

## NUMERICAL EVALUATION ON COUNTERMEASURES AGAINST TRAIN-INDUCED GROUND VIBRATION AROUND SHINKANSEN VIADUCTS BASED ON TRAIN-BRIDGE INTERACTION ANALYSIS

Xingwen He\*<sup>1</sup>, Mitsuo Kawatani<sup>2</sup>, Toshiro Hayashikawa<sup>1</sup>, Takashi Matsumoto<sup>1</sup>

<sup>1</sup>Faculty of Engineering, Hokkaido University  
{xingwen\_he, toshiroh, takashim}@eng.hokudai.ac.jp

<sup>2</sup> Graduate School of Engineering, Kobe University  
m-kawa@kobe-u.ac.jp

**Keywords:** Train-Induced Ground Vibration, Train-Bridge Interaction, Shinkansen Viaduct, Environmental Vibration, Vibration Reduction Countermeasure.

**Abstract.** *The Shinkansen main lines usually pass directly over densely populated urban areas, where the railway structure mainly comprises viaducts of reinforced concrete in the form of portal rigid frame. The bridge vibration caused by running trains propagates to the ambient ground via footing and pile structures, thereby causing some environmental problems. Along with the further urbanization and development of more rapid trains, there is rising public concern about the environmental problems in modern Japan. Although the importance of environmental problems has been recognized, the development and propagation mechanism of the ground vibration caused particularly by running trains on viaducts remains unclear because of its complicated nature. In Japan, environmental vibration problems caused by trains running along viaducts are traditionally investigated based on field test data. The efficiency of such a process is limited to particular cases. For more general cases, a reliable numerical approach to the environmental vibration problems is anticipated to perform accurate predictions and develop effective countermeasures.*

*In this research, a numerical approach is established to simulate and evaluate the ground vibration around shinkansen viaducts considering the three-dimensional train-bridge as well as foundation-ground dynamic interactions. In this approach, the responses of the viaducts under running bullet trains are calculated to obtain the dynamic reaction forces at the pier bottoms. By applying those reaction forces as input excitations, the ground vibration around the viaducts is then simulated with a general-purpose program, taking into account the foundation-ground interaction. Then, the countermeasures against the excessive ground vibration are proposed based on the dynamic characteristics of the bridge response. The effects of the proposed countermeasures are confirmed numerically using the developed approach. The results showed that the developed numerical approach is effective to simulate and evaluate the bridge and ground vibration as well as the effect of related countermeasures.*

## 1 INTRODUCTION

Japanese high-speed railway system, the Shinkansen, serves a vital role in the national transportation network. The Shinkansen main lines usually pass directly over densely populated urban areas, where the railway structure mainly comprises elevated bridges of reinforced concrete in the form of portal rigid frame. The bridge vibration caused by running trains propagates to the ambient ground via footing and pile structures, thereby causing some environmental problems. Those vibrations can influence precision instruments in hospitals and laboratories, or people who are studying or resting in schools, hospitals and residences and so on. Along with the further urbanization and development of more rapid transport facilities, there is rising public concern about the environmental problems in modern Japan [1].

Although the importance and urgency of environmental problems have been recognized, the development and propagation mechanism of the site vibration caused particularly by running vehicles on viaducts remains unclear because of its complicated nature. In Japan, environmental vibration problems caused by trains running along viaducts are traditionally investigated based on field test data [2]. The efficiency of such a process is limited to particular cases. For more general cases, a reliable analytical approach to the environmental vibration problems is anticipated to perform accurate predictions and develop effective countermeasures. In recent years, effort has been devoted to analytical studies of site vibrations induced by trains moving on the viaducts [3, 4]. In Japan, Hara et al. [5] attempted to clarify the site vibration around Shinkansen viaducts by both experiments and analytical procedure. However in their analyses, the wheel load of the trains is only treated as simple equivalent moving force based on the measured results. Recently, He et al. [6] developed an approach to simulate and evaluate the site vibration around shinkansen viaducts considering the three-dimensional (3D) train-bridge as well as foundation-ground dynamic interactions.

In this study, using the developed numerical approach [6], the responses of the viaducts are calculated to obtain the dynamic reaction forces at the pier bottoms. By applying those reaction forces as input excitations, the site vibration around the viaducts is then simulated with a general-purpose program [7, 8], taking into account the foundation-ground interaction. The numerical results are validated through comparing with the experimental results of both the bridge and the ground vibrations. The countermeasures against the excessive vibration are proposed based on the dynamic responses of the viaduct. The predominant vibration is confirmed at the cantilever parts by both analytical and field test results. With consideration of this dynamic feature, countermeasures by reducing the vibration of such parts are conceived to mitigate the bridge vibration as well as the ground response. The effects of the proposed countermeasures are then confirmed numerically using the developed approach.

## 2 DEVELOPED NUMERICAL APPROACH

In this approach, the dynamic interactions between the train and bridge as well as between the foundation and ground are considered. Currently, it is still difficult to model the entire train-bridge-ground interaction system as a whole, because of not only the extreme complexities of their interaction but also the limit of the computational capacity. Therefore to simplify the problem in this study, the entire interaction system is divided into two subsystems: train-bridge interaction and foundation-ground interaction. In the stage of the train-bridge interaction problem, the analytical program to simulate the traffic-induced bridge vibration is developed, with which the dynamic responses of the viaduct are calculated to obtain the dynamic reaction forces at the pier bottoms. Then, applying those reaction forces as input excitation forces in the foundation-ground interaction problem, the site vibration around the viaducts is simulated and evaluated using a general-purpose program named SASSI2000 [7, 8].

## 2.1 Train-bridge interaction analysis

To perform train-induced site vibration analysis around the viaducts, it is first necessary to simulate the dynamic responses of the bridge as accurately as possible to obtain the dynamic reaction forces. Theoretical studies of the bridge vibration caused by travelling trains have been carried out since the middle 1960s by many researchers. Kawatani et al. [9, 10] recently established an approach to simulate the dynamic response of Shinkansen viaducts caused by running trains, considering the train-bridge coupled vibration. In that approach, a nine-degree-of-freedom (nine-DOF) bullet train model is developed and the viaducts including the rail structure are modelled as finite elements. The simultaneous differential equations of the bridge are derived using modal analysis technique. The Newmark's  $\beta$  step-by-step numerical integration method is applied to solve the differential equations. The validity of the analytical procedure is demonstrated through comparing analytical results with experimental ones. The dynamic reaction forces at the pier bottoms are then simulated using the influence value matrix of the reaction force. The detailed formulation of the train-bridge interaction can be found in the references [6, 10].

## 2.2 Foundation-ground interaction analysis

For the foundation-ground interaction system in this study, site vibrations around the Shinkansen viaducts are simulated using a general-purpose program named SASSI 2000 [8]. In this program, the soil-structure interaction problem is analyzed conveniently using a sub-structuring approach by which the linear soil-structure interaction problem is subdivided into a series of simple sub-problems. Each sub-problem is solved separately and the results are combined in the final step of the analysis to provide a complete solution using the principle of superposition. In particular for site response analysis, the thin layer element method is adopted, which can remove the limitation of half-space elastic theory of isotropic homogeneous media. Detailed theoretical information can be referred to References [7, 11].

# 3 NUMERICAL MODELS

## 3.1 Viaduct model

A typical Shinkansen reinforced concrete viaduct with the form of a portal rigid frame as shown in Figure 1 is adopted in this study. The viaducts are built with 24 m length bridge blocks which are structurally separated with each other and connected by rail structures and ballast at adjacent ends. Each block consists of three 6 m length center spans and two 3 m cantilever girders, so called hanging parts, at each end. Considering the boundary condition, three blocks (72 m) of the bridge with 24 m length of each block are adopted as the analytical model. Only the response of the middle block will be examined. Figure 2 shows that the three-block bridge is modelled 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. Double nodes defined as two independent nodes sharing the same coordinate are adopted at the bottoms of the piers to simulate the effect of ground springs. Rayleigh damping [12] is adopted for the structural model. The damping constant of 0.03 is assumed for the first and second natural modes of the structure. The rail structure is also modelled as 3D beam elements with six-DOF at each node. Double nodes are also defined here to simulate the elastic effect of sleepers and ballast at the positions of sleepers. Only roughness in the vertical direction of the rail is taken into account and the measured values are used. The actual field test to measure the vibration of Shinkansen viaduct was conducted at such a viaduct in the Tokaido Shinkansen. The bridge vibrations recorded at point-1 through point-3 of the viaducts indi-

cated in Figure 2 are examined by comparison with the numerical results to validate the developed analytical procedure. The details of the field test can be further referred to the references [10, 13].

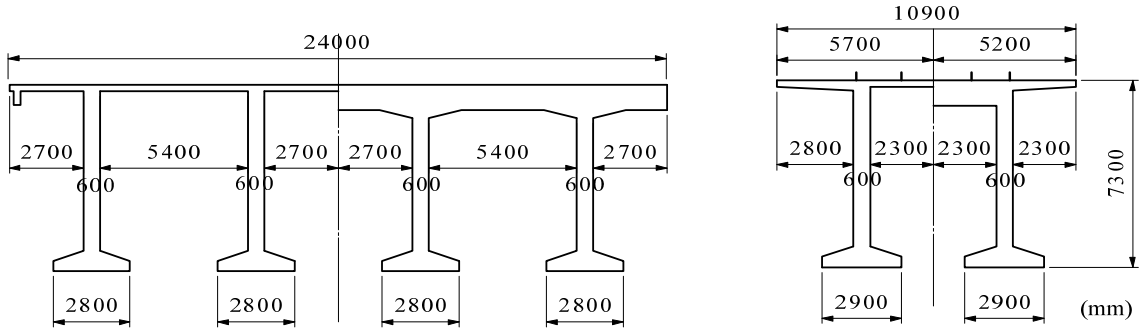


Figure 1: Bridge dimensions.

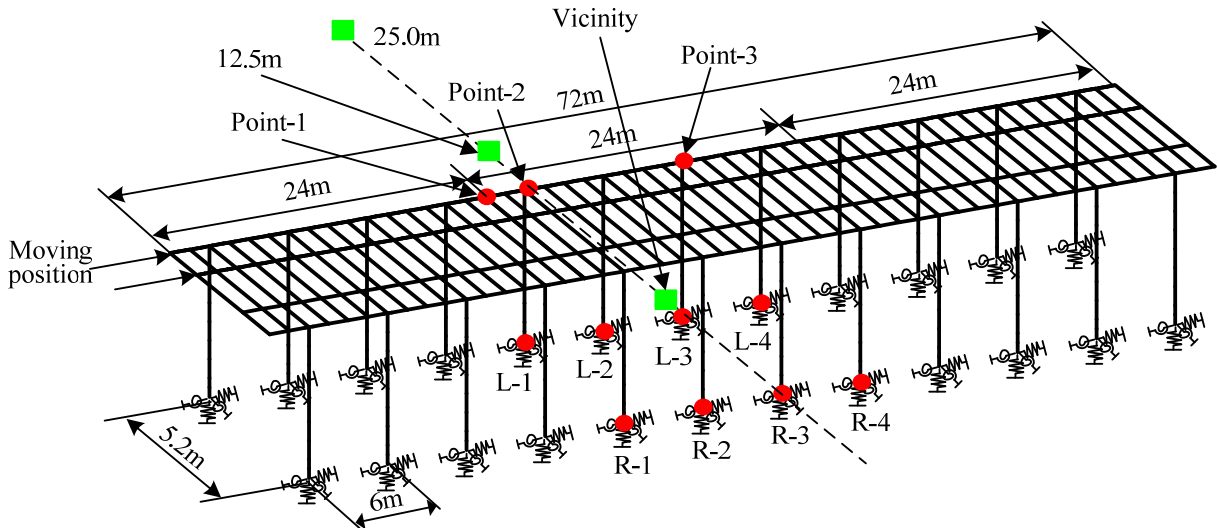


Figure 2: Finite element model of the bridge.

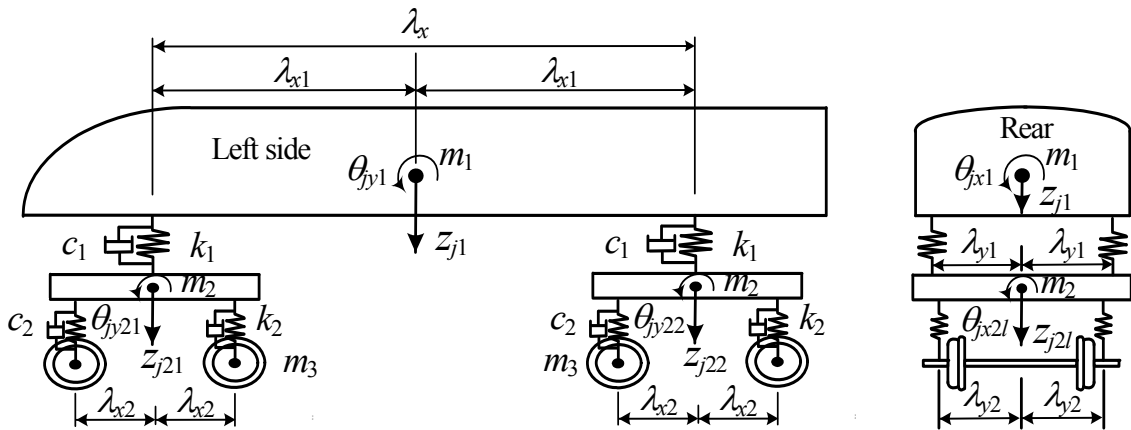


Figure 3: Nine-DOF car model.

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 <sup>2</sup> ·m
	$I_{y1}$	2512.6 kN·s <sup>2</sup> ·m
Mass moment of inertia of bogie	$I_{x2}$	2.9 kN·s <sup>2</sup> ·m
	$I_{y2}$	4.1 kN·s <sup>2</sup> ·m
Spring constant	$k_1$	443 kN/m
	$k_2$	1210 kN/m
Damping coefficient	$c_1$	21.6 kN·s/m
	$c_2$	19.6 kN·s /m

Table 1: Properties of the train.

Distance of bogie centres	$\lambda_x$	17.50 m
1/2 distance of bogie centres	$\lambda_{x1}$	8.75 m
1/2 distance of axles	$\lambda_{x2}$	1.25 m
1/2 distance of upper springs	$\lambda_{y1}$	1.23 m
1/2 distance of lower springs	$\lambda_{y2}$	1.00 m

Table 2: Dimensions of the train.

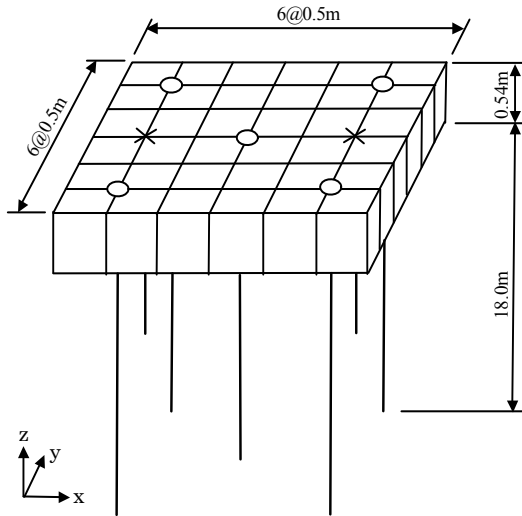


Figure 4: Substructural model.

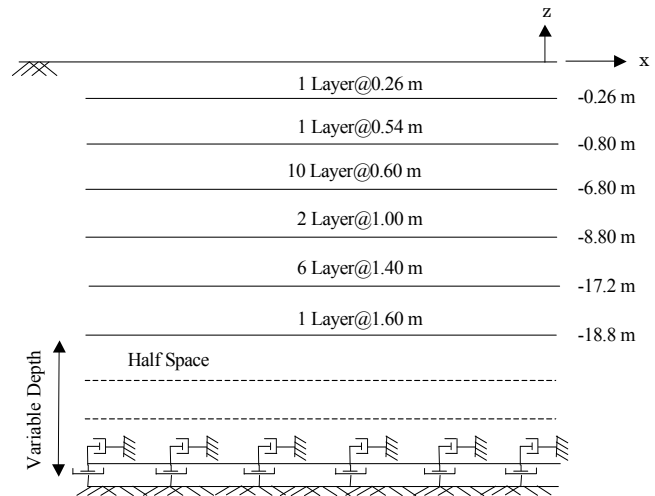


Figure 5: Layer element site model.

### 3.2 Bullet train model

Each car of the train is modeled as nine-DOF sprung-mass model as shown in Figure 3. In this study, considering the focuses of the analysis, the DOFs of the car model are limited to the ones that significantly contribute to vertical vibration of the bridge. Bouncing ( $z_{j1}$ ), pitching ( $\theta_{jy1}$ ) and rolling ( $\theta_{jx1}$ ) of the car body, parallel hop ( $z_{j21}$ ,  $z_{j22}$ ), axle windup ( $\theta_{jy21}$ ,  $\theta_{jy22}$ ) and axle tramp ( $\theta_{jx21}$ ,  $\theta_{jx22}$ ) motions of the axles are considered. The properties and the dimensions of the train are respectively shown in Table 1 and Table 2. The train is composed of 16 cars. The velocity is set as 270 km/h, referring to the actual operational speed. The detailed formulation of the train can be found in the references [6, 10].

### 3.3 Substructural model

The substructural model contains the footing and pile structures and one structural set composed of one footing and seven piles is modelled as Figure 4. The properties of the footing and piles can be found in the reference [6]. The actual footing structure is in the shape of rectangular parallelepiped at the base and a trapezoid at the top. To simplify the analyses in this analysis, the footing is approximated as a rectangular parallelepiped divided into 36 solid elements according to the conversion of volume. The sizes of the solid elements meet the criterion that they be less than 1/5 of the shortest S wavelength in the corresponding layer [8]. The upper footing surface is set to lie 0.26 m under the ground surface. The piles are divided into two types according to their length: Type 1 is 7 m long and Type 2 is 18 m. The  $\circ$  and  $\times$  marks indicate the positions at which the piles are connected vertically to the footing. Herein,  $\circ$  represents 18-m-long piles and  $\times$  represents 7-m-long piles. The piles are modelled as 3D beam elements. The ends of the beam elements are established at the soil layer interfaces.

### 3.4 Site model and measured ground surface points

The actual site mainly comprises three strata separated at depths of 6.8 m and 17.2 m. The velocity of the S-wave in the first stratum is 80 m/s, from which the soil can be considered as relatively soft. The damping constant is assumed as 5%, determined from experiential values. For analysis, the site model is divided further into 21 thin layer elements, whose profiles are shown in Figure 5. The maximum thickness of each layer is determined in compliance with the criterion that it does not exceed 1/5 of the shortest S wavelength in that layer [8]. Layer elements are established down to the depth of 18.8 m, to which the structural model is embedded. The program then automatically adds some extra layer elements and the viscous boundary at the base to simulate the effect of half space.

The ground vibration around Shinkansen viaduct was measured simultaneously in the same actual field test of the bridge vibration [13]. The positions of the measurement points are shown in Figure 2 as Vicinity, 12.5 m and 25.0 m. In all, 24 footings of the three blocks of bridges are adopted to be excited. The measured points are lying on the line passing through the centers of footings R-3 and L-3. They are respectively 3.5 m, 12.5 m and 25.0 m distant from the longitudinal central line of the bridge. In the analysis, the site vibration response of a certain point is obtained from the superposition of those engendered by each footing. The properties of the ground can be found in the reference [6].

### 3.5 Proposed vibration reduction countermeasures

In this study, according to both the numerical and the field test results, the dynamic feature that the predominant vibration occurred at the hanging parts of the viaducts is confirmed. Therefore, it can be easily conceived that the excessive vibration will be reduced by means of reducing the vibration of the hanging parts. Based on such an idea, two countermeasures are proposed in this research to reduce the ground vibration.

The first countermeasure is to directly reinforce the hanging parts. Two simple reinforcement methods are designed. The first one is to connect the adjacent hanging parts rigidly (Rigid joint), and the second is to reinforce the hanging parts with steel struts (With strut). The stiffness of the steel strut is designed to be 1/2 of that of a pier. The depiction of this method is shown in Figure 6 and the struts are rigidly connected to the piers and hanging parts.

The second countermeasure is to use stiffer rails, which can reduce the wheel loads transmitted to the hanging parts. The idea of such stiffer rails has already been applied to the railway systems in Japan in recent years. The properties of the current rail (referred as 60kg rail) and the newly designed one (referred as 70kg rail) used in this paper are shown in Table 3.

	60kg rail	70kg rail
Cross-sectional area (cm <sup>2</sup> )	77.50	88.16
Moment of inertia (cm <sup>4</sup> )	3090	4311
Torsional constant (cm <sup>4</sup> )	400	560
Unit weight (kgf/m)	60.80	69.16

Table 3: Properties of difference rails.

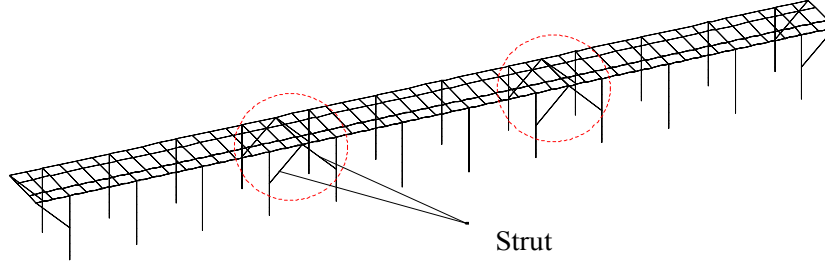


Figure 6: Depiction of reinforcement with steel struts.

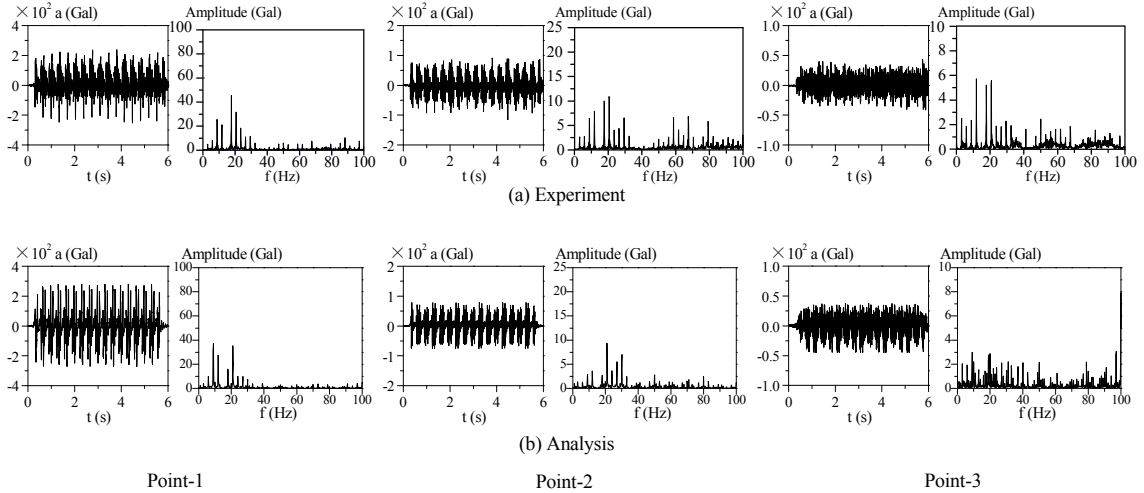


Figure 7: Bridge acceleration response under moving train (Train speed 270 km/h).

## 4 NUMERICAL RESULTS AND DISCUSSIONS

### 4.1 Dynamic response of the viaduct

The analytical acceleration responses and the experimental ones, of point-1 through point-3 of the viaducts indicated in Figure 2, are shown respectively in Figure 7 with Fourier spectra under trains with 270 km/h velocity. Herein, point-1, point-2, and point-3 respectively indicate the point of 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. As shown in Figure 7, analytical results using the nine-DOF train model indicate good agreement with experimental results, thereby validating this numerical procedure. The hanging parts of the elevated bridge are connected with neighbouring ones by rails and ballast in the actual structure, but only the connecting effect of the rails can be incorporated into analysis. Presumably for that reason, the vibrations are predominant at lower frequencies and analytical acceleration responses display larger amplitudes than do experimental ones at point-1. More detailed description and discussion of the dynamic responses of the viaducts are conducted in the Reference [10].

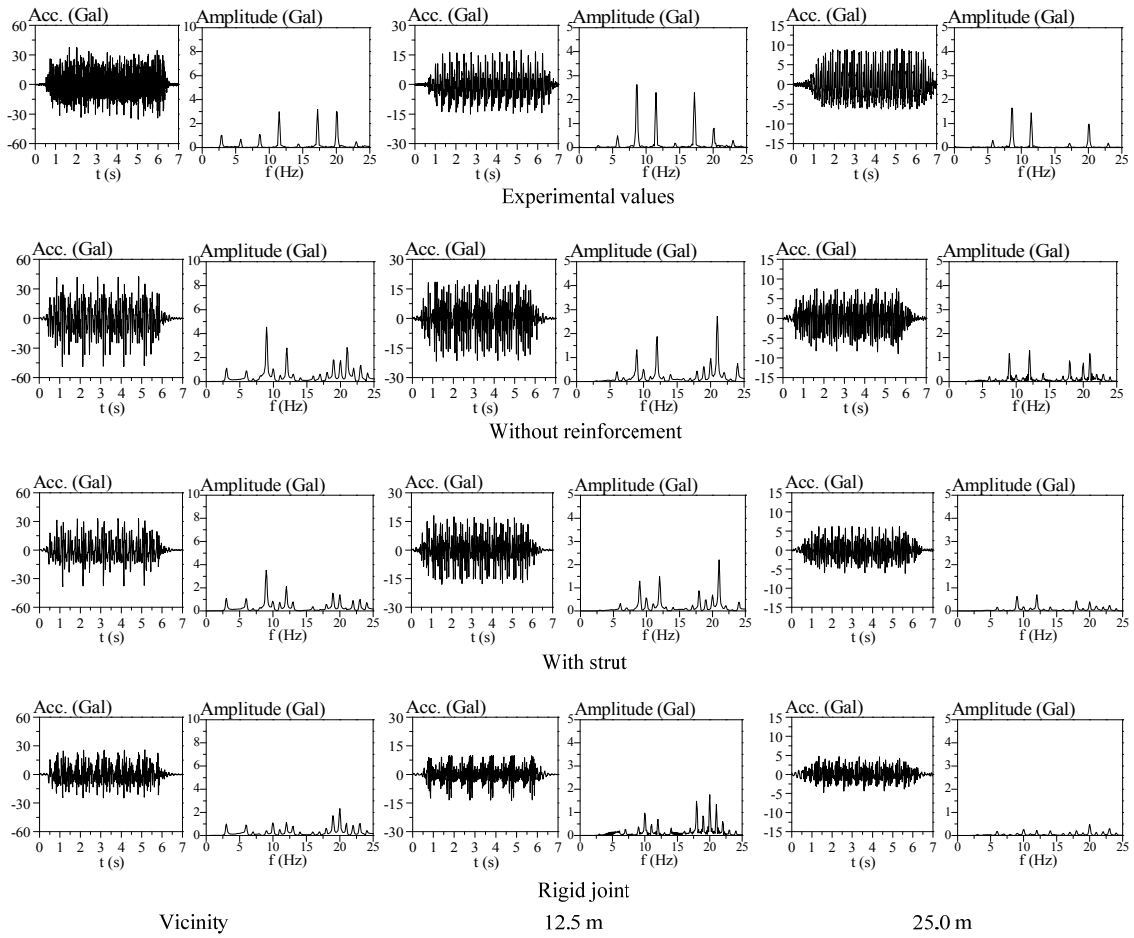


Figure 8: Ground acceleration response (Train speed 270 km/h).

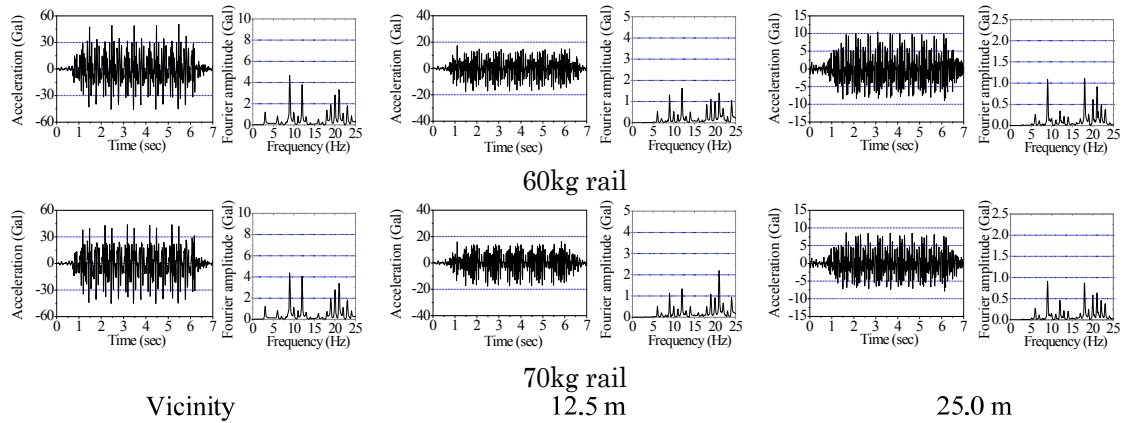


Figure 9: Ground acceleration response due to different rails.

	Maximum (Gal)			RMS (Gal)		
	Vicinity	12.5m	25m	Vicinity	12.5m	25m
60kg rail	48.13	17.44	9.67	15.49	6.43	3.83
70kg rail	44.47	16.92	8.25	15.24	6.42	3.21

Table 4: Ground acceleration values due to different rails.



## 4.2 Ground vibration response

Considering the predominant frequency components of the external forces that are confirmed within 15 Hz, the soil damping effect, and the efficiency of the analysis, the highest frequency considered in the analysis is determined as 25 Hz. By applying the dynamic reaction forces obtained in the bridge vibration analysis to 24 total footings, the site response analyses using the analytical models described previously is carried out with the SASSI2000 computer program [8].

Numerical results of points of Vicinity, 12.5 m and 25.0 m indicated in Figure 2 with the experimental values are shown in Figure 8, named as Without reinforcement. In spite of the complicated nature of the whole train-bridge-ground interaction system and the approximations or assumptions that have to be made in modelling the system, the analytical acceleration responses indicate comparatively good agreement with the experimental ones. The amplitudes of the analytical results are considerably coincident with the experimental ones. The components of the Fourier spectra of the analytical results to a certain extent re-create the experimental ones. Thereby the validity of the analytical procedure can be confirmed.

## 4.3 Reduction effect of proposed countermeasures

For the first countermeasure, the vertical acceleration responses simultaneously with their Fourier Spectra of the ground vibration before and after reinforcements are also shown in Figure 8. For both reinforcement methods, the acceleration responses decrease in comparison with those before reinforcements. Focusing on the frequency components, the responses in the higher frequency ranges can be confirmed to be decreased apparently. The reason is considered as that the lower frequency responses due to the structurally free hanging parts are reduced by the reinforcements. Furthermore, the rigid connection reinforcement method seems to be more effective compared with the strut reinforcement. This is not only because of the increased rigidity of the hanging parts, but also because that the independent bridge blocks are connected and become structurally continuous. Thus the high speed bullet train can run through the viaducts smoothly and the impact effect of the wheel loads can be mitigated. However, it is not realistic to completely connect the hanging parts rigidly because the structural type is changed and some mechanics problems may be induced. Therefore, in actual application of the reinforcement methods, a reinforcement structure similar to the strut reinforcement should be designed to realize a close effect like the rigid connection method.

For the second countermeasure, the vertical acceleration response of the 60kg and 70kg rails are shown in Figure 9, and the corresponding Maximum and RMS values of the accelerations are given in Table 4. From Table 4, it can be confirmed that the responses due to the 70kg rail are to some extent reduced compared with those of the 60kg rail. Therefore, if designed carefully, it can be also effective to use stiffer rails as the countermeasure of ground vibration reduction.

## 5 CONCLUSIONS

In this study, an analytical approach to simulate the dynamic response of the train-bridge-ground interaction system is established and the countermeasures to reduce the ground vibration around Shinkansen viaducts are proposed and numerically evaluated. The effectiveness of the developed analytical procedure to predict and evaluate the train-induced ground vibration problems, and then propose corresponding countermeasures is demonstrated. Such an analytical approach not only can be used in the case of investigating the environmental vibration problems in the existing viaducts, it is also possible to be employed to discuss in advance the dynamic problems that a proposed viaduct may induce in the design stage.

**REFERENCES**

- [1] Seki M., Inoue Y. and Naganuma Y.: Reduction of subgrade vibration and track maintenance for Tokaido Shinkansen, *WCRR '97* (Firenze, Italy), Vol. E, pp. 395-402, 1997.
- [2] Yoshioka O. and Ashiya K.: Dependence of Shinkansen-induced ground vibration upon their influence factors, *QR of RTRI*, Vol. 29, No. 4, pp. 176–183, 1988.
- [3] Xia H., Cao Y. M. Zhang N. and Qu J. J.: Vibration effects of light-rail train-viaduct system on surrounding environment, *International Journal of Structural Stability and Dynamics*, Vol. 2, No. 2, pp. 227–240, 2002.
- [4] Wu, Y. S. and Yang Y. B.: A semi-analytical approach for analyzing ground vibrations caused by trains moving over elevated bridges, *International Journal of Soil Dynamics and Earthquake Engineering*, Vol. 24, No. 12, pp. 949-962, 2004.
- [5] Hara T., Yoshioka O., Kanda H., Funabashi H., Negishi H., Fujino Y., and Yoshida K.: Development of a new method to reduce Shinkansen-induced wayside vibrations applicable to rigid frame bridges: bridge-end reinforcing method, *Journal of Structural and Earthquake Engineering*, JSCE, Vol. 68, No. 766, pp. 325-338, 2004. (In Japanese)
- [6] He X., Kawatani M. and Nishiyama S.: An analytical approach to train-induced site vibration around Shinkansen viaducts, *Structure and Infrastructure Engineering*, Vol. 6, No. 6, pp. 689–701, 2010. (First published on: 06 August 2008, DOI: 10.1080/15732470802087799.)
- [7] Lysmer J., Ostadan F. and Chin C.C.: *SASSI2000 theoretical manual – A system for analysis of soil-structure interaction*, Academic Version, University of California, Berkeley, 1999.
- [8] Lysmer J., Ostadan F. and Chin C.C.: *SASSI2000 user's manual – A system for analysis of soil-structure interaction*, Academic Version, University of California, Berkeley, 1999.
- [9] Kawatani M., He X., Sobukawa R. and Nishiyama S.: Traffic-Induced Dynamic Response Analysis of High-speed Railway Bridges, *Proceedings of the Third Asian-Pacific Symposium on Structural Reliability and Its Applications*, Seoul, Korea, pp. 569-580, 2004 (CD-ROM).
- [10] Kawatani M., He X., Shiraga R., Masaki S., Nishiyama S. and Yoshida K.: Dynamic response analysis of elevated railway bridges due to Shinkansen trains, *Journal of Structural and Earthquake Engineering*, JSCE, Vol. 62, No. 3, pp. 509-519, 2006. (In Japanese)
- [11] Chin C.C.: *Substructure subtraction method and dynamic analysis of pile foundations*, Ph.D. Dissertation, University of California, Berkeley, 1998.
- [12] Agabain M. E.: The Effect of Various Damping Assumptions on the Dynamic Response of Structure, *Bulletin of International Institute of Seismology and Earthquake Eng.*, Vol. 8, pp. 217-236, 1971.
- [13] Yoshida K. and Seiki M.: Influence of improved rigidity in railway viaducts on the environmental ground vibration, *Journal of Structural Engineering*, JSCE, Vol. 50A, pp. 403-412, 2004. (In Japanese)