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Vibration serviceability of timber pedestrian bridge based on field test and analysis

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1 Introduction

Investigation of vibration serviceability for pedestrian bridges is an important issue in de-sign and maintenance. The vibration serviceability of pedestrian bridges has been studied for many pedestrian bridges made by concrete and steel, for example (Kajikawa, 1979), (Kobri, 1990), (Tanaka, 1994), (Obata, 1997) and (Yoneda, 2003). However, the study on vibration serviceability for timber pedestrian bridges is almost never carried out, except by (Yamada, 2003) and (Kusaka, 2006).

Figure 1. Tokiwa Bridge; subject of this study

The purpose of this study is to evaluate vibration serviceability of timber pedestrian bridges based on experiment and analysis. The subject bridge is Tokiwa Bridge, a large scale three spans continuous girder made by bongossi wood material and constructed at 1995 in Kitakyushu City in Japan as shown in Fig.1. The design geometry is: bridge length: 84.4 m, maximum span length: 31.0 m and clear width: 6.0 m.

The dynamic field test of Tokiwa Bridge was carried out in 2007. The vibration characteristics and dynamic behavior were investigated, and then vibration serviceability of the bridge was verified by the measured response velocity. Next, a three dimensional dynamic response analysis due to walking of pedestrian on the bridge was also carried out, and then the experimental and analytical results were compared. Furthermore, the investigation of the dynamic response characteristics by the dynamic response analysis and evaluation of vibration serviceability were carried out, and vibration serviceability was finally verified from both results of the experiment and the analysis. As a result, this study verified that it is possible to evaluate vibration serviceability of the timber pedestrian bridge analytically.

2 Subject Bridge and Vibration Limit for Vibration Serviceability

2.1 Subject bridge

The general drawing of Tokiwa Bridge; the subject of this study is shown in Fig. 2 to Fig.4. The bridge length is 84.4 m, and the maximum span length is 31.0 m. Each main girder with the cross section of 1200 x 265 mm is 3 spans continuous girder laminated by bongossi material of 5 layers. For this lamination method, drift pins driven in 25 cm interval are used. Fig. 5 and Fig. 6 show the outside and inside of joint part. The reinforce steel element shown in Figure 6 was newly installed in reinforcing work at 2005. Table 1 shows the design condition of the bridge.

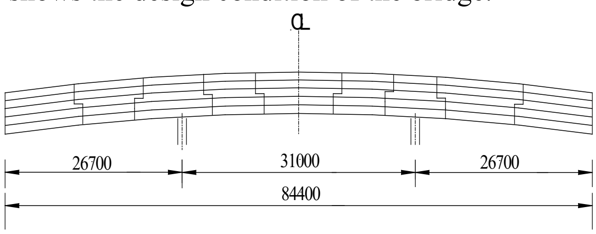

Figure 2. General drawing of Tokiwa Bridge

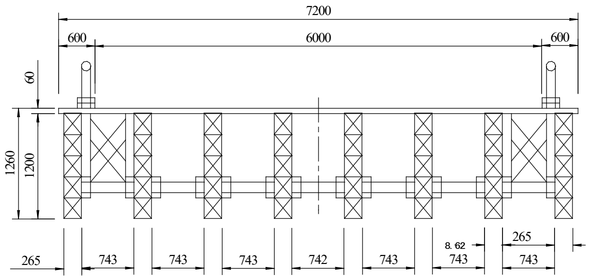

Figure 3. General drawing of Tokiwa Bridge

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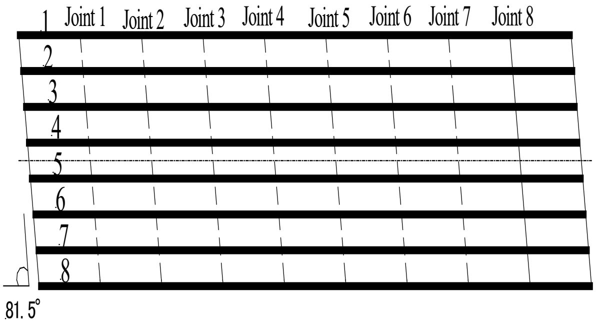

Figure 4. General drawing of Tokiwa Bridge

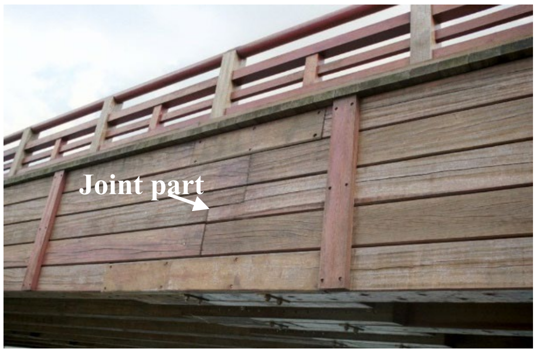

Figure 5. Outside of joint part

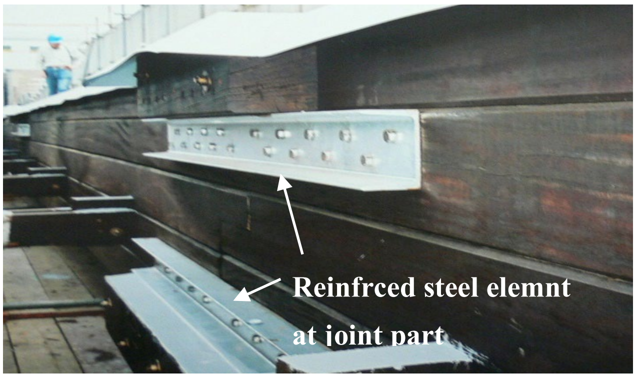

Figure 6. Inside of joint part

Table 1. Design condition of the subject bridge

Constructed Place	Fukuoka, Japan (1995)
Bridge class	Timber pedestrian bridge
Bridge type	3 span continuous girder laminated in 5 layers
Bridge length	85.0m
Span, length, width	26.75+31.0+26.75m
Main timber kind	Bongossi

2.2 Vibration limit for vibration serviceability

Some international specifications and codes such as British Standard (BS5400), Eurocode 5 and Ontario Code in Canada specify vibration limit for vibration serviceability of pedestrian bridges. These vibration limits are based on the evaluation of response acceleration.

In Japan, the design guideline of pedestrian bridges was instituted in 1967. The design code on vibration serviceability has been revised in 1979, with some revision carried out afterwards. The present vibration limit of pedestrian bridges which is more strict than the design specification of highway bridges has been adopted as the code on vibration serviceability. In present code, the maximum deflection of main girder by live load is forbidden to exceed 1/600 of span length. However, when the vibration effect of pedestrian is considered in design, deflection limit is relduced to 1/400 . To consider the vibration effect of pedestrian means to exclude natural frequency of 1.5-2.3 Hz in the design of pedestrian bridges. The code on deflection and vibration has been adopted in the design method that the discomfort of pedestrian may not occur in waking on pedestrian bridges.

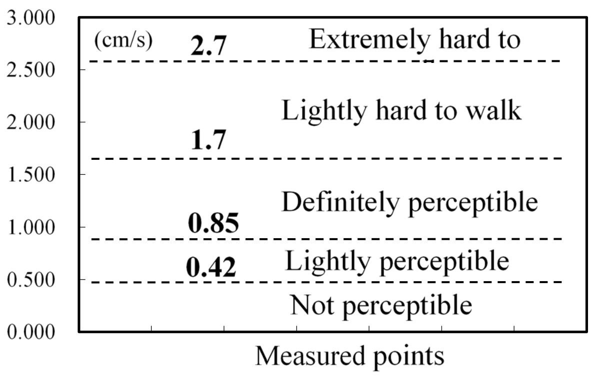

Figure 7. Vibration limit by maximum value of response velosity

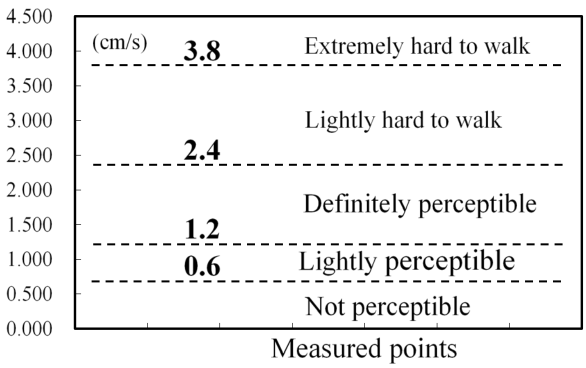

Figure 8. Vibration limit by root mean square (rms) value of response velosity

In Japan, the vibration limit on discomfort of pedestrian is based on the evaluation of response velocity. The studies by Kobori and Kajikawa (1974), and Kajikawa (1982) greatly contribute for the vibration limit. There are 2 methods to evaluate vibration serviceability by response velocity. Fig. 7 shows the vibration limit by maximum value of response velocity for vibration serviceability. The response velocity of 2.4 cm/s with the sense that pedestrian reacts with a lightly hard to walk feeling is evaluated as a vibration limit value for vibration serviceability of pedestrian bridges. Fig. 8 shows the vibration limit by root mean square (rms) value of response velocity for vibration serviceability. The response velocity of 1.7 cm/s with the sense that pedestrian reacts with a lightly hard to walk feeling evaluated as a vibration limit value for vibration serviceability of pedestrian bridges.

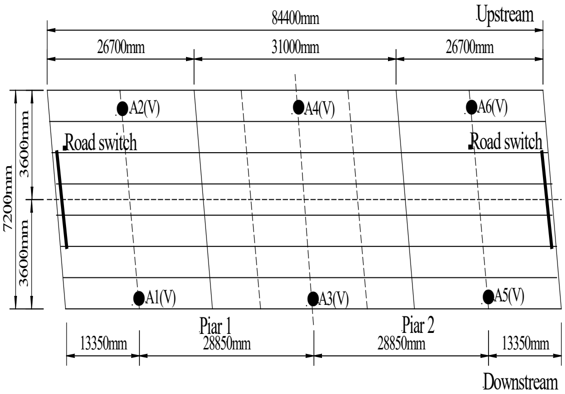

Figure 9. Measurement points of resnant waking and running tests

3 Field Dynamic Test

3.1 Testing method

The field dynamic test was conducted in order to investigate dynamic characteristics of Tokiwa Bridge in 2007. The dynamic test was done by tests such as (1) ambient vibration test, (2) impact loading test by dropping of sand bag with weight 0.3 kN and (3) resonant waking and running tests of pedestrians. The measured data is checked and stored in digital recorder and personal computer, and then the dynamic behaviour and characteristics of the bridge are analyzed in monitoring system of field test. In resonant waking and running tests, the 1-2 pedestrians walked and ran in a pace equal to vertical flexural first natural frequency of 2.25 Hz of the bridge obtained by the ambient vibration and impact loading tests, and then the response velocity and acceleration were measured. This is a critical experimental method in which the bridge becomes in resonant state by the external force of pedestrian with 2.25 Hz. Fig. 9 shows the measurement points of resonant waking and running tests. The symbol of V in this figure indicates the vertical direction of the bridge. The signal of In and Out on transfer of pedestrian is measured by the load switch shown in this figure. The transfer cases of pedestrian are walking and running at edge part of upstream.

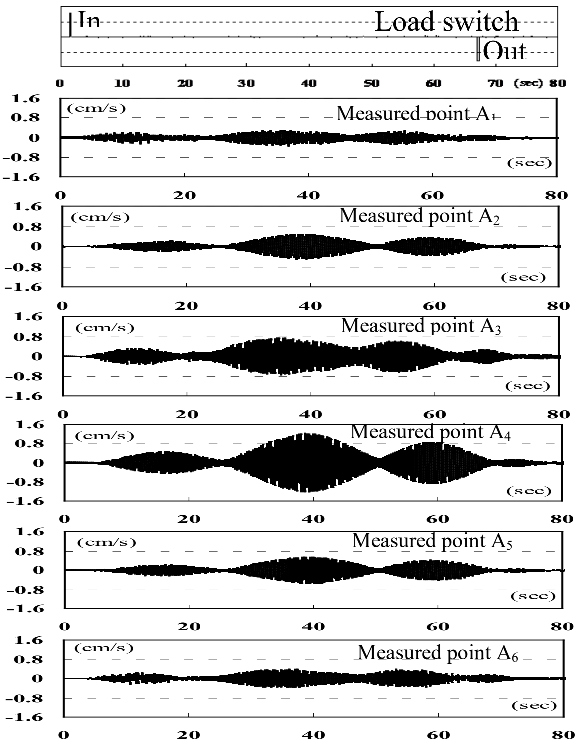

Figure 10. An example of response velocity in one person walking test with resonant pace of 2.25Hz at edge p art of upstream

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3.2 Result of dynamic test

Fig. 10 shows an example of response velocity in one person walking test with resonant pace of 2.25Hz at edge part of upstream. The structural characteristics of three spans continuous girder has appeared clearly from this vibration response wave. In the walking test of one pedestrian, the maximum value of response velocity was 1.23 cm/s at measured point of A4. That value was 1.63 cm/s at A4 in the walking test of two pedestrians. In the running test of one pedestrian, that value was 4.25 cm/s at A4. In the walking test of one pedestrian, the rms value of response velocity was 0.44 cm/s at A4. That value was 0.51 cm/s at A4 in the walking test of two pedestrians. In the running test of one pedestrian, that value was 1.69 cm/s at A4.

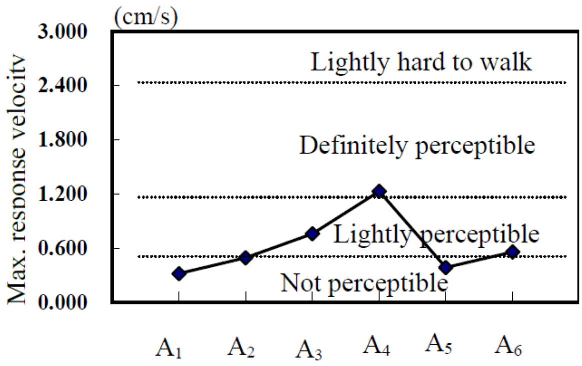

Figure 11. Evaluation of vibration serviceability

Fig. 11 shows an example for evaluation of vibration serviceability by maximum response velocity in the walking test of one pedestrian. Pedestrian may have vibration sense of lightly perceptible, when one person walks in the resonant state. Fig. 12 shows an example for evaluation of vibration serviceability by maximum response velocity in the running test of one pedestrian. Pedestrian may have vibration sense of extremely hard to walk, when one person is running in the resonant state.

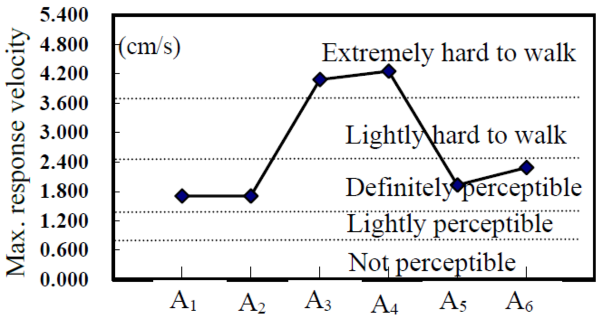

Figure 12. Evaluation of vibration serviceability

Spectral analysis using FFT was conducted with data measured by the dynamic test. The experimental and analytical dynamic characteristics are shown in Table 2 until vibration of the 4th degree. The vibration modes and natural frequencies of the bridge were also computed by three dimensional eigenvalue analysis using MSC/NASTRAN. There is good agreement between experimental and analytical results.

Table 2. Dynamic characteristics of the objected bridge

No.	Mode types	Natural frequencies f_i (Hz)		Damping coefficients h_i
		Measured	Analyzed	
1	Horizontal 1st	1.37	1.40	0.0050
2	Vertical 1st	2.25	2.27	0.0145
3	Vertical 2 nd	3.10	2.85	0.0128
4	Torsional 1st	3.71	3.69	0.0154

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4 Dynamic Response Analysis

4.1 Analysis method

In the three dimensional dynamic response analysis of the bridge due to walking and running of pedestrian, Fig. 13 shows the coordinate system of x, y and z axis on node displacement and node force of element member. The ui and uj in this figure are axis displacement in x direction at node i and j. The vi and vj are axis displacement in y direction. The wi and wj are axis displacement in z direction. The θxi and θxj are rotational angle in x axis. The θyi and θyj are rotational angle in y axis. The θzi and θzj are rotational angle in z axis. The xi and xj are axis force in x direction at node i and j. The yi and yj are axis force in y direction. The zi and zj are axis force in z direction. The Mxi and Mxj are bending moment with respect to x axis at node i and j. The Myi and Myj are bending moment with respect to y axis. The Mzi and Mzj are bending moment with respect to z axis.

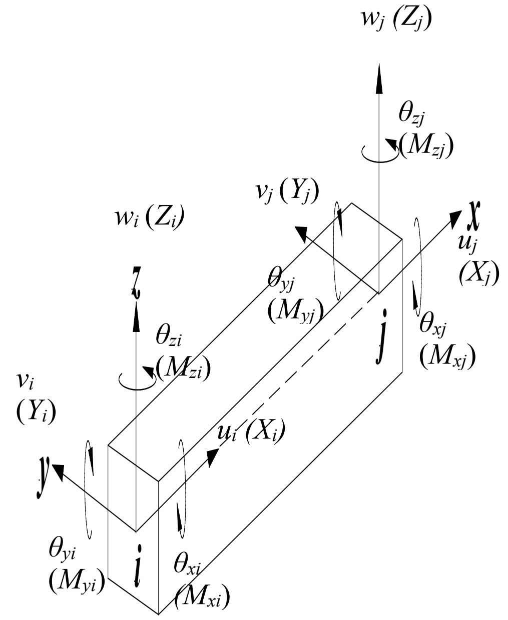

Figure 13. Coordinate system of element member

In the dynamic response analysis method, there are modal analysis method and direct integral method. In this study, the analysis shown in the following was carried out using direct integral method. The equation of motion on vibration system of pedestrian and bridge is given by:

$$[M]\{Z\&\&\}+[C]\{Z\&\}+[K]\{Z\}=\{F\} \tag{1}$$

where: [M] - mass matrix; [C] - damping matrix; [K] - stiffness matrix;
{F} - external force vector by pedestrian.

As numerical integration method, Newmark's β method was used to solve the equation (1) in this study. The integral time interval Δt is 0.05 seconds. The arbitrary constant γ is 1/2. The parameter β is 1/4. The convergence accuracy of acceleration is 1/1000 in Δt.

The damping matrix in this study is assumed by Rayleigh damping. The parameters of α and η on damping are obtained from natural frequency fi and damping coefficient hi in vertical vibrations based on experimental values as shown in Table 2. The Rayleigh damping is given by

$$[C]=\alpha[K]+\eta[M] \tag{2}$$

where: α=(h1 f1-h2 f2)/(π(f12-f22)); η=4π f2 (h2-πf2 α).

The external force by pedestrian is given by

$$\{F\}=F(t)=\{mg+mg\xi\sin(2\pi p_s t)\}\vartheta(t) \tag{3}$$

where: mg - body weight of pedestrian; ps – pace of pedestrian (Hz); ξ- impact force ratio; θ(t) - coefficient vector done proportional distribution at both nodes of element in which pedestrian loads.

The field experiment at the subject bridge and laboratory experiment which changed body weight, pace and step size of pedestrian were conducted in order to calculate the impact force ratio ξ with four load cells of plate type. Fig. 14 shows the result obtained by those experiments. There are many cases in which pedestrian generally does walking and running in 2.0 - 2.5 Hz pace. The external force of pedestrian in this study improved the method by using sine wave which Kajikawa (2000) proposed. As it is shown in Fig. 15, the method where half sine wave removed negative load part (load to top from under floor system) of input sine wave was used.

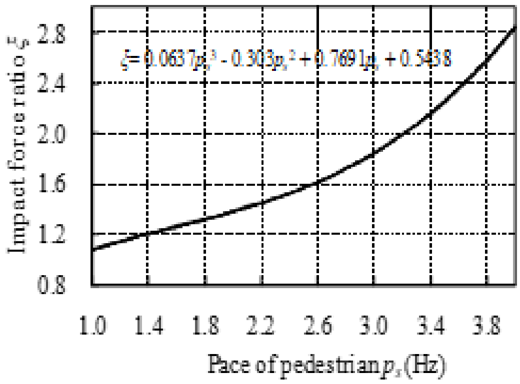

Figure 14. impact force ration of pedestrian

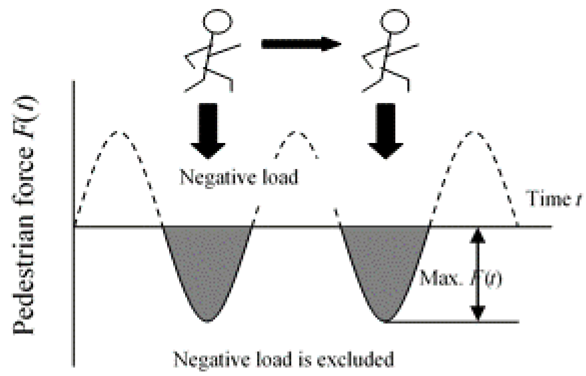

Figure 15. Half sine wave of external force

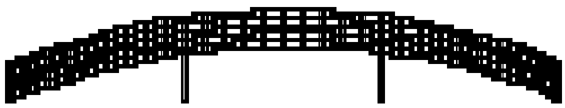

Figure 16. Structural girder model of 5 layers

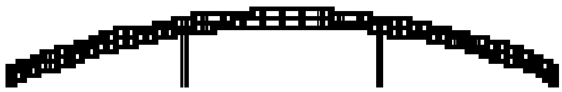

Figure 17. Structural girder model of 2 layers

The partial structural models in case with joint and in case without joint were made, and the effect of joint part was investigated from static and eigenvalue analyses. As a result, there was no difference by both structural models from both sides of natural frequency and static deflection. From this fact, the structural analysis model in this study has been modelled as without joint part. At the beginning of structural modelling, the structural analysis model of this bridge was made as faithful model of 3 spans continuous girder laminated in 5 layers shown in Fig.16, and then eigenvalue analysis using the subspace method was carried out in order to confirm the validity of three dimensional structure analytical model. As a result, the vertical first vibration was 2.19 Hz, which is a value that was an approximation of the experimental value.

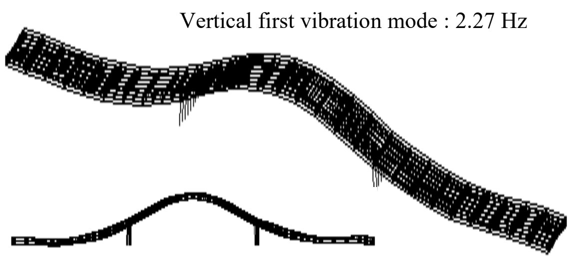

Figure 18. Analyzed vibration mode

However, it was proven that three dimensional dynamic response analysis was difficult on this structural model, because the node number is enormous. From this fact, the structural analysis model was improved again, and a structural model of 3 spans continuous girder laminated in 2 layers which considered the effect of bracing member was made. This structural model is shown in Fig. 17. The eigenvalue analysis was carried out for this structural model. The results are shown in the analyzed natural frequencies of Tables 2. The vertical first vibration mode analyzed is shown in Fig. 18.

4.2 Dynamic response Analysis

Three dimensional dynamic response analysis of this bridge due to walking of one pedestrian was carried out using the girder model laminated by 2 layers shown in Fig. 17. The response velocity waves at each analyzed point in resonant walking are shown in Fig. 20. As well as measured wave in Fig. 12, the vibration behaviour of three spans continuous girder has appeared clearly from this analyzed wave. The maximum value of measured point and analyzed point which is close to walking lane increases from other values, when analyzed wave in Fig. 20 and measured wave in Fig. 12 are compared. It is shown that lateral load sharing is clearly small, though the simple bracing member between each main girder has been installed. The maximum values and wave shapes based on experiment and analysis almost agree.

Power spectrum density function (PSD) was also required from spectrum analysis by FFT in order to examine frequency of response wave obtained the experiment and the analysis. As a result, the experiment wave became 2.25Hz, and the analysis wave became 2.24Hz. The validity of this three dimensional dynamic response analysis can be confirmed, because both peek spectra agree.

5 Vibration Serviceability

The evaluation of vibration serviceability may become less for rms value which has meaning of mean value for variational data. From this fact, vibration serviceability in this study was evaluated by maximum value of response velocity. Fig. 21 shows evaluation of vibration serviceability for the subject bridge by maximum response velocity obtained by the experiment and the analysis due to resonate waking of one pedestrian at edge part of up-stream. From this figure, it is proven that the measured and analyzed values cause both reactions of "not perceptible for vibration" or "lightly perceptible" on vibration serviceability of this bridge. It is also proven that that analyzed and measured values agree well and that the shape of response velocities resemble. From this fact, they seem to be able to evaluate vibration serviceability of the subject bridge analytically, even if dynamic testing of the actual bridge is not done, when the vibration characteristic of the subject bridge has been proven.

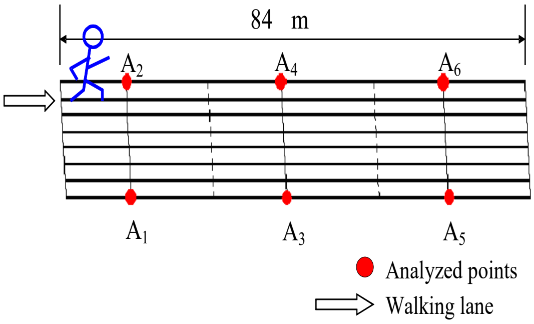

Figure 19. Analyzed point and walkin lane

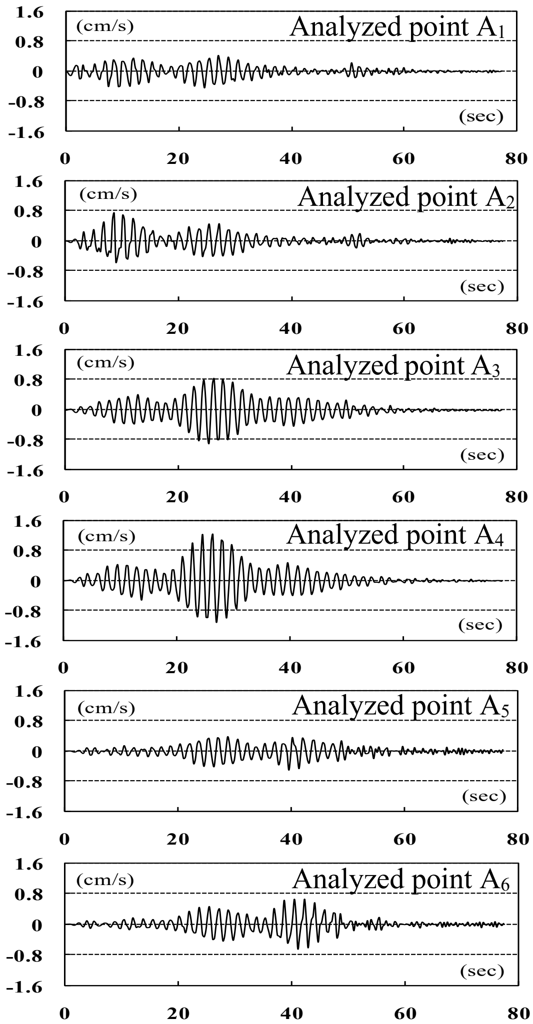

Figure 20. An example of response velocity analyzed by waking of one person with resonant pace of 2.25Hz at edge part of upstream

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As an example, vibration serviceability of the subject bridge was investigated by maximum value analyzed and measured response acceleration using the vibration limit of Canadaian Ontario Code and BS5400. The result is shown in Fig. 22. The measured and analyzed values are proven of becoming very close values, and both values are also within the permission region of the Ontario Code.

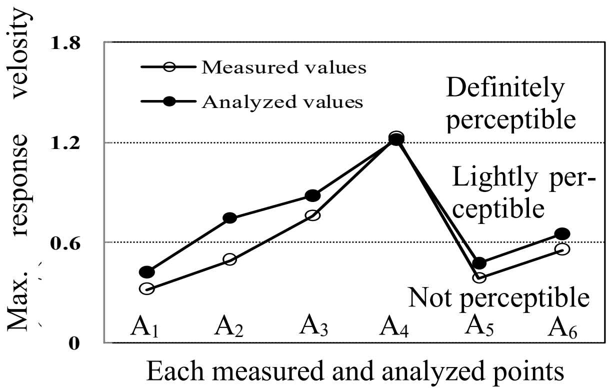

Figure 21. vibration serviceability of this bridge

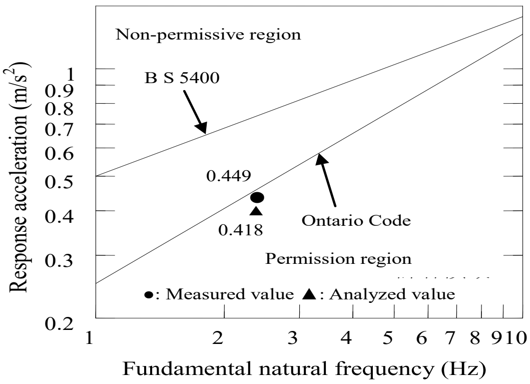

Figure22. An example on vibration serviceability of the objected bridge by maximum acceleration values using Ontario code and BS5400.

6 Conclusions

This study is done on a pedestrian bridge with large scale three spans continuous girder made by bongossi wood material. It investigated vibration serviceability of the timber pedestrian bridge based on experiment and analysis. The three dimensional dynamic response analysis of bridge due to walking and running of pedestrian was originally developed by FORTRAN program. The investigation of the dynamic response characteristics by the dynamic response analysis and evaluation of vibration serviceability were carried out, and vibration serviceability was finally verified from both sides of the experiment and the analysis for the bridge. The validity of this analysis was verified because measured and analyzed value agreed well. It seems that vibration serviceability of this bridge is the degree at which pedestrian senses "lightly perceptible" vibration, which causes no problem. As the result, this study verified that it is possible to evaluate vibration serviceability of the timber pedestrian bridge analytically.

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