VIBRATION EFFECTS OF LIGHT-RAIL TRAIN-VIADUCT SYSTEM ON SURROUNDING ENVIRONMENTS

He XIA
Professor
School of Civil Engineering & Architecture
Northern Jiaotong University
Beijing 100044, China
Fax:+86-10-63240217
e-mail: hxia@center.njtu.edu.cn

Nan ZHANG
Doctoral Student
School of Civil Engineering & Architecture
Northern Jiaotong University
Beijing 100044, China
Fax:+86-10-63240217
e-mail: hxia@center.njtu.edu.cn

Abstract: This paper studies the dynamic impacts of moving light-rail trains on viaduct bridges and the vibration effects of the surrounding ground. The study is based on theoretical analysis and field experiment. The theoretical study is performed by two models: the 2-dim interaction model of "train-bridge" system for obtaining the dynamic loads of moving trains on the bridge piers; and the 2-dim dynamic model of "pier-foundation-ground" system for analyzing the vibration responses of the surrounding ground. In the field experiment, the dynamic responses of bridge piers and the ground accelerations are measured at different distances and under different train speeds.

Keywords: Vibration, environment, light-rail, viaduct, experiment

1 INTRODUCTION

With the rapid development of modern industry, vibration effects on people's living and working environment in big cities have become more and more serious. The traffic-induced vibrations that seemed to have been tolerated in the past are today increasingly being considered as a nuisance. The vibration source, transmission path and control methods are all under study, among which the influences of light-rail train-borne vibration on the surrounding environment have also brought to the attentions of engineers and researchers in China and abroad (Tassily, 1988, Xia *et al*, 2000).

Today, light-rail traffic systems are under planning in several big cities in China, owing to its advantages of fast speed, high safety, on schedule in operation and its large transportation capacity. This paper attempts, based on theoretical analysis and field measurement, to study the properties of the ground vibrations induced by light-rail train and viaduct system.

2 THEORETICAL STUDY

In this paper, the theoretical study is performed by two dynamic models: a 2-dimensional "train-bridge" system model for obtaining the dynamic loads on piers and a "pier-foundation-ground" system model for analyzing the vibration responses of the surrounding ground.

2.1 Train-Bridge System Model

The analysis model for train-bridge interaction is a dynamic system composed of the bridge model and the vehicle model that are linked by an assumed wheel-track relation. As is shown in Fig.1, the viaduct bridge system consists of multi-plan simply-supported PC beams, column piers and rubber bearings. The modal comprehension analysis is performed for such a multi-span bridge system: First, the free vibration frequencies and modes of the system are solved. Upon the orthogonality of the modes, the coupled FEM equations of the bridge model become the superposition of independent modal equations. Owing to the fact that the dynamic response of a structure is dominantly influenced by its several lowest modes, this approach has a very great advantage that an adequate estimation of the dynamic response can be obtained by considering only a few modes of vibration, thus the computational efforts can be significantly reduced.

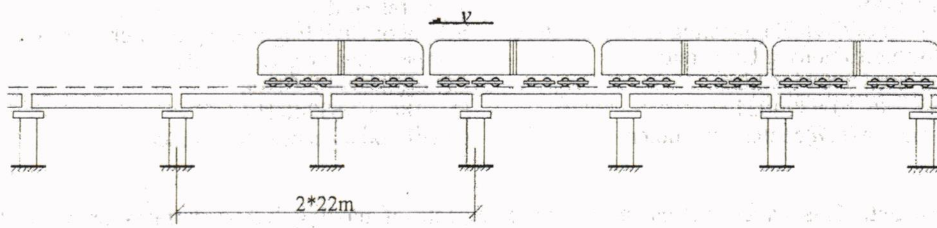

Fig.1 Dynamic Model of Train-Bridge System

According to the principle of modal superposition, the deflection of any section of the beam $Z_b(x)$ can be expressed by:

$$Z_b(x) = \sum_{n=1}^{N} q_n \phi_n(x) \tag{1}$$

where ϕ_n is the function of the nth bridge mode, q_n is the modal amplitude. When the modes are standardized in terms of $\phi_n^T m \phi_n = 1$, the nth modal equation becomes:

$$\ddot{q} + 2\omega\xi\dot{q} + \omega^2 = F_n \tag{2}$$

in which F_n is the generalized force of the nth mode.

The vehicle model is a light-rail train consisting of several 8-axle hinged vehicles. The vehicle model is established based on the following assumptions:

- The vehicle car body consists of two rigid parts hinged together;
- Effects of the elastic deformations of car bodies and wheel sets are neglected.
- The vehicle is simplified into the single-level-suspension system. The spring stiffness and the damping coefficient of a bogie are shared out among its wheel sets.

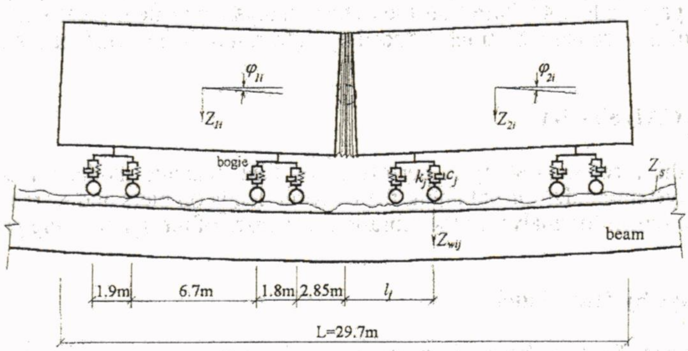

Fig.2 Dynamic model of vehicle

The idealization of the 8-axle hinged vehicle model is shown in Fig.2. For such a model, the configuration of each car body is specified by 4 degrees of freedom (floating Z_{1i}, Z_{2i} and nodding φ_{1i}, φ_{2i}); and that of each wheel set by 1 degree of freedom (floating Z_{wij}). Since the two parts of the vehicle is hinged, the independent degrees of freedom of a vehicle body are three, where φ_2 is expressed by the other 3 degrees of freedom as $\varphi_2=Z_2/L-Z_1/L-\omega_1$. Therefore the total analytical degrees of freedom are 3+8=11 for an 8-axle hinged vehicle.

The dynamic equilibrium equations of a vehicle is

$$[\mathrm{M}]\{\ddot{\mathrm{v}}\}+[\mathrm{K}]\{\dot{\mathrm{v}}\}+[\mathrm{K}]\{\mathrm{v}\}=\{\mathrm{F}\} \tag{3}$$

where $[M]$, $[K]$ and $[C]$ are respectively the mass, stiffness and damping matrices, $\{v\}$ is the displacement vector and $\{F\}$ the force vector. The wheel-set movements Z_{wij} and the beam displacements $Z_b(x_{ij})$ satisfy the relation:

$$Z_{wij}=Z_b(x_{ij})+Z_s(x)=\sum_{n=1}^{N} q_n \phi_{ij}^n + Z_s(x) \tag{4}$$

where Z_s is the vertical displacements of the wheel relative to the rail, representing the vertical rail irregularities of the track on the bridge. The wheel movements can then be expressed by the linear composition of the bridge modal coordinates q_n $(n=1,2,\ldots N_q)$.

The dynamic equilibrium equations of the vehicle-bridge system can be obtained by the composition of the vehicle equations and the bridge equations:

$$\begin{bmatrix} M_i+\frac{J_2}{L^2} & -\frac{J_2}{L^2} & \frac{J_2}{L} \\ -\frac{J_2}{L^2} & M_i+\frac{J_1}{L^2} & -\frac{J_2}{L} \\ \frac{J_2}{L} & -\frac{J_2}{L} & J_1+J_2 \end{bmatrix} \begin{Bmatrix} \ddot{Z}_{1i} \\ \ddot{Z}_{2i} \\ \ddot{\varphi}_{1i} \end{Bmatrix}$$

$$+\begin{bmatrix} \sum_{j=1}^{4} c_j+\sum_{j=5}^{8}\frac{l_j^2}{L^2}c_j & -\sum_{j=5}^{8}\left(\eta\frac{l_j}{L}+\frac{l_j^2}{L^2}\right)c_j & -\sum_{j=1}^{4}\eta l_j c_j+\sum_{j=5}^{8}\frac{l_j^2}{L}c_j \\ -\sum_{j=5}^{8}\left(\eta\frac{l_j}{L}+\frac{l_j^2}{L^2}\right)c_j & \sum_{j=5}^{8}\left(1+\eta\frac{l_j}{L}\right)^2 c_j & \sum_{j=5}^{8}\left(\eta-\frac{l_j}{L}\right)c_j \\ -\sum_{j=1}^{4}\eta l_j c_j+\sum_{j=5}^{8}\frac{l_j^2}{L}c_j & \sum_{j=5}^{8}\left(\eta-\frac{l_j}{L}\right)c_j & \sum_{j=1}^{8} l_j^2 c_j \end{bmatrix} \begin{Bmatrix} \dot{Z}_{1i} \\ \dot{Z}_{2i} \\ \dot{\varphi}_{1i} \end{Bmatrix}$$

$$+\begin{bmatrix} \sum_{j=1}^{4} k_j+\sum_{j=5}^{8}\frac{l_j^2}{L^2}k_j & -\sum_{j=5}^{8}\left(\eta\frac{l_j}{L}+\frac{l_j^2}{L^2}\right)k_j & -\sum_{j=1}^{4}\eta l_j k_j+\sum_{j=5}^{8}\frac{l_j^2}{L}k_j \\ -\sum_{j=5}^{8}\left(\eta\frac{l_j}{L}+\frac{l_j^2}{L^2}\right)k_j & \sum_{j=5}^{8}\left(1+\eta\frac{l_j}{L}\right)^2 k_j & \sum_{j=5}^{8}\left(\eta-\frac{l_j}{L}\right)k_j \\ -\sum_{j=1}^{4}\eta l_j c_j+\sum_{j=5}^{8}\frac{l_j^2}{L}k_j & \sum_{j=5}^{8}\left(\eta-\frac{l_j}{L}\right)k_j & \sum_{j=1}^{8} l_j^2 k_j \end{bmatrix} \begin{Bmatrix} Z_{1i} \\ Z_{2i} \\ \varphi_{1i} \end{Bmatrix}$$

$$=\sum_{n=1}^{N}\begin{Bmatrix} \sum_{j=1}^{4}\phi_{ij}^n(k_j q_n+c_j\dot{q}_n)-\sum_{j=5}^{8}\eta\frac{l_j}{L}\phi_{ij}^n(k_j q_n+c_j\dot{q}_n) \\ \sum_{j=5}^{8}\left(1-\eta\frac{l_j}{L}\right)\phi_{ij}^n(k_j q_n+c_j\dot{q}_n) \\ -\sum_{j=1}^{8}\eta l_j\phi_{ij}^n(k_j q_n+c_j\dot{q}_n) \end{Bmatrix}$$

$$
+\left\{\begin{array}{l}
\sum_{j=1}^{4}[k_j Z_s(x_{ij})+c_j \dot{Z}_s(x_{ij})]-\sum_{j=5}^{8}\eta\frac{l_j}{L}[k_j Z_s(x_{ij})+c_j \dot{Z}_s(x_{ij})] \\
\sum_{j=5}^{8}\left(1-\eta\frac{l_j}{L}\right)[k_j Z_s(x_{ij})+c_j \dot{Z}_s(x_{ij})] \\
-\sum_{j=1}^{8}\eta l_j[k_j Z_s(x_{ij})+c_j \dot{Z}_s(x_{ij})]
\end{array}\right\}
$$

$$
\ddot{q}_n+2\xi_n\omega_n\dot{q}_n+\omega_n^2 q_n=\sum_{i=1}^{N_v}\phi_{ij}^n\left\{-\sum_{j=1}^{8}\sum_{m=1}^{N}\phi_{ij}^m[m_{ij}\ddot{q}_m+c_i\dot{q}_m+k_i q_m]\right.
$$

$$
+\sum_{j=1}^{4}(k_j Z_{1i}+c_j\dot{Z}_{1i})-\sum_{j=5}^{8}\eta\frac{l_j}{L}(k_j Z_{1i}+c_j\dot{Z}_{1i})+\sum_{j=5}^{8}\left(1+\eta\frac{l_j}{L}\right)(k_j Z_{2i}+c_j\dot{Z}_{2i})
$$

$$
\left.-\sum_{j=1}^{8}\eta l_j(k_j\varphi_{1i}+c_j\dot{\varphi}_{1i})-\sum_{j=1}^{8}[m_{ij}\ddot{Z}_s(x_{ij})+c_j\dot{Z}_s(x_{ij})+k_j Z_s(x_{ij})]+\frac{1}{8}g(M_i+m_{ij})\right\}
$$

$$
(i=1,2,\cdots\cdots N_v,\qquad n=1,2,\cdots\cdots N_q) \tag{5}
$$

Since the coefficients "ϕ" is always changing when the train runs on the bridge, the dynamic equations of the vehicle-bridge system are 2-order linear simultaneous differential equations with time-varying coefficients. The equations are solved by the *Newmark* β step by step integration algorithm with β=1/4. When the generalized coordinates $q_n(t)$ is obtained, the reaction $R_r(t)$ of support r can be determined by

$$
R_r(t)=\sum_{n=1}^{N}q_n(t)\phi_r^n\cdot 4S_r\cdot E_r/H_r \tag{6}
$$

where ϕ_r^n is the value of the nth modal function of the beam at support r; S_r, E_r and H_r are cross area, elastic modulus and thickness of the rubber bearings.

2.2 Dynamic Loads of Trains on Bridge Piers

By the vehicle-bridge system model, the whole histories of a train running on the bridge is simulated on computer. Two spans are taken as the calculation part, in which six modes are considered for each span. The train speed is 70km/h; the integration time interval is 0.005sec.

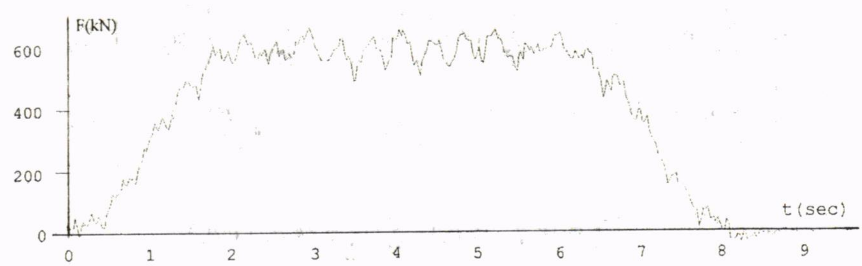

Fig.3 Load history on pier induced by moving train

Fig.3 illustrates a calculated time history of the dynamic reaction force acting on the pier. The figure clearly shows the dynamic behaviour of the force on the pier when the train is running on the bridge.

The principal frequency of the force wave is approximately 2.6Hz. It is just the loading frequency $f = v/L_a = 19.44 \div 7.425 \approx 2.6$Hz of the train (with the average interval of bogies L_a=7.425m and the train speed v=70/3.6=19.44m/s), which indicates that the loading frequency of the train plays an important role on the forces.

Table 1 Reaction of Beam Supports and Their Dynamic Factors

Reactions	Support 1	Support 2	Resultant reaction (1+2)
Static (kN)	363.9	363.9	591.5
Dynamic (kN)	435.0	435.0	705.0
Dynamic factor	1.195	1.195	1.192

Table 1 lists the comparison of the static and dynamic reactions and their dynamic factors. Since the two support reactions on a pier are in different phases (with the interval 1.10m between the two supports on one pier, the average bogie distance 7.425m, the phase difference between the two support reactions is about 170° when v=70km/h, approximately in inverse phases), the dynamic loading (resultant reaction 1+2) of the train on the pier is much smaller than the sum of the two individual supports.

Table 2 Comparison of reactions for different trains

Number of vehicles	1	2	3	4	4 (without rail irregularities)
Max reaction	690	705	705	705	695

Table 2 shows the maximum reactions induced by different train combinations. In spite of great differences between the dynamic waves for trains of two vehicles and four vehicles, the maximum reactions remain the same, which is because that the loading length of two vehicles has been longer than the total length of the two spans. In such case, the more the vehicles of a train, the longer the stable part of the dynamic reaction curves becomes, while the maximum value of the reaction is not influenced. As for a train with only one vehicle, the load can not be distributed to the whole bridge, the maximum reaction, therefore, is only 84% of the other three cases.

Table 3 Maximum reactions in different load combinations

Maximum reactions	Running on one line	Two lines and same directions		Two lines but opposite directions	
		Simultaneously	Insimultaneously	Simultaneously	Insimultaneously
Max. (kN)	705	1410	1410	1410	1410
Min. (kN)	--	--	1176	--	1176

Since the load combinations for trains running on double lines are very complicated, it is difficult to consider all the possible cases, instead, only some special cases are taken into account, see Table 3. The table shows that the most unfavourable combination is when two trains run on both lines and simultaneously onto the bridge whether in the same or in opposite directions. The results of other load combinations should be enveloped within these results, which are between 1410kN (two times of loading on one line) and 1176kN (1.6 times of one line), depending on the time interval of the two trains onto the bridge.

2.3 Dynamic Model of Pier-Foundation-Ground System

When the dynamic forces acting on piers are obtained, the ground vibrations can be further studied by the pier-foundation-ground system model. The model is also 2-dimensional, established on the basis of the following assumptions:

- ♦ The foundation and the surrounding soils keep close contact during the whole course of response;
- ♦ The calculation is limited in elastic state;

As is shown in Fig.4, the ground range under study is $L \times H$ = 104m$\times$18m. The piers and the foundations are modelled as beam elements, while the surrounding ground as quadrangular elements. The train consists of four vehicles, for the damping of ground is high and longer stable vibration has to be considered. The train and the beams are treated as a concentrated mass on the top of the pier.

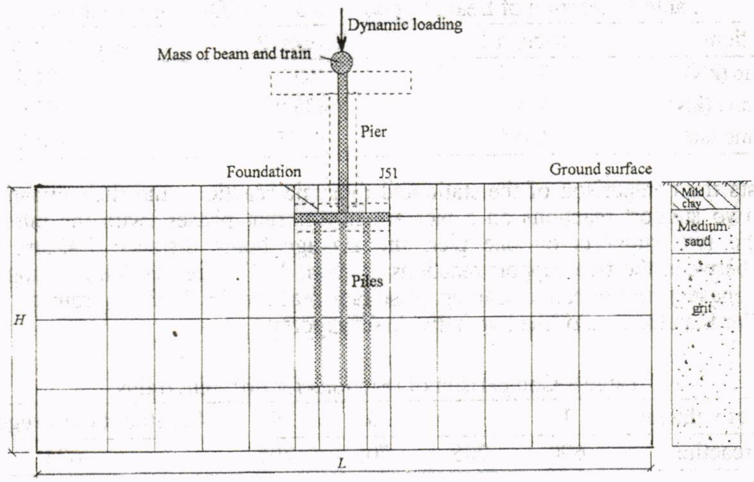

Fig.4 The 2-dimensional pier-foundation-ground system model

2.4 Vibration of Ground

The calculation is carried out by the general structural analysis program SAP84. Fig.5 shows the typical dynamic responses of vertical acceleration, velocity and displacement of the ground surface at joint J51 near the pier on shallow foundation. The time histories of responses at other points are almost the same, only the magnitudes are different. Fig.6 is the distribution of maximum acceleration and velocity responses of ground surface versus the distance to the pier on shallow foundation and pile foundation. It can be concluded that the calculation range of FEM should be large enough to reduce the boundary damping and reflecting effects.

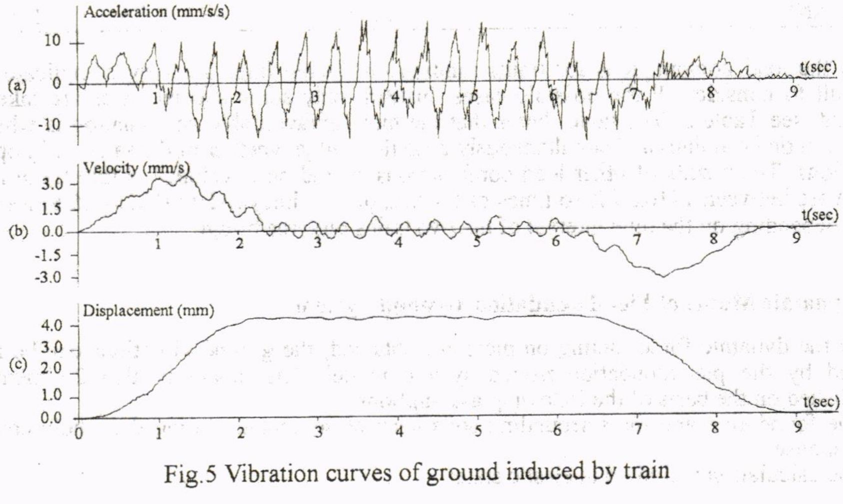

Fig.5 Vibration curves of ground induced by train

The FEM network for pile foundation is the same as that for shallow foundation. The pile length is 13m. The shapes of the vibration curves are almost the same as those in shallow foundation. One can see from the figures that the ground vibrations are much smaller than those in shallow foundation. Compared with the results of shallow foundation, the calculated acceleration and velocity are only 2.51/4.02=1/1.6 and 0.24/1.13=1/4.7, respectively.

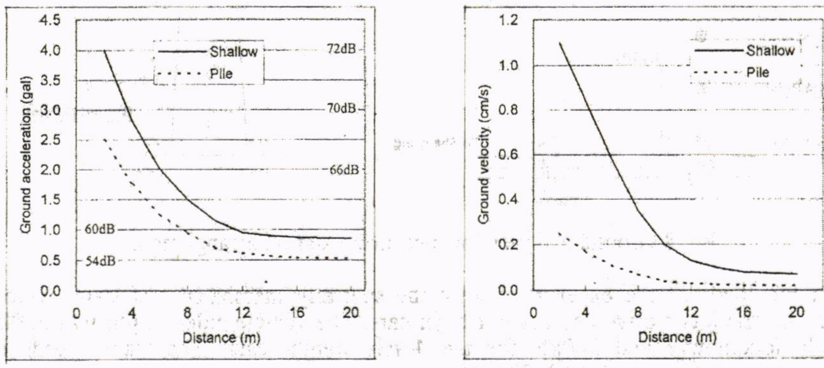

Fig.6 Distribution of maximum response versus distance to the pier

3 EXPERIMENTAL STUDY

The experiment was carried out at the Daqinghe Bridge site. The purpose of this experiment was to study the vibration level and the distribution laws of the train induced ground vibration at different distances and under different train speeds.

3.1 Introduction to the Experiment

The Daqinghe Bridge is located at an ordinary railway line in Northeast China (Fig.7). It has a full length of 750m, composed of 22 spans and 21 piers. Each span consists of two pieces of I-steel-plate-beams of 33.6m in length and 2.61m in height. The cross center-to-center distance between the two beams is 2.0m. The web thickness of the I-beam is 12mm. Its top and bottom flange thickness varies from 15mm to 45mm. The bracing systems connecting the two I-beams are made of angle and slot steel members. The heights of the concrete bridge piers are from 6m to 10m. The ground soil of the site is fine sand and clay.

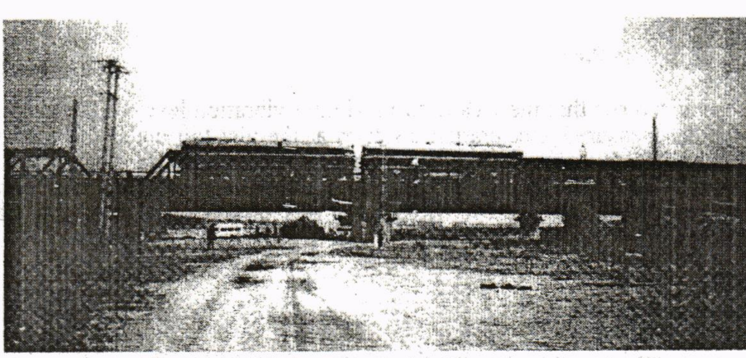

Fig.7 Experiment site at Daqinghe Bridge

In the experiment, the accelerations of the pier top and the surrounding ground at some points were measured by accelerometers when the test train was running on the bridge. The measuring stations set on the pier top and the ground at the perpendicular distances to the bridge of 0m (foot of the pier), 20m, 40m and 60m are outlined in Fig.8.

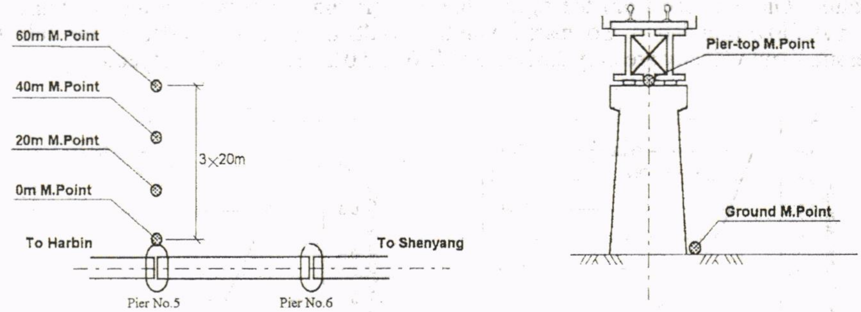

Fig.8 Ground and pier-top measuring station arrangement

The dynamic loads of the experiment were the specially marshaled test train, which was composed of one locomotive and seven freight cars. The vehicle axle-weight was 230kN for the 6-axle locomotive and 200kN for the 4-axle freight cars. The train speeds in the experiment were in the range of 60~80 km/h.

Totally, 28 groups of experiments were carried out during the experiment. The maximum and the average vibration levels of the bridge pier and the ground at different distances under different train speeds were obtained from the measurement, which are listed in Table 4.

Table 4 Measured pier and ground acceleration levels (dB)

Train speed (km/h)	Pier top		Ground at distances to the bridge pier-foot center (m)							
			0		20		40		60	
	Max	Ave	Max	Ave	Max	Ave	Max	Ave	Max	Ave
60	84.8	83.9	80.5	78.9	67.6	65.6	58.2	54.6	43.3	41.4
65	86.5	86.1	89.2	87.3	79.0	77.3	67.8	63.4	51.9	50.2
70	88.0	87.2	91.9	89.5	82.9	79.8	67.1	63.4	53.0	50.9
75	90.0	89.5	94.3	91.5	87.4	85.1	70.2	67.6	54.2	51.7
80	91.6	91.3	100.5	96.1	89.2	86.9	68.3	65.0	57.7	52.7

The vibration levels are calculated on the basis of $G = 20\lg(a) + 60$, where G is the vibration level, in dB; a is the measured acceleration, in cm/s^2.

3.2 Vibration of Bridge Pier

From Table 4 one can see that the measured maximum vibration level of the pier is 91.6dB. Two of the typical measured acceleration histories at the pier-top under the train speeds of 60km/h and 80km/h are shown in Fig.9 (a) and (b), respectively. It is clear that with the increase of the train speed, the accelerations of the pier-top become greater. There are 7-8dB differences of maximum and average vibration levels of the pier under the train speed difference of 20km/h.

3.3 Vibration of ground

Some typical measured acceleration histories of the ground at the foot of the pier (0m) and 40m to the bridge under the train speed of 50km/h are shown in Fig.10. The distribution of the maximum ground vibration levels induced by the passing trains at different speeds *versus* the distance to the bridge is shown in Fig.11.

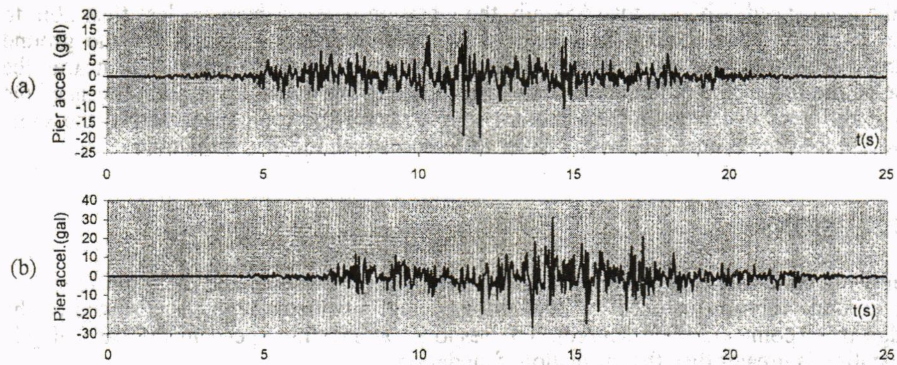

Fig.9 Measured acceleration waves at pier top

The main trend is that the vibration level attenuates linearly with the increase of the distance to the pier (point vibration source). The highest maximum vibration level is 100.5dB, occurring at the foot of the pier (0m). The measured data show 37-43dB decrease for the vibration levels in the distances from 0 to 60m at different test train speeds. The distribution of the average vibration levels at different train speeds versus the distance to the vibration source has the same trends as the maximum ones, but the maximum average value is 96.1dB.

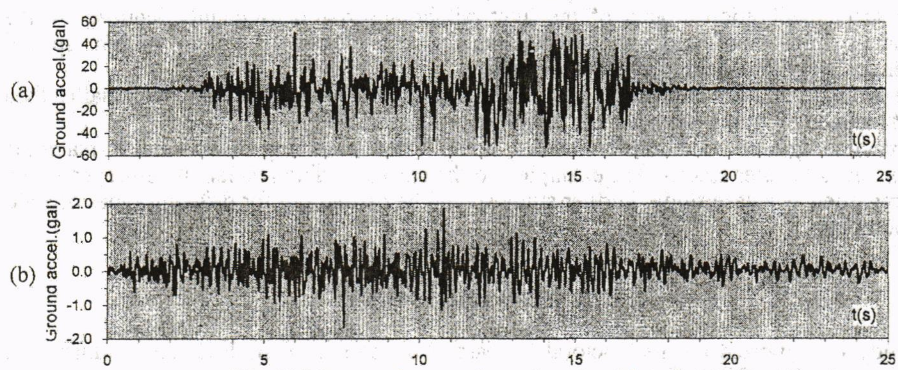

Fig.10 Measured ground acceleration histories

The distribution of the maximum ground vibration levels at different distances to the bridge pier *versus* the train speed is shown in Fig.12. Within the train speed range from 60km/h to 80km/h, the ground vibrations at different distances have the main trend of increasing with the train speed.

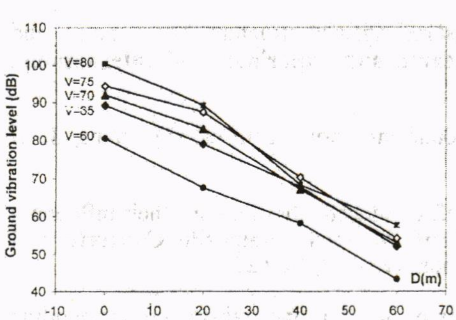

Fig.11 Vibration vs distance

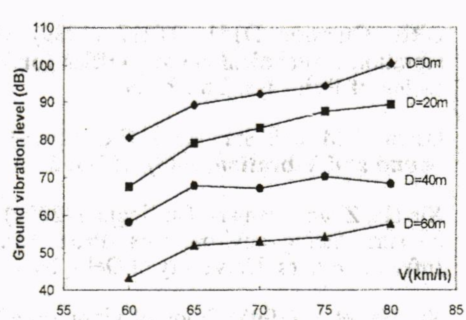

Fig.12 Vibration vs train speed

When the train speed is higher than 65km/h, the vibration level of the point less than 20m to the pier is greater than 80dB. The results show 10dB to 20dB increases for the ground vibration levels when the train speed increased from 60km/h to 80 km/h. The nearer the distance is, the bigger the difference is. The distribution of the average vibration levels of the ground at different distances to the vibration source *versus* the train speed shows the same trends as the maximum ones, but with smaller differences than those in maximum ones.

4 CONCLUSIONS

The following conclusions have been drawn from the theoretical study and the experiment:

- The ground vibration induced by light railway train attenuates with the distance to the bridge pier (point vibration source). The response attenuation speed in the case of pile foundation is greater than that in shallow foundation.
- The ground vibration induced by running trains near a bridge increases with the train speed. When the speed of the test train changes from 60km/h to 80km/h, the maximum vibration level of the ground near the pier foot is 57.5dB-77.5dB.
- The type of bridge foundations of light rail system has very important influences on the ground vibration. Using pile foundations can greatly reduce the responses induced by moving train loads.
- For a light rail system in the example where rubber bearings are adopted, the maximum velocities at ground surface 5.2m from the pier are 0.63cm/s and 0.129cm/s for shallow foundation and pile foundation, respectively.
- The ground vibration around of the light-rail bridge system is mainly induced by the dynamic impacts of trains as moving loads. In the design of bridge system, special attention should be paid to avoid resonance.
- The ground vibration induced by light rail trains and viaduct system is much smaller than that induced by trains (with heavier loads) in ordinary railway.
- In practice, ground conditions are usually complicated, assuming soils as elastic materials often leads to safer results. The example and the corresponding results, therefore, can only be regarded as a case study. Further study should be carried out for different types of field conditions.

ACKNOWLEDGEMENT

This study is supported by the Doctoral Research Foundation of the Education Ministry of China (Number 2000000407).

REFERENCES

1. Tassily, E. (1988) Interaction dynamique voie/roue: modeles existants et perspectives de recherche, **Revue Generale des Chemins de Fer, 107**, 23-30,

2. ORE. Question D151 DT217 (1989) Measures against structure borne noise and vibrations. **Technical report, Office for Research and Experiments of International Union of Railways**, Utrecht, NL.

3. Dawn, T.M. and Stanworth, C.G. (1979) Ground vibrations from passing trains, **J. of Sound and Vibration, 66(3)**, 355-362.

4. Xia He, Zhang Nan and Qu Jingjian (2000) Traffic-induced vibrations and their influences on surrounding environments. **Proc. Int. Conf. Modern Composite Concrete and Infrastructures**, University of Delft, Netherlands, 123-130, Nov 2000.

5. Xia He, et al (1995) Study of vibration effects of underground trains upon surrounding environments. **Advances in Structural Engineering**, CRPH, 116-122.