Numerical analysis on dynamic response of viaducts induced by running trains

Hokkaido University Student member ○ Liangming SUN
Hokkaido University Fellow member Toshiro HAYASHIKAWA
Hokkaido University Member Xingwen HE
Kobe University Fellow member Mitsuo KAWATANI
Hokkaido University Member Takashi MATSUMOTO

1. INTRODUCTION

Along with the rapid urban development in modern Japan, Japan's high-speed railway system, the Shinkansen, serves a vital role in the national transportation network since 1964. However, when trains are running on the railway bridge, the bridge is subjected to dynamic loads which induce bridge vibrations. In major urban areas, Shinkansen viaducts are often so adjacent to residences or important facilities. Bridge vibrations propagate to the ambient ground via footing and piles, thereby causing some long-term environmental vibration and noise problems such as influence to precision instruments or people who are studying or resting[1)].

Theoretical studies of bridge vibrations caused by running trains have been carried out since the mid 1960s[2-7)]. Train-bridge interaction problems have been prosperously researched by many researchers and remarkable progress in modeling the dynamic vehicle systems and bridge structures as well as their interaction has been achieved. A series of renowned studies on vehicle-bridge interaction problems were conducted by Frýba[2)], and were applied to many actual engineering problems. Notable studies were also carried out by Matsuura et al[3)], Xia et al[4)] and Yang et al[5)]. These studies treated the vehicles as well as structures as three-dimensional (3D) models and their interactions were accurately considered. These involved several matters: modeling of trains, bridges and the interface between trains and bridges, adopting of rail irregularity, solving the train-bridge dynamic interaction problem and interpreting the results. Recently, as a preparation for site vibration analysis, Kawatani et al[6-7)] established an approach to simulate the dynamic response of Shinkansen elevated railway bridges due to running trains by taking the train-bridge interactions into consideration. In their approach, focusing on the vertical response of elevated bridges, a 9 degrees-of-freedom (DOFs) bullet train model was developed and viaducts were modeled as 3D beam elements. The validity of the analytical procedure was demonstrated through comparing analytical results with experimental ones. Although enormous efforts had been devoted and significant progress had been achieved towards elucidating the train-bridge interaction problems, the actual train-bridge interactions still remained because of the complicated natural phenomena and the horizontal response of viaducts was rarely considered.

In this study, in order to research the dynamic response of viaducts and investigate the dynamic reaction forces at pier bottoms that can be used as external excitations for future environmental vibration analysis, the 3D train, viaducts and rail models were established and an numerical approach to simulate dynamic interactions between the train and viaducts was established by taking advantage of 3D dynamic analysis. The analytical results of viaducts were compared with the experimental ones to confirm the validity of the analysis.

2. ANALYTICAL MODELS

2.1 Train model

Figure 1 shows one car of the train that is modeled as a 15 DOFs system, assuming that the car body and the bogies are rigid bodies and that they are connected to each other three-dimensionally by linear springs and dampers. In this train 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 DOFs system. The effect of rail surface roughness is considered. The train is comprised of 16 cars. 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. Table 1 shows the dynamic properties of the trains. The velocity is set as 270 km/h, referring to the actual Shinkansen operational speed.

2.2 Viaduct model

Figure 2 shows a typical high-speed railway reinforced concrete viaduct in the form of a rigid portal frame. The viaducts are built with 24m length bridge blocks which are separated with each other and connected only by rail structure at adjacent ends. Each block has three 6m length center spans and two 3m length cantilever girders, so called hanging parts, at each end. Three blocks of the bridge are adopted for the analysis and are modeled with 3D beam elements as shown in Figure 3. Thus the connecting effect of the rail structure and the influence of train's entering and leaving can be naturally taken into account. Double nodes defined as two independent nodes sharing the same coordinate are adopted at the pier bottoms to simulate ground spring effect and between the rail and slab to express the elastic effect of the sleeper and ballast. Rayleigh damping is adopted for the structural model. According to past field test results, a damping constant of 0.03 is assumed for the first and second natural modes of the structure.

Table 1 Dynamic properties of train

Definition	Notation	Value
Weight of car body	w_1	321.6kN
Weight of bogie	w_2	25.9kN
Weight of wheel	w_3	8.8kN
Mass moment of inertia of car body	I_{x1}	49.2kN·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}	19.96 kN·s/m

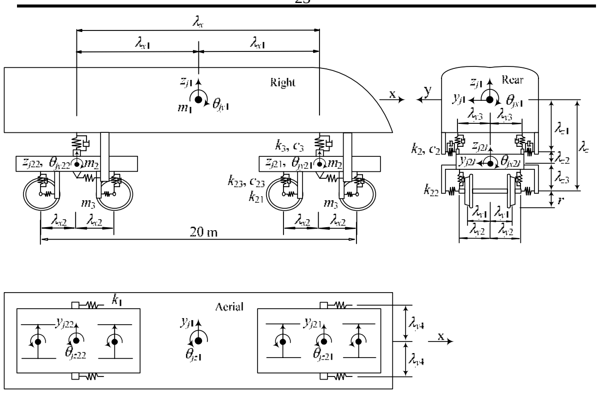


Fig. 1 15-DOF bullet train car model

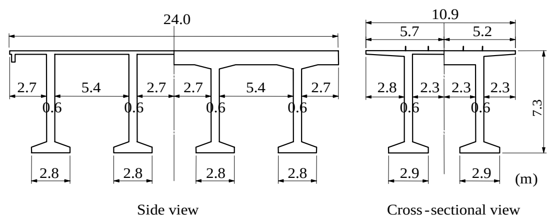


Fig. 2 Dimensions of viaduct

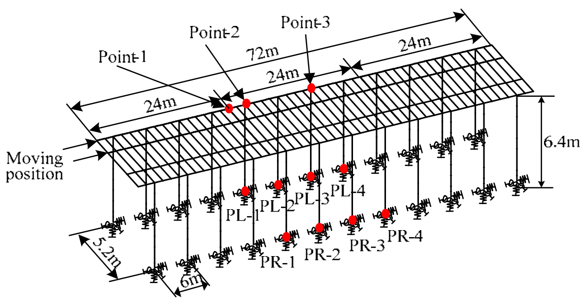


Fig. 3 Analytical model of viaduct

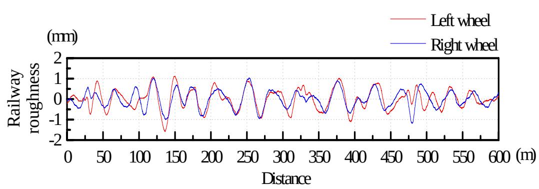


Fig. 4 Vertical railway roughness

Table 2 Property of railway

Definition	Notation	Value
Area	A_r	7.75×10^{-3} m^2
Mass	m_r	0.0608 t/m
Moment of inertia	I_r	3.09×10^{-5} m^4
Spring constant of track	k_r	70 MN/m

2.3 Rail model

The rail structure is also modeled as 3D beam elements with 6 DOFs at each node. Double nodes 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 2. The roughness in both vertical and horizontal directions of the rail surface is considered in the analysis. The measured values of roughness in the vertical direction which are shown in Figure 4 are used and those in the horizontal direction are assumed based on experiences.

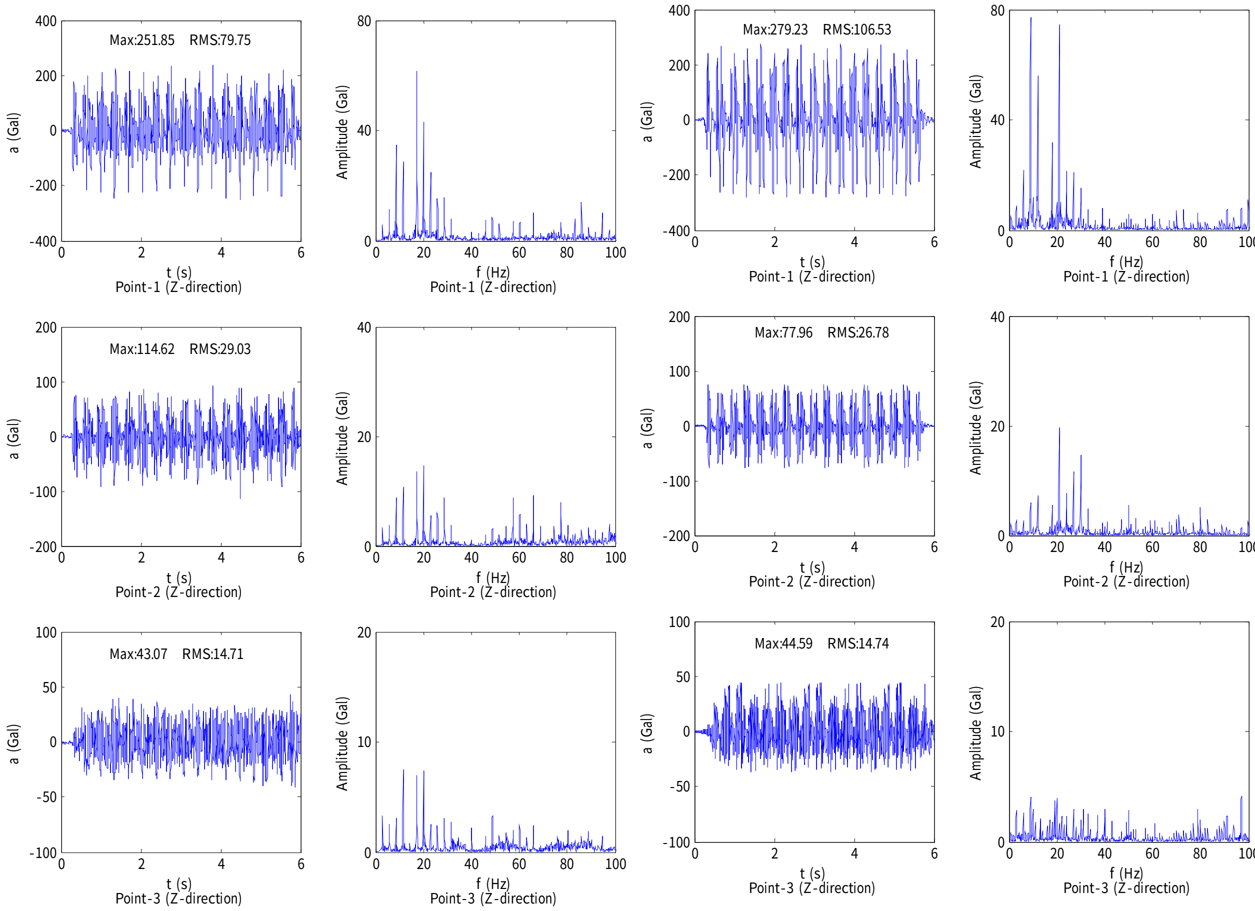
3. NUMERICAL ANALYTICAL APPROACH

Dynamic responses of the viaducts under running high-speed trains are analyzed by taking the train-bridge interaction into consideration based on the computer program. The viaducts, including the rail structure, are modeled as 3D beam elements. The dynamic differential equations of the bridge are derived using modal analysis. Newmark's β step-by-step numerical integration method is applied to solve dynamic differential equations. The validity of the numerical procedure is demonstrated through comparing analytical results with experimental ones. The dynamic reaction forces at the pier bottoms which will be used as external excitation forces in the foundation-ground interaction problem are then simulated using the influence value matrix of the reaction force. Considering the extremely high speed of train, the time step interval in the numerical integral is set to 0.0005s.

4. ANALYTICAL RESULTS

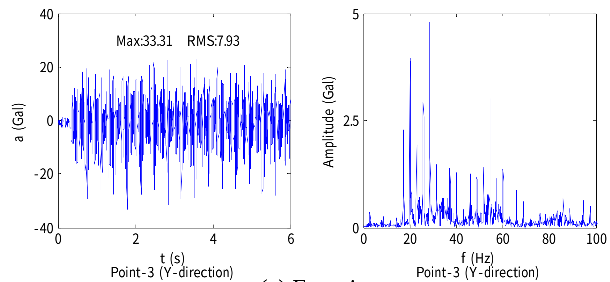
4.1 Dynamic responses of the viaduct

Through the eigenvalue analysis of the bridge model, the predominant frequency of the horizontal natural mode is observed as 2.20Hz, showing good agreement with the value obtained from the field test, which is 2.19Hz. Therefore, the bridge model validation can be confirmed. The analytical acceleration responses and the measured ones of observation points of viaducts indicated in Figure 3, are shown in Figure 5 and Figure 6. They showed that analytical results using the 15 DOFs bullet train model indicated good agreement with the experimental results, thereby validating this analytical procedure. In the train-induced bridge vibration analysis, the bridge vibration recorded at point-1 through point-3 of the viaducts indicated in Figure 3 will be examined. Here, point-1, point-2 and point-3, respectively, are the hanging part, the top of the first pier, and the top of the third pier of the viaduct, with respect to the direction that the train runs towards.

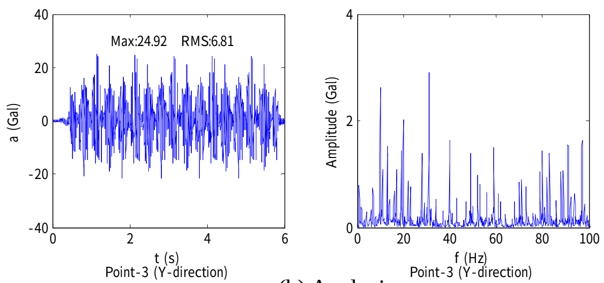


(a) Experiment (b) Analysis

Fig. 5 Vertical acceleration of viaduct



(a) Experiment

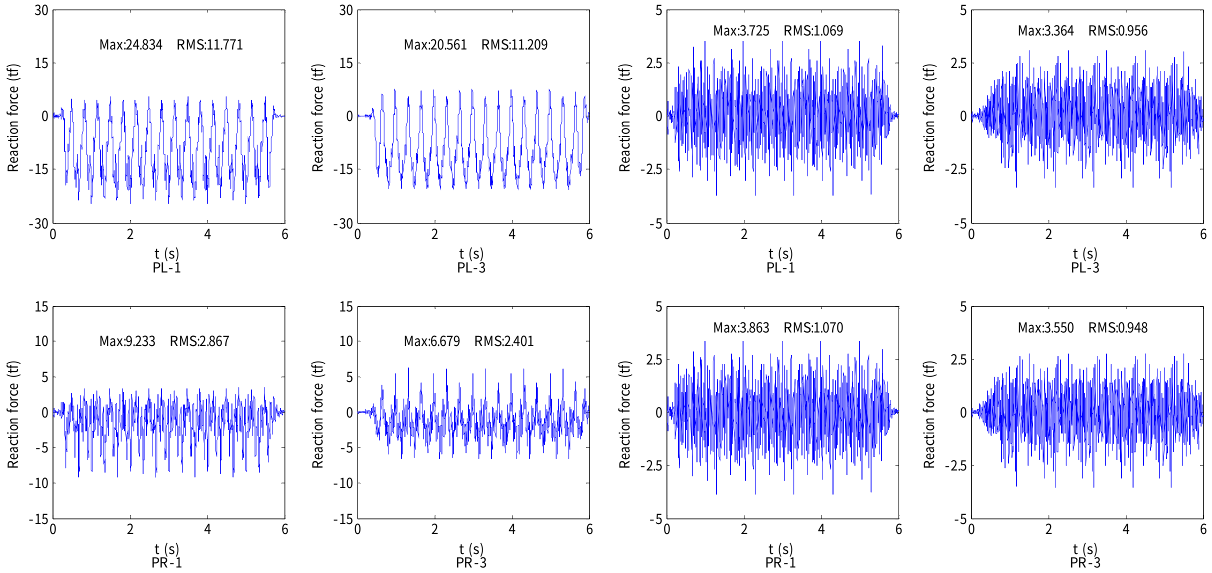


(b) Analysis

Fig. 6 Horizontal acceleration of viaduct

The analytical acceleration responses and the experimental ones in the vertical directions at point-1 through point-3 of the bridge indicated in Figure 3 are shown in Figure 5. Their maximum (Max) and root-mean-square(RMS) values together with the Fourier spectra are also indicated in the figure. As shown in Figure 5, the analytical results indicate relatively good agreement with experimental results, from which the validity of the train-bridge interaction analytical procedure can be confirmed. Here, the vertical acceleration responses indicate the tendency of Point-1>Point-2>Point-3. For all points, the acceleration responses are predominant at around 10Hz and 20Hz. Furthermore, the responses at point-1 display a relatively larger difference between the analysis and experiment, compared with the other points. The reason is considered as follows. Point-1 is at the hanging part, which induces a predominant structural dynamic response because it is a cantilever beam. Therefore, the vibration amplitude at point-1 is rather bigger than the other points and significantly influenced by the conditions of rail continuity, irregularities and the ballast damping effect, which are difficult to accurately consider in the structural modeling.

The analytical acceleration responses and the experimental ones in the horizontal directions at point-3 of the bridge are shown in Figure 6. Their Max and RMS values together with the Fourier spectra are also indicated in the figure. As shown in Figure 6, although there are a little different around 10Hz and 80Hz between the experimental result and the analytical result, the analytical response of point-3 indicates relatively good agreement with the experimental one, thereby validating this analytical procedure. Here, it shows that the horizontal acceleration response is very smaller than the vertical one. So the horizontal vibration influence is very small to the dynamic response of viaducts.



(a) Vertical reaction forces　　　　(b) Horizontal reaction forces

Fig. 7 Reaction forces at the pier bottoms

4.2 Dynamic reaction forces at pier bottoms

Reaction forces at the bottoms of the piers are calculated using the influence value matrix of reaction forces described. As shown in Figure 3, PL-1 to PL-4 and PR-1 and PR-4 respectively indicate the piers on the left and right sides of the bridge, with respect to the train's direction. Their Max and RMS values are also indicated in the figure. As shown in Figure 7, the vertical dynamic reaction forces of the piers on the left side are much stronger than those on the right side because the trains are assumed to run along the left sides of the viaducts. In particular, the Max and RMS at PL-1 are somewhat larger than that of PL-3, and the Max and RMS at PR-1 are also somewhat larger than that of PR-3. The probable reason is that the maximum acceleration response that engenders a larger inertia force appears at the hanging part of the bridge in Figure 5. But the horizontal dynamic reaction forces of the piers on the left side are the similar with those on the right side. Dynamic reaction forces at the bottoms of the piers obtained here are used as input external excitations in the consequent site vibration analysis.

5. CONCLUSIONS

In this study, a 15 DOFs train model was developed and the dynamic responses of Shinkansen viaducts were simulated by taking the train-bridge interaction into consideration. Analytical results were validated through comparison with experimental ones. The vertical and horizontal acceleration of viaducts and the dynamic reaction forces at pier bottoms were investigated through the analytical procedure. The vertical acceleration responses indicated the tendency of Point-1>Point-2>Point-3 and were predominant at around 10Hz and 20Hz. The horizontal acceleration response at Point-3 was very smaller than the vertical one. And the vertical dynamic reaction forces of the piers on the left side were much stronger than those on the right side but the horizontal ones were similar. Using the developed numerical approach, it is possible not only to simulate and evaluate dynamic response of viaducts that is caused by running trains, but also to investigate the effectiveness of presumed countermeasures against bridge and site vibration by reinforcing the bridge structure, or employing other means.

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