Evaluation of ground vibration induced by an urban train system

YIT-JIN CHEN[1], YI-JIUN SHEN[2] & SHENG-HUOO NI[3]

[1] *Chung Yuan Christian University. (e-mail: yjc@cycu.edu.tw)*
[2] *China Engineering Consultants, Inc.. (e-mail: yjs@ceci.org.tw)*
[3] *National Cheng-Kung University. (e-mail: tonyni@mail.ncku.edu.tw)*

Abstract: In recent years, more and more vibration sensitive buildings are located near the planned route of urban train systems, thus the ground vibrations induced by train systems have drawn engineer's interest. In previous research this area has focused on the vibration characteristics and mitigation methods. Research on these two subjects has progressed well. However, the evaluation methods for the train-induced vibration among current assessments present obvious differences.

The main purpose of this research is to examine these evaluation methods using field measured data. The evaluation methods from different available representative assessments are reviewed briefly, and then a series of field vibration measurements of urban train systems were compared with these evaluation methods.

For the site vibration measurement, the three-dimensional Tokyo Sokushin Model 15D servo-type velocity transducer will be used to measure the velocity level of vibration. The sampling frequency used for time domain data is 512 Hz. The HP 3565S Vista system is mainly used to collect the data and to analyze the dynamic signals. The frequency response is presented using one-third octave band for the central frequency ranged from 1 to 100 Hz.

The measurements include a wide variety of soil and structure conditions. The main vibration sources include rapid transit system and ordinary railway system with different speeds on different train supporting structures, such as on concrete or steel bridges. A broad base of geological conditions (including soil, gravel and rock) and foundation types such as spread footing and pile foundation are used for this evaluation. These various conditions present different vibration characteristics. Based on this evaluation, the suitability of each vibration assessment method is presented and some recommendations for further studies are proposed.

Résumé: Les ingénieurs donnent une grande attention aux tremblements de terre due à la système de voie, parce qu' aujourd'hui l'on a de plus en plus de batiments sensibles de tremblement qui y sont à l'entour. Bien que les recherches sur les caractéristiques des tremblements et sur les mesures d'absorption de tremblement aient obtenu des progressions, les différences sur les mesures d 'évaluation de tremblements du transport de voie entre les pays du monde vaux la peine qu'on les discute.

Cette recherche est de vérifier si les mesures d 'évaluation de tremblements de voie des pays du monde sont adaptables en utilisant les informations de tremblements de terre qu'on a ramassé sur place. Dans cette article, on va d'abord faire l'introduction des mesures d 'évaluation de tremblements du transport de voie, puis les compare par les informations de tremblements de terre qu'on a obtenu sur place. Les systèmes principales du transport de voie qu'on en a fait des mesurages sont les systèmes du metro et celles de voie ferré. Les résultats de mesurages sur les tremblements sont obtenus sous les différentes conditions de vitesses, de matières de construction des ponts, de types fondamentales des systèmes et de situations géoloqiques.

Keywords: vibration, railroads, environmental impact, environmental protection, bridges, infrastructure

INTRODUCTION

In the recent decades, the urban train system has been widely used as a main transportation solution within a city and its region in several countries across the world. However, the ground vibrations induced by the train system may cause disturbance and discomfort to the occupants. Experience shows that the building vibration induced by the train system can reach levels that cause human annoyance, possible damage to old and historical buildings, and interruption of sensitive instrumentation and processes. Therefore, the ground vibration problems have drawn engineer's highly interest in recent years. The engineers have to study the vibration characteristics so as to find the solution schemes of vibration mitigation.

In this paper, the vibration characteristics induced by the train system and some related vibration assessments are introduced. The evaluation methods from different available representative assessments were reviewed briefly, and then a series of field vibration measurements of urban train systems were compared with these evaluation methods. Several field measurement results are presented to show different vibration amplitudes for various structures and soil types. For bridge structures, many factors are found to affect the train-induced ground vibrations. It is anticipated that the vibration characteristics of concrete structures differ from those of steel structures. The paper will present the main key points of these two different structures. In addition to the different bridge types, various different underground geological conditions are encountered. Obviously, the different vibration characteristics will be obtained for the different foundation conditions. The response of structure foundation located on both layered soil and rock will be compared in the paper. The main vibration characteristics are focused on the frequency contents and vibration levels from the results of the field measurements.

LITERATURE REVIEW OF AVAILABLE EVALUATION METHODS

Vibration impact assessment of U.S.

The U.S. Department of Transportation (FTA 1995, FRA 1998) developed the manuals to provide guidance of vibration impact assessment for rapid transit and high-speed rail. For the case of high-speed rail project, some important factors for the vibration assessment are discussed in this section.

Screening Distances

The screening procedure uses Table 1 to determine whether the vibration-sensitive land uses are close enough for the impact from ground-borne vibration. The screening distances were decided by the train speed, the frequency of trains and the designated land use. More detailed analysis is required if any sensitive land uses are within the screening distances.

Table 1. Screening Distances for Vibration Assessments (FRA 1998)

Land Use	**Train Frequency***	**Screening Distance, ft**		
		Train Speed		
		Less than 100 mph	**100 to 200 mph**	**Up to 300 mph**
Residential	**Frequent**	120	220	275
	Infrequent	60	100	140
Institutional	**Frequent**	100	160	220
	Infrequent	20	70	100

*Frequent is greater than 70 pass bys per day.
Infrequent is less than 70 pass bys per day.
1 ft = 0.3048 m

Developed the generalized ground-borne vibration based curve

According to the manual of U.S. Department of Transportation, the general level of assessment uses generalized data to develop a curve of vibration level as a function of distance from the track. The vibration levels at specific buildings can be estimated by reading values from the curve and applying adjustments to account for factors such as the track support system, train speed, track and wheel condition, type of building, and receiver location within the building. The general level deals only with the overall vibration velocity level and it does not consider the frequency spectrum of the vibration.

Factors affecting vibration source

Train speed:

The levels of ground-borne vibration vary approximately at 20 times the logarithm of speed. The relationship was used to calculate the adjustments for other speeds are listed as follows:

adjustment (VdB) = 20 log[V(speed)/V($speed_{ref}$)] (FRA 1998)

This relationship means that doubling train speed will increase the vibration levels approximately 6 decibels and halving train speed will reduce the levels by 6 decibels.

Transit:

The levels of ground-borne vibration generated by a train pass by also depend on the transit's suspension system, wheel condition, and wheel type. The vehicle suspension consists of springs and dampers that affect the vibration transmitted to the track support system by the wheel/rail interaction. The stiff springs tend to increase the frequency and amplitude of vibrations.

Track structure:

The weight and size of the supported track structure affects the vibration radiated by that structure. Vibration levels will generally be lower for heavier supported track structures. Hence, the vibration levels from a cut-and-cover concrete double-box tunnel can be assumed to be lower than the vibration from a lightweight, concrete-lined bored tunnel. Whether or not the tunnel will be founded in bedrock is another factor affecting the radiated vibration. Bedrock is considered to be hard rock. It is usually appropriate to consider soft siltstone and sandstone to be more similar to soil than hard rock. Whether the tunnel is founded in soil or rock will make up to a 15 decibel difference in the vibration levels. The vibration from aerial structures is lower than from at-grade track because of the mass of the structure and the extra distance that the vibration must travel before it reaches the receiver.

Factors affecting vibration path

The geologic conditions are known to have a significant effect on the vibration levels, however it is difficult to develop more than a broad-brush understanding of the vibration propagation characteristics on general assessment stage. Some geologic conditions are likely associated with efficient propagation. For example, shallow bedrock, less than 30 feet (≒9 m) below the surface, is likely to cause efficient propagation. Other factors that can be important are soil type and stiffness. In particular, soils with heavy clay content have sometimes been associated with efficient vibration propagation too. Investigation of soil-boring records can be used to estimate depth to bedrock and the presence of problem soil conditions. If there is any reason to suspect efficient propagation conditions, then a detailed analysis during final design should include vibration propagation tests at the areas identified as potentially efficient propagation sites.

Factors affecting vibration receiver

Vibration generally reduces in level as it propagates through a building with a 1- to 2-decibel attenuation per floor. Resonances of the building structure, particularly the floors, will tend to counteract this attenuation and will cause some amplification of the vibration.

Basic features of train-induced vibration in Japan

In Japan, the studies in the train-induced vibration are mainly for Shinkansen, the name of high-speed rail in Japan. Many factors are found to influence the ground vibration amplitude and spectrum induced by the train system. The main features of the train-induced vibrations for the Shinkansen rail system are briefly described in the section. (Yoshioka, 2000).

Dependence on the characteristics of cars

The characteristics of cars related to ground vibration are car weight, car number, car length, wheelbase, bogie-centres distance and train speed. The original axle load of Shinkansen was 16T, but for 300, 500, 700 series, the axle loads have been remodelled to 11T to reduce the ground vibration. The overall vibration level of Shinkansen ranges from 63~57dB, and the measured distance from the track is 12.5m. Figure 1 illustrates the 1/3-octave band VL-z spectrum of Shinkansen vibration measured at 10m away from the track, which is averaged over 103 sites after normalization of each overall value to 0 dB (Yoshioka, 2000). The site condition is: train speed 200km/hr, ballast track, rigid-frame bridge and alluvial ground. The acceleration spectrum shows three visible peaks in 6.3, 16~20 and 40~50Hz, and the 16~20Hz are the most dominant peak. Yoshioka (2000) interprets these three peaks as the periodicity of axle arrangement and speed of train.

$f_1=V/d_{1(\text{distance of axles})}$, $f_2=V_{\text{train-speed}}/d_{2(\text{distance of bogie-center})}$, $f_3=V_{\text{train-speed}}/d_{3(\text{car length})}$

The periodic effects of axle arrangement on different train speed decided the peak frequency of the train-induced vibration spectrum.

The relations of train speed and vibration level are developed by the measured data. The equation is listed as follows.

$$VL=10\ n \log_{10}(v/v_0) + VL_0$$

where v_0 is a reference speed, VL_0 is an average VL for the speed v_0, and n is a coefficient to represent the incremental rate of VL values to speeds. The n value ranges from 1.5 to 3.5, depending on the measured results of different rail lines.

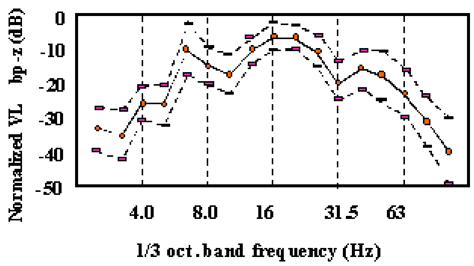

Figure 1. One-third octave band VL-z spectrum of Tokaido-Shinkansen-induced vibration (Yoshioka, 2000)

Dependence on the conditions of track and structure

Around 20 years ago, Ejima (1980) conducted a large-scale measurement for the issue of bridge structure sections. The selected structure types are rigid-frame bridge structures. For the track system, the results show that the vibrations in ballast tracks are somewhat smaller than the ones in slab track. The results also demonstrate that the overall vibration level in vertical direction (VL-oa-z) decreases as concrete volume in the superstructure becomes larger. The overall vibration levels in vertical direction are dependent on the bending rigidity of beams. The bending rigidity is evaluated by nI/L (n: number of beams, I: moment of inertia for vertical bending of beam, L: bridge span length). It shows that the VL-oa-z values also decrease when the bending rigidity becomes larger. For the substructure, the results demonstrate that VL-oa-z value becomes smaller when concrete volume in the substructure of bridge increases. The substructures of bridges are defined as the footings and beams in ground of the bridges, and the volume is summed up in one span range of bridge.

Dependence on the geological condition

On examination of Ejima's data, Yoshioka (2000) concluded that the ground velocity level and the attenuation are positively correlated for the near field vibration, i.e., for the distance of 5m to 50m from the track centre. A bridge foundation built in soft ground may be more easily excited to vibration than the one in hard ground, however, the near field vibration of high-frequency attenuation is also larger in soft ground than the case in hard ground.

MEASUREMENT METHOD

Measuring equipment

The measuring equipment for this study includes velocity transducer and data acquisition system. The three-dimensional (3D) Tokyo Sokushin model 15D and model VSE11 (horizontal) and VSE12 (vertical) servo-type velocity transducers were used to measure the velocity levels of vibration. The resonant frequency for the 15D and VSE11/12 are 0.1 Hz and 0.025 Hz, respectively. The calibration factor for both models of transducers is 10 Volt/cm/sec. The resolution of these transducers for 12 bit A/D converter with the full range of 1.26 mV is 1.26mV/4096 = 0.3076 micro-V (or 0.3076 micro-mm/sec)(Ni, 1999).

The HP 3565/7A Vista system with six channels input is mainly used to collect the data and to analyze the dynamic signals. The sampling frequency used for time domain measurement is 512 Hz. The frequency response of the measurement is presented using one-third Octave band for the central frequency ranged from 1 to 100 Hz.

Measuring procedure

The following procedure is used to measure the ground vibration for each location:

1. Place firmly the three-dimensional sensor on the ground besides the bridge column and the requested distances.
2. Set the sensor direction: the X-direction is in the car moving direction; the Y-direction is perpendicular to the train moving direction; the Z-direction is for the direction of gravity.
3. Configure the measuring system and set the measuring parameters.
4. Trigger the measuring system as the transit train passing.
5. Measure ten and one passing, respectively, of the mass rapid transit (MRT) and ordinary railway (OR) trains for each location.

The typical result from the acceleration measurement in X-direction (the train moving direction) for MRT system is shown in Figure 2. For this case, it can be found that the train is consisted of 4 cars. The eight wheels' peaks are marked distinctly in the figure. The speed of the train can be determined from the passing time with the given length of the train.

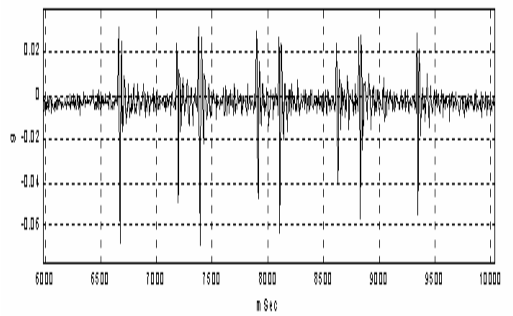

Figure 2. Time domain data of acceleration measured in the X-direction (Ni et al. 2002)

Presentation of measuring results

There are several ways, such as narrow band and octave band, etc., to present the vibration characteristics from time domain measurement. The IOS 2631 recommends that, where many closely-spaced frequencies or broadband (random) energy are involved, one-third octave band filters should be used to differentiate the effects of one frequency range from another. In this paper, the one-third octave band velocity spectra were used to present the time domain data. All the measurements are carried out using the one-third octave band analyzer with central frequency from 1 Hz to 100 Hz. The ground vibration level is expressed in terms of its root-mean-square (RMS) velocity using the decibel scale. The RMS velocity level in decibels is defined as:

$$VL \text{ (in dB)} = 20 \log_{10} (v_m/v_{ref})$$

in which v_m = the measured velocity and v_{ref} = the referred velocity (where $v_{ref} = 10^{-6}$ in/sec in this study).

DATA BASE

There are five sites selected to measure the ground vibrations in accordance with the variation of railway systems, soil deposits, and bridge types. The mass rapid transit (MRT) system (Ni et al. 2002) and ordinary railway (OR) system are two major vibration sources in this study. The measurement for MRT system has only a unique train type, car number and train speed, etc., while the measurement for OR system has different car numbers and speeds. The details of different train types, including car, wheel and track information are listed in Table 2. In Table 2, the major differences between OR1 and OR2 are car numbers. Furthermore OR1 was divided into two types based on their train speed.

For the selection of structure type, material of superstructure, foundation type, and geological condition are the main reference bases. For super-structure, prestressed concrete (PC) and steel, in general, are the major materials. The spread footing, pile and caisson are commonly used as bridge foundations. Table 3 presents the detailed information for these structure types of measurement. A total of six structure types were included in this vibration measurement. It can be seen that these structure types represent a wide variety.

Geological condition is also a very important concern on the site selection. The foundation bearing ground in MRT system includes hard ground (rock formation) and soft grounds (soil layers). For the two measurement locations of ordinary railway system, OR1 is located at central Taiwan with hard ground (gravel formation), while OR2 is at south Taiwan with soft ground (soil layer). In addition, the OR1 was measured at different speeds and distances for same train, and OR2 was measured with the different distances. In summary, the database can be used to check the influence factors of train type, geological condition, bridge type, train speed, and vibration dissipation with distance.

Table 2. Information for different train types

Type[1]	Car number	Car length (m)	Speed (km/hr)	Wheels				Supporting Track
				type	max. axle load (t)	dist. of bogie centre (m)	Track	
MRT	4	13.78	70	rubber	13.76 x 2	10	concrete	concrete
OR1-1	9	20	114	steel	14 x 4	14	steel	ballast
OR1-2	9	20	81	steel	14 x 4	14	steel	ballast
OR2	14	20	113	steel	14 x 4	14	steel	ballast

Note: 1. MRT is mass rapid transit system; OR is ordinary railway system.

Table 3. Information for different structure types

No.	Train/ Structure Type	Superstructure				Pier			Foundation	
		type	mater-ial[1]	shape	span (m)	mater-ial[1]	height (m)	cross sect. dim. (m)	type	dimen-sions (m)
S1	MRT/bridge	cont.	PC	box	30-30	RC	6.4	1.8ϕ	footing	8 x 8
S2	MRT/bridge	cont.	steel	box	40-70-37	RC	10.0	1.6-2.0ϕ	pile	4-0.8-1.3ϕ L=33-37
S3	MRT/bridge	cont.	PC	box	23-23	RC	10.0	1.6ϕ	pile	4-0.8-1.3ϕ L=33-37
S4	MRT/bridge	cont.	PC	box	35-25-25 15-30	RC	8.0	1.6-1.8ϕ	pile	4-0.8ϕ L=63
S5	OR/bridge	simple	PC	I	25	RC	18.0	2.5-5.0ϕ	footing	10 x 10
S6	OR/bridge	simple	steel	open deck	19.2	RC	7.0	2-1.1x1.1	caisson	6.8ϕ L=6.8

Note: 1. PC is prestressed concrete; RC is reinforced concrete.

MEASUREMENT RESULTS

As described above, the MRT system has only one train type. Therefore, each location can be easily measured ten passings of the transit train. The measurement result was taken by the average of ten measurements. However, each location has been measured only once for ordinary railway system. Through appropriate Fast Fourier Transform (FFT), the measurement data are expressed in terms of the one-third octave band analyzer with central frequency from 1 Hz to 100 Hz. The measurement result for each location is described briefly as below.

Site 1

The site is located at Lin-Kung bus maintenance station of Taipei, which is near the hillside. The elevated structure is a two-span continuous pre-stressed concrete (PC) bridge with the span length of 30 m. The bridge is connected to open-cut in one side and tunnel in another side. The super-structure of the bridge is PC box girder with a depth of

1.4 m. For the sub-structures, the diameter of the pier is 1.8 m and the average heights of the piers are 6.4 m. The shallow foundation (8mx8m) was adopted since the subsurface of this site is interbedded with slightly weathered sandstone and shale, which is considered to be a good bearing layer.

The results measured at site 1 for the X, Y, and Z directions are shown in Figure 3. Ten curves are presented in Figure 3, and five for train toward east direction (dash line) while another five for train toward west direction (solid line). It can be seen that there are peak values at 4, 6.3, and 12.5 Hz in the X direction while there are peak values at 2.5, 3.15, 5, 6.3 and 12.5 Hz in the Y direction. However, there are peak values at 5, 6.3, 12.5, 16 and 25 Hz in the Z direction. The largest velocity level is about 60 dB in X and Y direction and about 70 dB in Z direction.

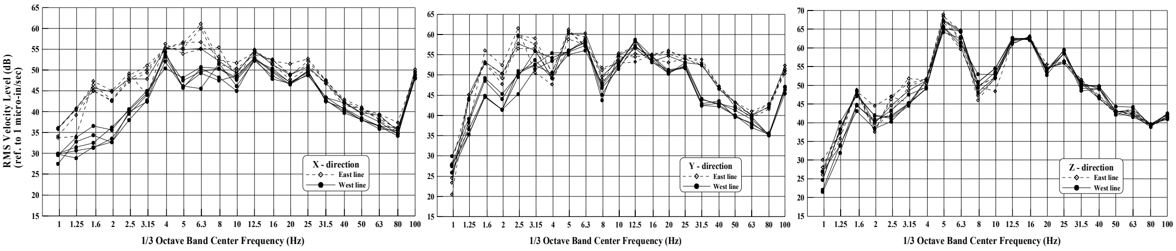

Figure 3. One-third octave bandwidth frequency response of site 1

Site 2

The site 2 is located at intersection of Tunhua south road and Hoping east road of Taipei. The prestressed concrete (PC) l bridge is 40m-70m-37m. For the three-span continuous P.C. bridge, the span lengths are all 23 m. The superstructure of the P.C. bridge is box girders with depth of 1.4 m. The superstructure of the steel bridge is also box girders but with depths varied form 2.8 m to 2.0 m. For the sub-structures, the diameter of the piers for the longest span (70 m) is 2.0 m, and the diameter of other piers is 1.6 m. The average height of the piers is 10 m. The shaft foundation is adopted since the soil conditions are soft in this area. For the longer span of steel bridge, six piles are connected to a reinforced concrete footing and four piles are used for other foundations. The diameters of piles were designed as 0.8 m~1.2 m and the pile lengths are 33 m~37 m. The subsurface of soil deposits of this site is more complicated than that of the site 1. The soil profile mainly consists of soft clay (CL) with layers of fine sand (SP) or gravel (GP). The SPT-N value of clay layer is less than 10 while the SPT-N value of gravel layer is greater than 50. For the purpose of comparison, two ground vibration locations are simultaneously measured in this site as the transit car passing. One is for steel bridge deck while the other is for concrete bridge.

The one-third octave bandwidth frequency responses of site 2 for the steel bridge deck and the concrete bridge deck are shown in Figures 4 and 5, respectively. As shown in the figures, the amplitude in the vertical (Z) direction is greater than the other two directions for the case of steel bridge. However, for the concrete bridge the amplitude in the vertical direction is less than the other two directions.

The result measured at the steel bridge shows that there are peak values at 3.15, 5, 6.3 and 40 Hz in the X direction while there are peak values at 1.25, 1.6, 6.3, 12.5, and 25 Hz in the Y direction. However, there are peak values at 1.6, 2, 5, 6.3 and 10 Hz in the Z direction. The largest velocity level is about 58 dB in X direction, 60 dB in Y direction and 73 dB in Z direction. The result measured at the concrete bridge shows that there are peak values at 1.6, 6.3, 25, and 50 Hz in the X direction while there are peak values at 1.6, 8, 12.5, 16, 20 and 50 Hz in the Y direction. However, there are peak values at 1.6, 6.3, and 25 Hz in the Z direction. The largest velocity level is about 75 dB in X, 78 dB in Y direction and 73 dB in Z direction.

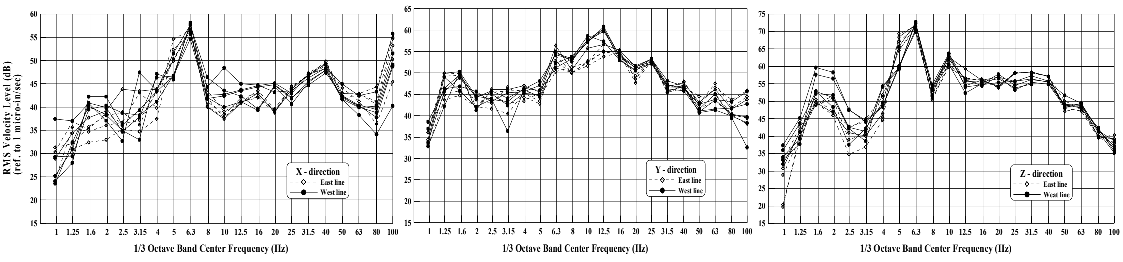

Figure 4. One-third octave bandwidth frequency response of Site 2

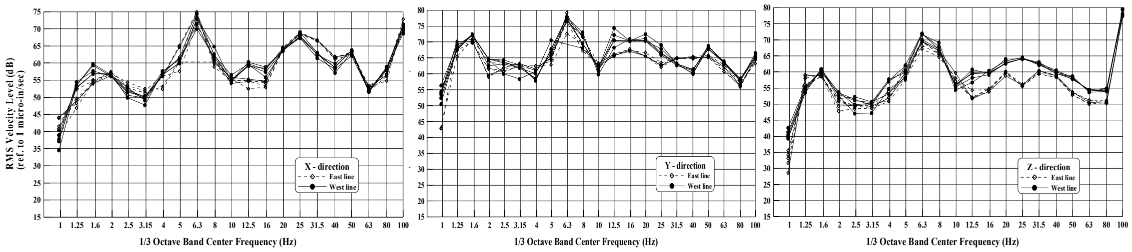

Figure 5. One-third octave bandwidth frequency response of concrete bridge at Site 2

Site 3

The site 3 is located at the intersection of Fuhsing north road and Hsin-An street of Taipei. The selected case in this location is a three-span continuous pre-stressed concrete (P.C.) bridge with the span length of 35m-25m-25m and a two-span continuous pre-stressed concrete (P.C.) bridge with the span length of 15 m-30 m. The superstructure of the bridge is P.C. box girder with a depth of 1.4 m. For sub-structures, the diameters of the pier are 1.8m and 1.6m and the average height of the piers is 8 m. The shaft foundations are adopted since the soil conditions are quite soft in this area. The soil profile of this site consists mainly of silty sand (SM) with layers of clay (CL). The SPT-N value for most layers is less than 10. The foundations were designed as 4-0.8 m~1.2 m shafts with pile length of 63 m.

The one-third octave bandwidth frequency response of site 3 is shown in Figure 6. The amplitude in the Z directions is slightly less than the other two directions. The result measured also shows that there are peak values at 6.3, 12.5, 20, 50 and 63 Hz in the X direction while there are peak values at 1.6, 6.3, 12.5, and 20 Hz in the Y direction. However, there are peak values at 1.6, 6.3, 20, 31.5 and 40 Hz in the Z direction. The largest velocity level is about 75 dB in both X and Y directions and 70 dB in Z direction.

The result also shows that the less vibration levels occur when the foundation is placed on the hard ground (bedrock) as comparing with placing on the softer soil deposits.

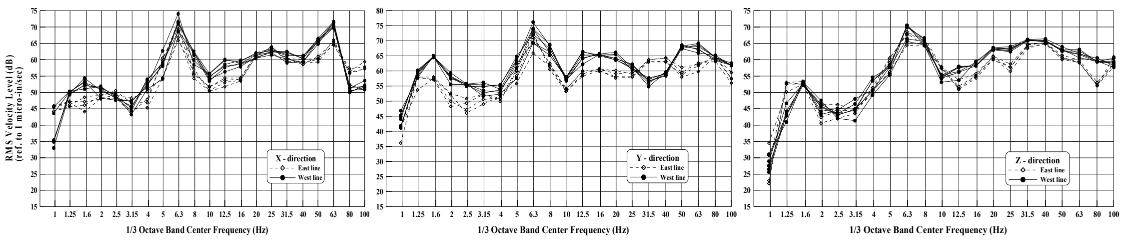

Figure 6. One-third octave bandwidth frequency response of Site 3

Site 4

The site 4 is located at central Taiwan and near the Tai-An Station of Taiwan Railway System. The selected case in this location is a simple pre-stressed concrete (P.C.) bridge with the span length of 25 m and between Tai-An Station and tunnel. The superstructure of the bridge is P.C. with I shape. For sub-structures, the diameters of the pier are changed from 2.5 m to 5.0 m and the average heights of the piers are 18 m. The spread footing foundation (10m x 10m) is adopted since the soil condition is a quite homogeneous gravel formation. The groundwater table is around 7-8 m below ground surface.

The one-third octave bandwidth frequency response of site 4 at 0 m distance (near the pier) is shown in Figure 7 for train with speeds 114 km/hr and 81 km/hr, respectively. All measurements for different distances show that the amplitude in higher speed is larger than in lower speed. The dominant amplitude and frequency in Z direction are 83 dB/63 Hz, 60 dB/25 Hz, 50 dB/10-12.5 Hz, and 45dB/8Hz, for distances with 0 m, 25 m, 100 m, and 200 m to the track centre, respectively. For the comparison of three directions, the amplitude in the Z-direction is larger than in the X and Y directions, however, the attenuation in the Z direction is faster than in the X and Y direction. For example, the amplitude in the Z direction is larger 7-8 dB than the X and Y directions at 0 m and the amplitudes for three directions are almost same at 100 m and 200 m distances.

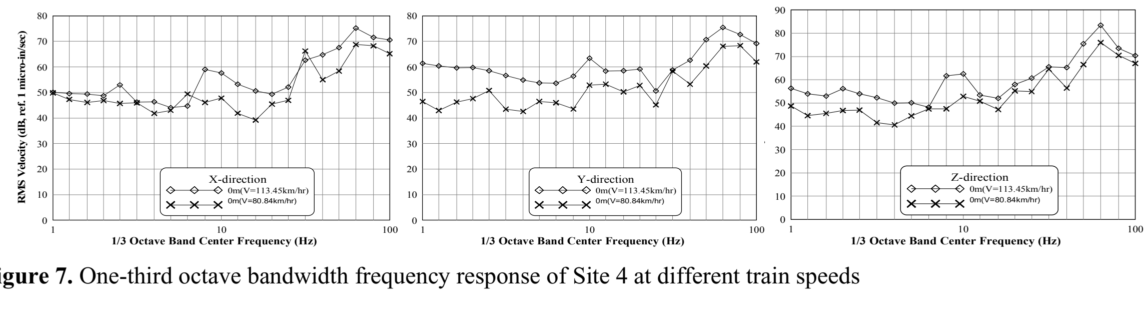

Figure 7. One-third octave bandwidth frequency response of Site 4 at different train speeds

Site 5

Site 5 is located at Tseng Wen River of southern Taiwan (Shen, 2002). The selected case in this location is a steel bridge with the span length of 19.2 m. The bridge is an open-deck and simple-supported type. For sub-structures, the piers are two 1.1 m x 1.1 m square type and the average heights of the piers are 7 m. The caisson foundations are adopted since a soft clay layer is underlain by a deep bearing stratum (sandstone and mudstone). The soil profile 15-20 m below ground surface is soft silty clay (CL) with the SPT-N value between 8 and 12. The caisson foundation was designed as 6.8 m diameter with length of 6.8 m. The train speed for the measurement is 112 km/hr.

The one-third octave bandwidth frequency response of site 5 is shown in Figure 8 for different distances. Because the site is located at soft soil and the stiffness of the bridge is low, the amplitude near the pier (distance = 0 m) exceeded the maximum acceleration of the sensors (> 7.07 m/sec^2). When the vibration amplitude propagates to the distances of 25 m and 200 m, their amplitude and frequency in the Z direction are 69 dB/8-10 Hz and 63 dB/5-10 Hz, respectively.

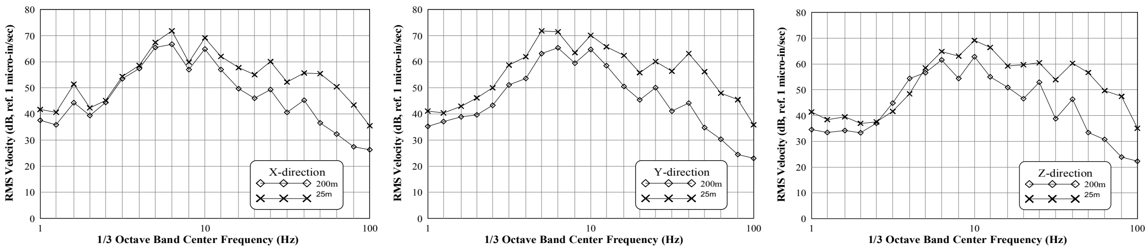

Figure 8. One-third octave bandwidth frequency response of Site 5 at different distances

All above measured results and their related information for Site 1 through Site 5 are summarized in Table 4.

Table 4. Measurement results

Site No.	Train type	Structure type	Train speed (km/hr)	Soil description	Dist. From source (m)	Max. dB (x) / freq. (Hz)	Max. dB (y) / freq. (Hz)	Max. dB (z) / freq. (Hz)
1	MRT	S1	70	SS / SH	0	60/6.3	60/2.5,3.15, 5,6.3	70/5,6.3
2	MRT	S2-S3	70	clay	0	75/6.3, 58/6.3	78/6.3, 60/12.5	73/6.3, 73/6.3
3	MRT	S4	70	silty sand	0	75/6.3	75/6.3	70/6.3
4	OR1-1	S5	114	gravel	0	75/63	75/63	83/63
4	OR1-1	S5	114	gravel	25	-	-	60/25
4	OR1-1	S5	114	gravel	100	-	-	50/10-12.5
4	OR1-1	S5	114	gravel	200	45/10	50/8	45/8
4	OR1-2	S5	81	gravel	0	69/63-80	68/63-80	76/63-80
4	OR1-2	S5	81	gravel	200	40/10	42/10	40/10,20,31.5
5	OR2	S6	112	clay/SS-MS	0	>7.07m/sec^2	>7.07m/sec^2	>7.07m/sec^2
5	OR2	S6	112	clay/SS-MS	25	72/6.3-10	72/5-10	69/8-10
5	OR2	S6	112	clay/SS-MS	200	67/5-10	65/5-10	63/5-10

EVALUATION OF MEASUREMENT RESULTS

Table 4 presents the summary results of maximum RMS velocity and their corresponding frequencies of the X, Y, and Z directions for all sites. The train type, structure type, geological condition and measured distance are also listed for the following comparison.

Factor of the characteristics of cars

The characteristics of cars related to ground vibration are car weight, car number, car length, wheelbase, bogie-centres distances and train speed, etc. The train weights and train speeds are considered as the major influence factors for the magnitude of the amplitude of ground vibration based on these available data. These two factors are the major sources of impact power for ground vibration.

Factor of speed

Site 4 was measured at different speeds. It can be seen that the maximum vibration level for high speed (114 km/hr) is larger than for low speed (81 km/hr). For the speed with 30% difference, the maximum vibration level of higher speed has about 5-8 dB more than the case of lower speed for different distances. It is consistent with general supposition. However, for each different frequency, the increase of vibration level due to the increasing of train speed does not have the same response. Therefore, it is proposed that the increase of vibration level is related to both speed and frequency.

Factor of geological condition

Both MRT and OR systems have been measured in different geological conditions. It is obvious that the geological condition is a major factor for the magnitude of ground vibration level. The results of measurement show that the less vibration levels for the case that the foundation is placed on the hard ground (Sites 1 and 4) as comparing with placing on the softer soil deposits (Sites 2, 3 and 5).

The frequency of maximum vibration level in hard ground and soft ground are also quite different. It can be found from Table 3 that, in general, the maximum level is happened at high frequency (6.3-12.5 Hz for MRT system and 63 Hz for OR system) for hard ground, and soft ground is happen in low frequency (6.3 Hz for MRT system and 5-10 Hz for OR system).

The attenuation of vibration level is also greatly affected by the geological condition. From Table 3, the maximum vibration level becomes around 36-42 dB for OR1 while the OR2 still remains 59-64 dB. It is about 22-23 dB difference. Therefore the attenuation of vibration in hard ground will be more obvious than in soft ground.

Factor of bridge type

For the purpose of comparison, the vibration level curves for the two different structures are shown in Figure 9. In the figure, the result of solid line is measured at the concrete structure while the result of the dash line is measured at the steel structure. It can be seen that the steel structure is effectively reduced the energy from X and Y directions as comparing with that from Z direction.

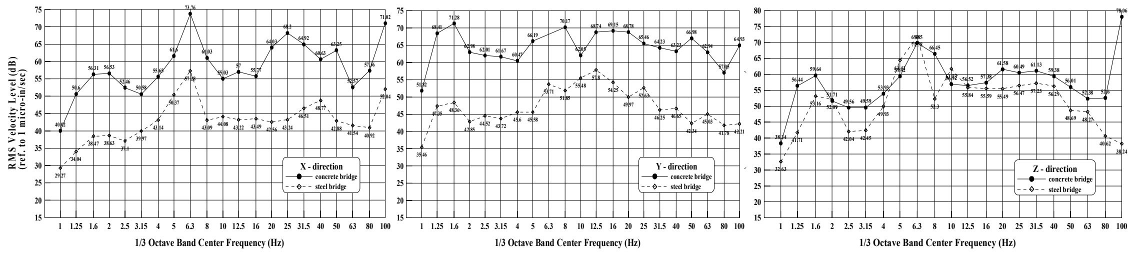

Figure 9. Comparison of frequency response between steel and concrete structure

The train-supporting structural condition is also a major factor for the magnitude of ground vibration level. The results of MRT system measurement show that the less vibration levels induced by trains when using the steel structure comparing with using the concrete structure (Site 2). It is very possibly induced by the factors of rigidity and concrete volume of the structures. A comparison of bending rigidity and concrete volume of steel structure and concrete structure is presented in Table 5. The steel bridge has much more bending rigidity than the case of concrete bridge. The results indicated that the more bending rigidity and concrete volume of the structure, the less of ground vibration level.

Table 5. Comparison of bending rigidity and concrete volume of different structures

Element / **Bridge type**	**Bending Rigidity (Beam)**			**Concrete Volume**		
	E (kN/m^2)	**I_x (m^4)**	**EI_x ($kN\text{-}m^2$)**	**pier (m^3)**	**footing (m^3)**	**pile (m^3)**
Steel	$2.04x10^8$	0.16-0.2	$3.67x10^7$	31.4	108	223
P.C.	$3.02x10^7$	0.48	$1.44x10^7$	20.1	37.5	69
Steel/PC Ratio	-	-	2.6	1.6	2.9	3.3

Evaluation of attenuation

All data show that the ground vibration level attenuates with increasing distance. However, the attenuation is different in various ground condition. The attenuation is more obvious in hard ground than in soft ground. In the other words, the attenuation in high frequency is more obvious than in low frequency. Comparing the vibration levels for the X, Y, and Z directions from these data, the attenuation in the Z-direction was more obvious with the increased distance.

Assessment of evaluation methods

Based on the measurement results, the evaluation methods are discussed as follows.

Influence distances

The influence distances of the train system for the area with soft ground might be greater than 200m from the track centre. In our measurement case of Site 5, the low frequency vibrations are still very obvious even as far as 200m from the track centre. If the train system is planed to pass by the area with soft ground, the screening distances might need to be increased as suggested in the current evaluation method.

Train speed

The relationship between the vibration level and train speed is not as simple as indicated in existing evaluation methods. It is complicated and there is a need for consideration of the vibration frequency. The train speed has some effects on the magnitudes and dominant frequency contents of ground-borne vibration. The relationship of train speed, distance of bogie-centre, vibration level and dominant frequency needs more measurements and studies to verify the existing evaluation methods.

Attenuation of vibration level

The attenuation of vibration levels for each frequency is quite different, and the current evaluation methods lack proper assessment methods of this factor. The evaluations of overall vibration level are not suitable for some high vibration sensitive area. The far field wave propagation behaves quite different to the near field wave. The current evaluation methods are lack proper assessment methods to estimate the far field vibration.

CONCLUSIONS AND RECOMMENDATIONS

The ground vibration was evaluated for the urban train system for a wide variety of vibration sources and transmitting media. The database included two train systems with different train speeds, geological conditions, structure types and measuring distances. All measurement results were expressed as the root-mean-square (RMS) velocity using the decibel scale. For analyses of each influence factor, detailed evaluations were done, preliminary conclusions were reached, and some summaries were developed.

1. The train speed and train weight are considered as the major influence factors for all train related factors to the ground vibration.
2. For geological factor, the maximum vibration level was at high frequency for hard ground and low frequency for soft ground. In addition, the attenuation of vibration in hard ground will be more obvious than in soft ground.
3. Comparing the steel bridge and concrete bridge, it can be found that the more bending rigidity and concrete volume of structure, the less of vibration level.
4. The maximum vibration level increases with the speed increase. However, the increase of vibration level for different frequencies is not proportional with speed.
5. The ground vibration attenuates with the increase of distance. Furthermore, the attenuation in Z-direction has the greatest effect with distance.

However, the vibration level and dominant frequency due to the variation of train speed and distance of bogie-centre needs more measurements and studies to improve the existing evaluation methods. It is also recommended that a more systemic classification for the geological database related to the vibration attenuation situation is essential for further vibration evaluation.

In summary of these evaluation results, a high speed train in the soft ground and low structure rigidity easily produces high ground vibration, and the ground vibration propagates longer distance in such situation.

Acknowledgements. This study was supported by the National Science Council, Republic of China (Taiwan), under contract number: NSC94-2211-E-033-012.

Corresponding author: Prof. Yit-Jin Chen, Chung Yuan Christian University, 200, Chung Pei Rd, Chung Li, Tao-Yuan County, 32023, Taiwan. Tel: +886 3 2654227. Email: yjc@cycu.edu.tw.

REFERENCES

U.S. DEPARTMENT OF TRANSPORTATION FEDERAL RAILROAD ADMINISTRATION (FRA). 1998. Manual for High-Speed Ground Transportation Noise and Vibration Impact Assessment

U.S. DEPARTMENT OF TRANSPORTATION FEDERAL TRANSIT ADMINISTRATION (FTA). 1995. Manual for Transit Noise and Vibration Impact Assessment

YOSHIOKA, O. 2000. Basic Characteristics of Shinkansen-Induced Ground Vibration and Its Reduction Measures. *Proceedings, the International Workshop Wave 2000*, 219-237

EJIMA, A.1980. Basic Research for Countermeasures Against Ground Vibration from Concrete Girder Type Elevated Bridges of Shinkansen. *Doctoral Thesis, University of Tokyo*

NI, S. H. 1999. The Experimental Study of Effectiveness of Wave Screening Using Trench (I): The Vibration Measurement and Investigation of Ground Noise and Related Vibration Parameters. *Report for National Science Council, National Cheng-Kung University*, 38-72 (in Chinese)

NI, S. H. YANG, C.W. SHEN,Y.J. YEN, P.J. 2002. The Characteristics of Vibration Induced by MRT-Mucha Line Rapid Transit System. *2002 WMSE Symposium, Taipei*

SHEN, Y.J. 2002. The Application of Deep Foundations to Mitigate Ground Vibration for a Train-Supporting Viaduct Structure in Soft Ground. *ASCE 2002 Deep Foundation International Conference*