer evaluations, for the standard model in Ground motion 1 the response of train running is to some extent larger than Case-2. Therefore, it can be said that considering train as mass is possible to underestimate the seismic response of the bridge, thus needs further discussions. On the other hand, comparing Cases 3 and 4, maximum and RMS values of Case-4 are larger under both seismic waves. It can be said that impact effect of the running train in earthquake also affects the seismic response of the bridge.

For the reinforcement models, comparing with the standard model, seismic responses of the bridge are smaller in all cases under Ground motion 1. In this case, the effectiveness of the reinforcement can be confirmed. However, for Ground motion 2, the seismic responses of the reinforcement models are larger than that of the standard model. It is considered to be a result of the acceleration response spectrum tendency of Ground motion 2, which are quite larger at around 0.254 sec and 0.297 sec that respectively are the natural periods of the reinforcement models 1 and 2. This phenomenon suggests the need to check carefully the changing effect of the dynamic structural characteristics caused by the reinforcement structure in the seismic retrofit works.

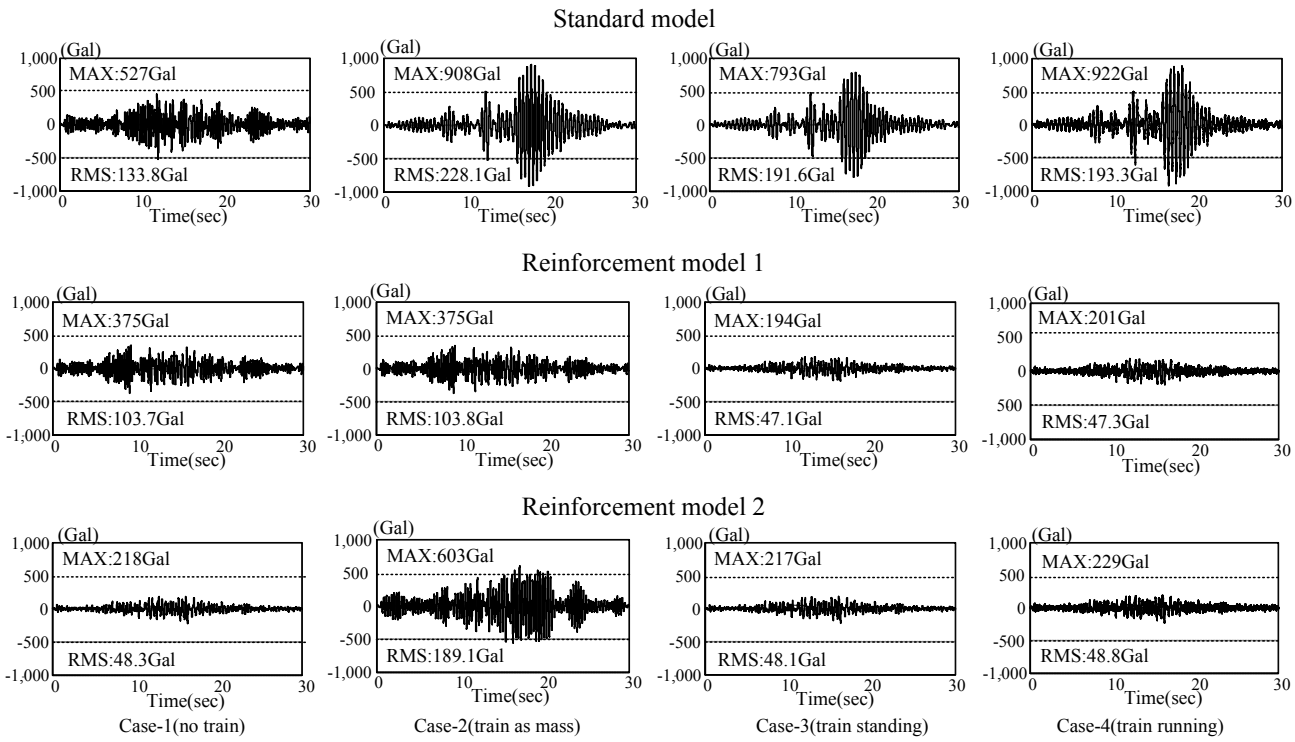


Figure7: Horizontal seismic Acceleration responses of the bridge (Ground motion 1)

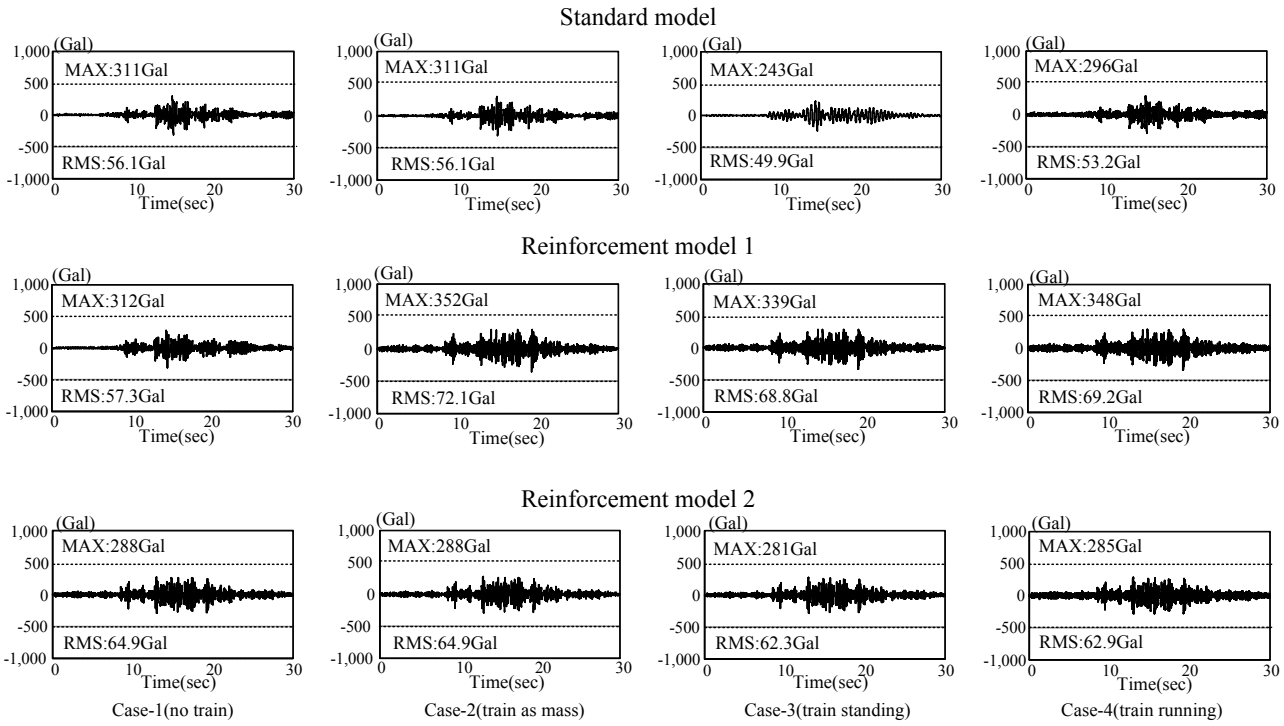


Figure 8: Horizontal seismic Acceleration responses of the bridge (Ground motion 2)

5. CONCLUSIONS

In this study, as a preliminary effort towards elucidating the earthquake performance of the Shinkansen bridge-train interaction system, an numerical approach to simulate the seismic response of the bridge-train system under moderate earthquakes is developed. A 15 DOFs sprung-mass dynamic bullet train model and a 3D finite element Shinkansen viaduct model are employed for the analyses. Then the seismic responses of the bridge and the train are simulated. The dynamic effect of the train on the seismic response of the bridge is examined. The countermeasures to reinforce the bridge structure against seismic loads are also proposed, whose effects are also numerical examined. The numerical approach developed here laid a foundation for future discussions on the seismic design of the bridge-train interaction system.

REFERENCES

Agabein M.E. (1971), "The Effect of Various Damping Assumptions on the Dynamic Response of Structure," Bulletin of International Institute of Seismology and Earthquake Eng, 8: 217-236.

He X., Kawatani M., Sobukawa R. and Nishiyama S. (2005), "Dynamic Response Analysis of Shinkansen Train-Bridge Interaction System Subjected to Seismic Load," Proc. of 4th International Conference on Current and Future Trends in Bridge Design, Construction and Maintenance, Kuala Lumpur, Malaysia, (CD-ROM), 1-12.

He X., Kawatani M., Hayashikawa T. and Matsumoto T. (2009), "Seismic response evaluation of Shinkansen train-bridge interaction system under moderate earthquakes," Proc. of 4th International Symposium on Environmental Vibrations (ISEV2009), Beijing, China, pp.1276-1281.

He X, Kawatani M, Hayashikawa T and Matsumoto T. (2011), "Numerical analysis on seismic response of Shinkansen bridge-train interaction system under moderate earthquakes", Earthquake Engineering and Engineering Vibration, Vol.10, No.1, pp.85-97. (DOI: 10.1007/s11803-011-0049-1.)

Kawatani M, He X, Sobukawa R and Nishiyama S (2004), "Traffic-Induced Dynamic Response Analysis of High-speed Railway Bridges," Proc. of the Third Asian-Pacific Symposium on Structural Reliability and Its Applications, Seoul, Korea, 569-580.

Kim C.W., Kawatani M. and Kim K.B. (2005), "Three-dimensional dynamic analysis for bridge-vehicle interaction with roadway roughness," Computers and Structures, 83(19-20): 1627-1645.

Newmark, N.M. (1959), "A Method of Computation for Structural Dynamics," Journal of the Engineering Mechanics Division, ASCE, 85(EM3): 67-94.

National Research Institute for Earth Science and Disaster Prevention (NIED) (2011), Kyoshin Network (K-NET), http://www.k-net.bosai.go.jp.

Railway Technical Research Institute (RTRI) (1999), "Seismic Design Code for Railway Structures", Maruzen Co., Ltd.: Tokyo, Japan. (in Japanese)