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Evaluation of damping properties of reinforced concrete bridges and viaducts based on vibration measurement

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Abstract

Damping properties are important in evaluating the safety of structures or the running safety of railway vehicles during earthquakes. For instance, it is pointed out that the differences of the damping constant of railway structures may have caused the different damage of structures in 2011 off the Pacific coast of Tohoku Earthquake. However, there is no adequate evaluation method of damping properties at present because there are only a few measurement examples for damping properties and the occurrence factors of damping are very complex. In this study, we measured the damping constants and the natural periods of railway structures of various structural types and in various ground conditions by using two vibration measurement methods. As a result, we can notice that some dispersion exists between the empiric formula and the measured data. Anyway the natural period and the damping constant are inversely proportional to each other. Furthermore we define the amplitude ratio of upper side to lower side of the structure, and analyse the relationship between the damping constant and the amplitude ratio. As a result, there is a positive correlation between the damping constant and the amplitude ratio, and it seems that the damping constant of the whole structure is decided by the weight of structural damping properties and ground's ones.

Keywords: damping properties, vibration measurement, railway structures, natural period

1. Introduction

It is common knowledge that damping properties are important in evaluating the safety of structures or the running safety of railway vehicles during earthquakes. For instance, it is pointed out that the difference of damping constant of railway structures may have caused the different damage of structures in the well-known earthquake in Japan named 2011 off the Pacific coast of Tohoku Earthquake [1]. That's the reason why it is very important to evaluate damping properties for extracting such railway structures as should be paid special attention to in seismic assessment.

In this paper, damping properties mean the damping constants of the whole structures at the main vibrational mode during earthquakes. Some researches [2,3,4] were conducted to evaluate modal damping constants of long span bridges from seismic observations. However, they were only case studies in which damping properties were not generally discussed. On the other hands, there are some researches [5,6,7,8,9,10,11] about damping constants of bridges by statistical approach based on the past measurement results. However, there is no adequate evaluation method of damping properties at present because measurement methods are different among the respective data and there are only a few measurement examples in contrast with many types of structure and many ground conditions.

For that reason, parameters of damping constants are widely set in seismic design code [12] for railway structures in Japan. Indeed, the damping constant of the concrete material is 3-5 % and the ground's one is 15-30 %.

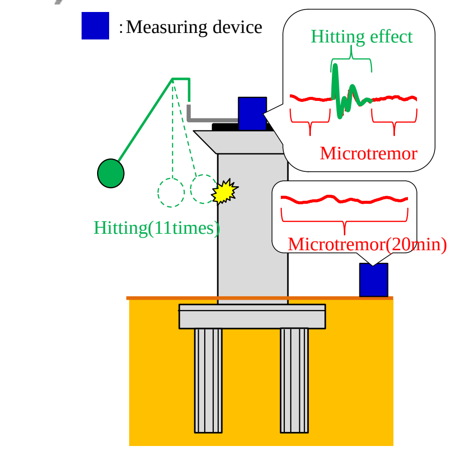


Fig.1–Microtremor measurement and impact vibration test

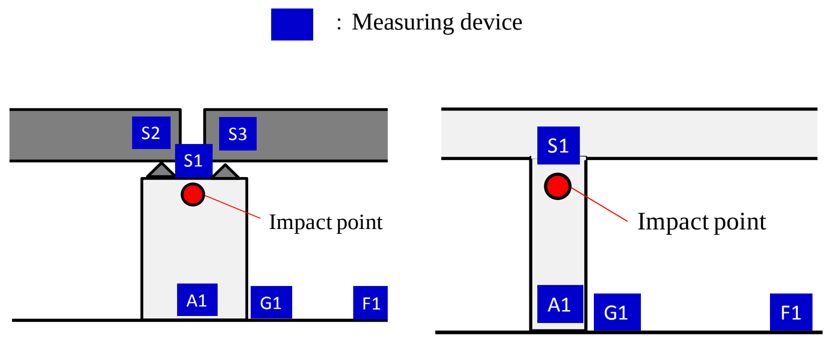


Fig.2–Installation position

In this study, we use two major vibration measurement methods named impact vibration test and microtremor measurement on reinforced concrete bridges and viaducts for railways of various structural types and in various ground conditions, and try to evaluate damping constants and natural periods.

2. Outline about vibration measurement

In this chapter, we explain about the outline of vibration measurement named impact vibration test (I.v.t) and microtremor measurement (M.m) in order to clarify the vibrational characteristics of railway structures. As is described later in this section, two types of measurement named “simple measurement” with minimum sensors and “detailed measurement” with more sensors are conducted.

2.1 Measuring condition

Measurement time was set to 20 minutes at one site, and the impact force was induced by hitting 30kg weight to the upper side of the structures 11 times during microtremor measurement (Fig.1). Sampling frequency was set to 200 Hz, and 100 Hz low pass filter was applied to the data.

2.2 Measuring device

In this study, high sensitive velocity measuring devices are installed and measure the microtremor and the force of impact in order to clarify the vibration characteristic of structures.

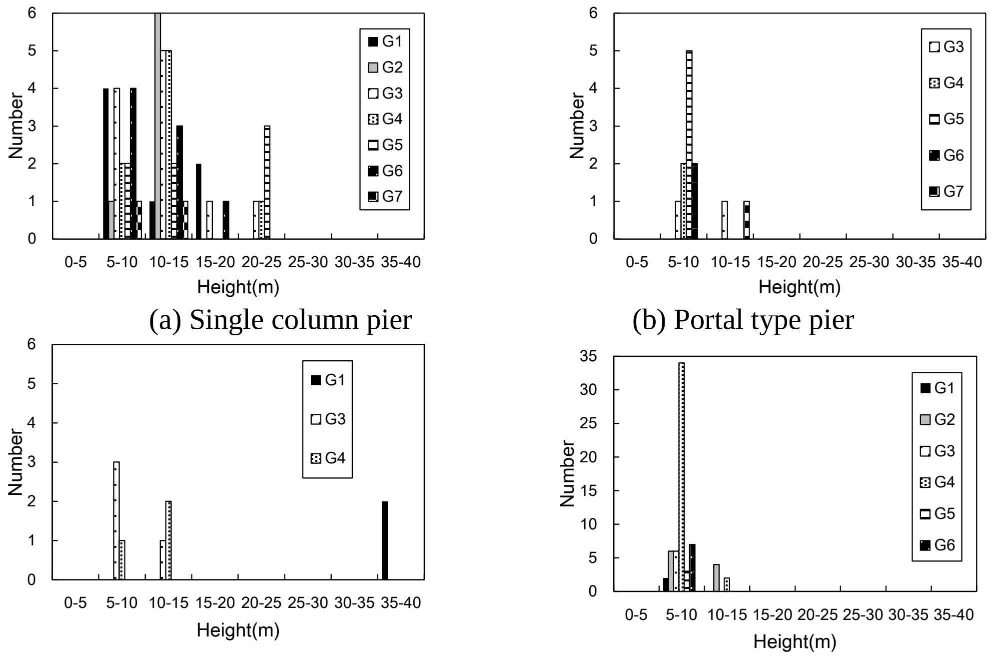
2.3 Installation position

Fig.2 shows the installation positions of high sensitive velocity measuring devices in the case of simple measurement. S1 and A1 are the devices installed at the upper side and the lower side of the structure, and G1 and F1 are the devices installed on the ground near the structure and in the free field. In the case of pier type structures, S2 and S3 are installed at bridge girders. Red circle means the point of impact.

In the case of detailed measurement, additional sensors are installed at neighboring piers and the middle part of girders in order to verify whether the data obtained from the simple measurement have enough quality and quantity for evaluating damping constants. Installation positions and analyses of detailed measurement are mentioned in chapter 4.

2.4 Reinforced concrete bridges and viaducts for the measurement

The total number of railway structures of many types and in various ground conditions for the measurement is 136 as shown in Fig.3. Furthermore, the legends G1-G7 in Fig.3 indicate ground classification defined in Japanese seismic design code for railway structures [12]. Fig.4 shows the images of the respective structural



(a) Single column pier (b) Portal type pier

(c)Wall type rigid frame (d) Portal type rigid frame

Fig.3–Numbers of structures for measurement

(a) Single column pier (b) Portal type pier (c)Wall type rigid frame (d) Portal type rigid frame

Fig.4–Structural types for measurement

types.

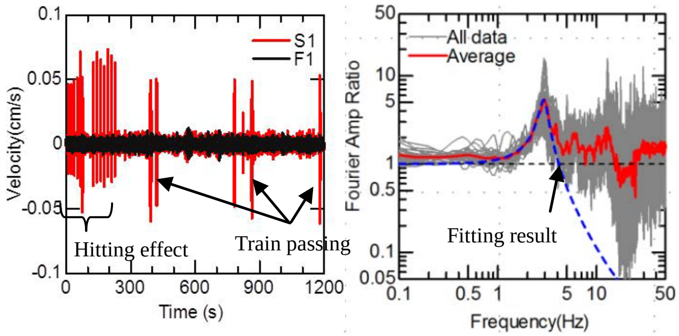
3. Methods for evaluating the damping constant

3.1 Microtremor measurement

First, the time history data were divided into data of 20.48 seconds (the number of data is 4096) and the fourier amplitude spectrum was calculated from each divided data set. Then, the data including noise from train passing or the weight being hit were excluded in the analysis (Fig.5(a) black arrows).
Second, the ratio of the fourier amplitude spectrum at the upper side of the structure (S1) to that on the free field (F1) was calculated. Then, the smoothing technique named Parzen windows at 0.2 Hz was applied.
The ratio of the fourier amplitude spectrum (Fig.5(b) red line) obtained from the above procedure represents the transfer function from input ground motion to structural response. Therefore, the damping constant of the first mode of the whole structure was evaluated by the half power method and the curve fitting method (Fig.5(b) blue dotted line). These two methods are as follows.

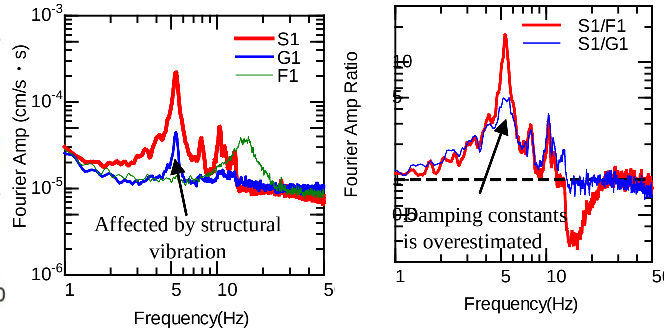
3.1.1 Half power method

Damping constant h is calculated from Eq. (1).



(a) Time history (b) Calculate transfer function

Fig.5– Evaluating method of damping constants by using microtremer measurement



(a) Fourier amplitude (b) amplification ratio

Fig.6– Exemplary result of microtremer measurement (S1:upper side of the structure, G1:ground near the structure, F1:free field)

$$h = \frac{f_1 - f_2}{2 f_0} \tag{1}$$

where f_0 means the first natural frequency of the ratio of the fourier amplitude spectrum, and f_1 and f_2 mean the frequencies where the respective amplitudes are $1/\sqrt{2}$ times as much as those at f_0 (1/2 times in power spectrum).

3.1.2 Curve fitting method

Damping constant h is obtained by the method of the least squares of the ratio between the observed fourier amplitude spectrum and the transfer function of single degree of freedom system $H(f)$ shown in Eq (2).

$$H(f) = \frac{\sqrt{1+(2h\beta)^2}}{\sqrt{(1-\beta^2)^2+(2h\beta)^2}} \tag{2}$$

where β means the ratio of a frequency f to the first natural frequency f_0 (f/f_0).

3.2 Consideration for applying microtremor measurement

Fig.6(a) shows the fourier amplitude spectrum obtained by microtremer measurement in the free field (F1), on the ground near the structure (G1), and at the upper side of the structure (S1), and Fig.6(b) shows the amplification ratio of the structure (S1) to each ground (G1 and F1). The ground vibration near the structure (G1) is larger than that in the free field at around the first natural frequency of the whole structure (about 5Hz) because of being affected by the structural vibration. Therefore, the amplification ratio S1/G1 is smaller than that of S1/F1 at the peak point. It leads to overestimating the damping constant. It means that it is necessary to measure the ground vibration which is not affected by the structural vibration, that is, the free field vibration.

3.3 Impact vibration test

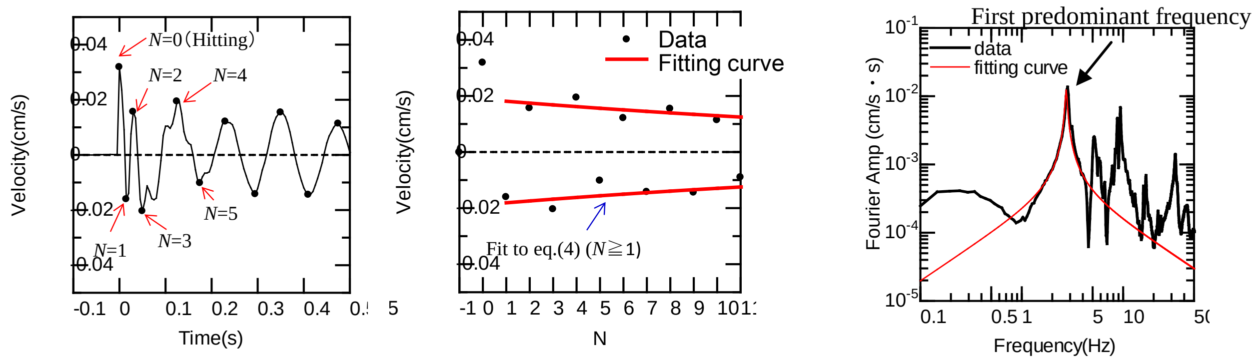
3.3.1 Method of using the time history of free vibration

The observed time history of velocity $v(t)$ in the state of free vibration after adding the impact force is shown in Fig.7(a). Damping constant h is determined by fitting the amplitude of $v(t)$ to the solution for the free vibration of a single degree of freedom system with damping as written in Eq. (3)

$$v(t) = A\exp(-2\pi fht) \tag{3}$$

where A is the initial amplitude and f is the natural frequency. There are three unknown parameters h, A and f in order to fit the observed results to Eq. (3). In this paper, the peak number N is defined (Fig.7(a) black circle) and the velocity waveform is rearranged by the peak number N shown in Fig.7(b). As a result, the solution is rewritten as Eq. (4) and the number of unknown parameters is decreased by including the facultativity of the natural frequency f in the independent variable N.

$$v(N) = A\exp(-2\pi hN) \tag{4}$$



(a) Free vibraion time history (b) Free vibration using parameter N (c) The method of using mobility

Fig.7–Evaluating methods of damping constants by using impact vibration examination

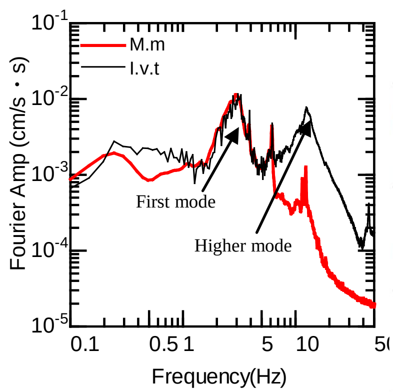


Fig.8–Case where impact vibration test is not applicable to the evaluation of the damping constant

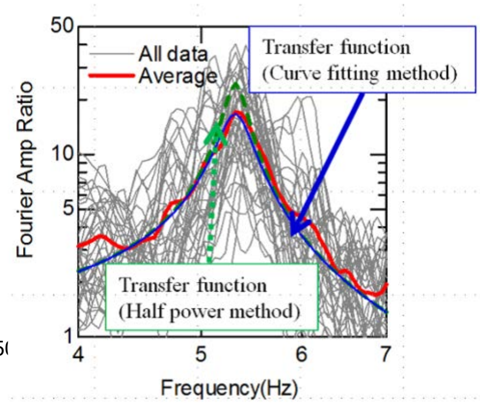


Fig.9–Comparison of accuracybetween the half power method and the curve fitting method

Table.1–Damping constants obtained by the respective methods

Method		*h*
I.v.t	Free vibration	0.024
	Mobility	0.032
M.m	Half power	0.020
	Curve fitting	0.030

Damping constant *h* is obtained by the method of the least squares of Eq. (4). In addition, the peak amplitude at hitting timing (corresponding to $N = 0$) is excluded because that timing is in a state different from free vibration.

3.3.2 Method of using mobility

Assuming that the impact force is proportional to the delta function, velocity fourier amplitude spectrum *F(f)* is proportional to the frequency response function about mobility *G(f)* as shown in Eq. (5).

$$F(f) \propto G(f) = \frac{f/k}{\sqrt{(1-\beta^2)^2 + (2h\beta)^2}} \tag{5}$$

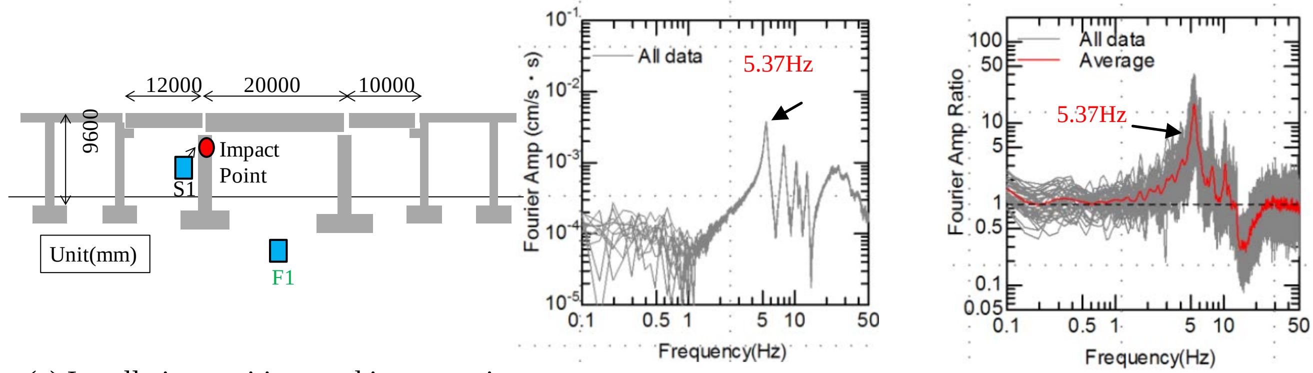
where *k* means the stiffness of the structure. Thus, damping constant *h* is obtained by the method of the least squares of the observed velocity fourier amplitude spectrum and Eq. (5) (Fig7.(c)).

3.4 Consideration for applying the impact vibration test

In some cases, the first mode of the structure is not excited by impact force and the amplitude at the first natural frequency obtained by the impact vibration test is nearly equal to that by microtremor measurement as shown in Fig.8. It is difficult to evaluate the damping constant from free vibration or mobility. Therefore, we use the results obtained by microtremor measurement.

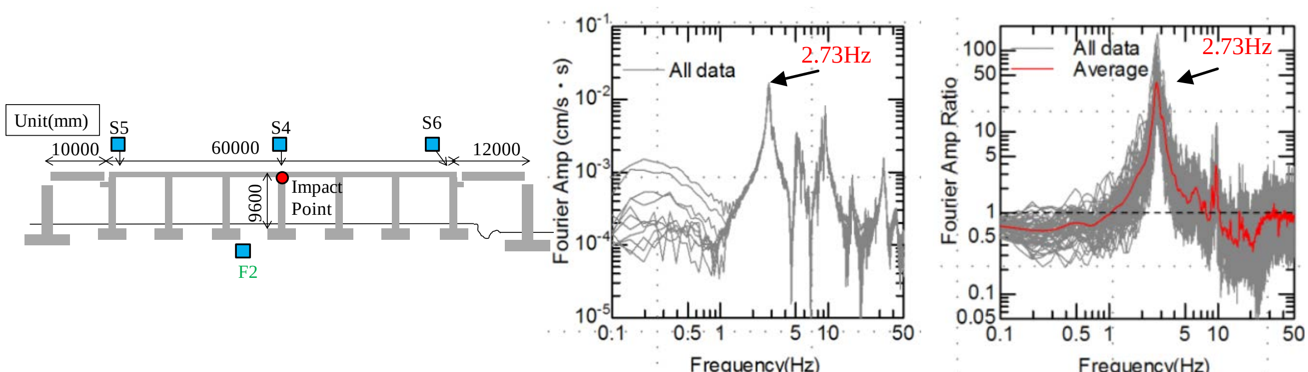
3.5 Difference in damping constants obtained by respective methods

Table.1 shows the example of damping constants obtained by respective methods. In the cases of using microtremor measurement, the transfer function evaluated by the curve fitting method give a more similar form to the observed results than that of the half power method (Fig.9). It means that the curve fitting method has



(a) Installation positions and impact point (b)Fourie ampulitude of S1 (c) Ratio of the fourier amplitude spectrum at S1 to that at F1

Fig.10–Installation positions and measurement results in the case of detailed measurement for the single column pier ((b)I.v.t, (c)M.m)



(a) Installation positions and impact point (b)Fourie ampulitude of S4 (c) Ratio of fourier amplitude spectrum at S4 to that at F2

Fig.11–Installation positions and measurement results in the case of detailed measurement for the Wall type rigid frame((b)I.v.t, (c)M.m)

higher estimate accuracy than the half power method because the former result is obtained by the method of the least squares to the transfer function $H(f)$.

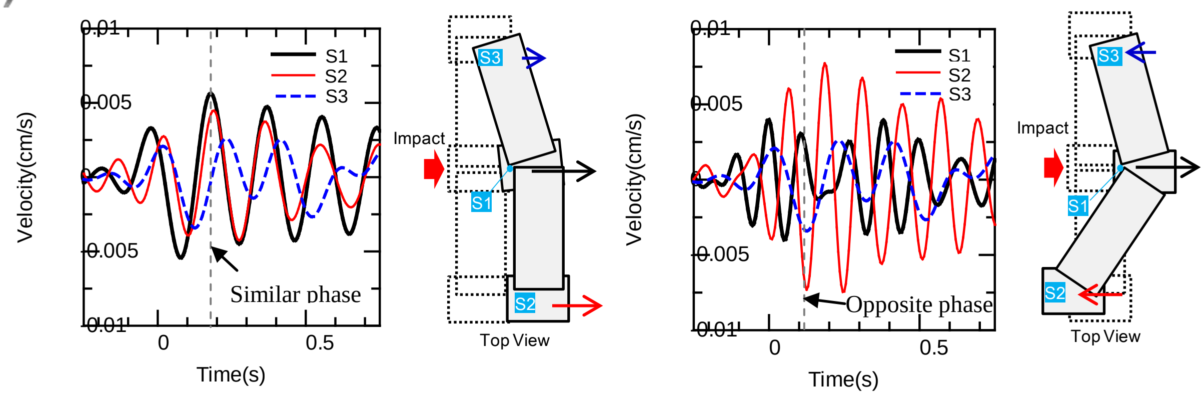
In the cases of using the impact vibration test, the damping constant obtained from the method of using mobility is similar to that from the curve fitting method. On the other hands, there is a divergence of the damping constant from the result obtained by the method of using the time history of free vibration with several components of natural frequency. It is necessary to analyse the estimate accuracy of respective methods in the future.

4. Verification of the damping constant obtained by the vibration measurement

In this chapter, we verify whether the damping constant obtained by the curve fitting method corresponds to the damping constant of the whole structures in the main vibrational mode during earthquakes. For that purpose, the vibrational characteristics of the structures regarding which detailed measurement was conducted are analysed.

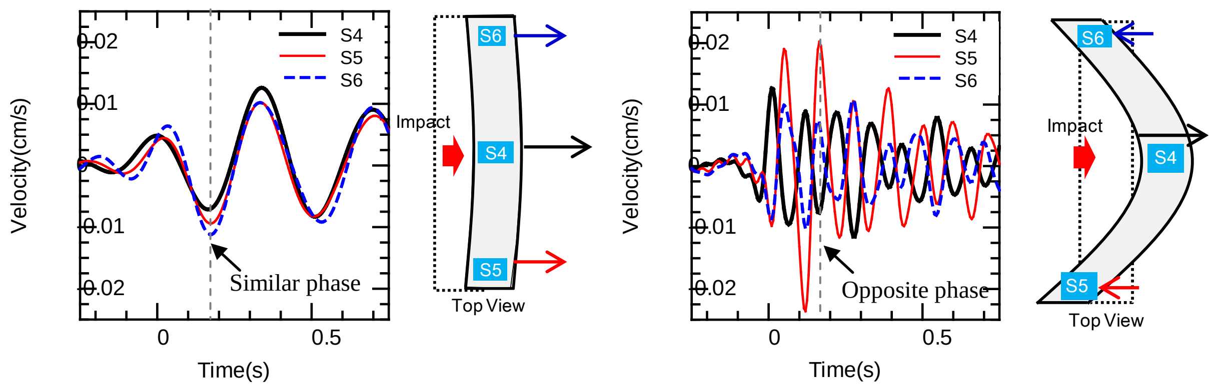
The installation positions of sensors in the cases of single column piers and wall type rigid frames are shown in Fig.10 and Fig.11.In addition, the fourier amplitude spectrums of the impact vibration test and the transfer functions from the free field to the upper side of the structure are shown in Fig.10 and Fig.11. Fig.10(b) and Fig.10(c) show that the structures have a number of predominant frequencies. Moreover, the first predominant frequencies of respective measurement results (arrows in respective Figures) are clearly identified and they are in close agreement with each other. The same tendency is seen in Fig.11(b) and Fig.11(c).

First, Fig.12 shows the velocity time histories of the hit pier and the neighboring piers after applying the impact force in the case of a single column pier. The respective velocity time histories are filtered by a digital filtering



(a) 7Hz low pass filter (b)7-12Hz bandpass pass filter

Fig.12–Velocity time histories and presumed mode shapes (single column pier)



(a) 5Hz low pass filter (b)5-20Hz bandpass pass filter

Fig.13–Velocity time histories and presumed mode shapes (Wall type rigid frame)

process in order to separate respective modes. In the case of the first mode of the hitted pier (Fig.12(a)), the phase difference among the respective velocity time histories is small. It means that the whole structure is vibrated in the same direction in the first mode (right of Fig.12(a)). On the other hands, in the case of the second mode (Fig.12(b)), the hit pier is vibrated with the phase opposite to that of the neighboring piers. It means that the hit pier is vibrated locally in the second mode (right of Fig.12(b)).

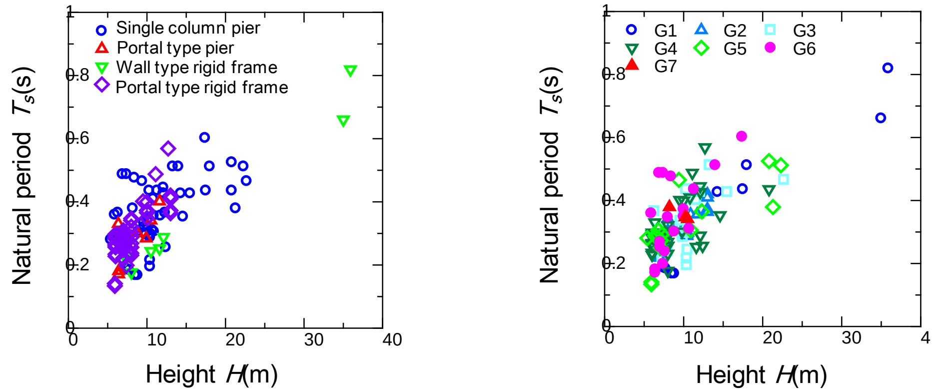
Next, Fig.13 shows the velocity time histories of the hit column and the neighboring columns after applying the impact force in the case of a wall type rigid frame. The respective velocity time histories are filtered by a digital filtering process in order to separate respective modes. In the case of the first mode of the hitted column (Fig.13(a)), the phase difference among the respective velocity time histories is small. It means that the whole structure is vibrated in the same direction in the first mode (right of Fig.13(a)). On the other hands, in the case of the second mode (Fig.13(b)), the hit column is vibrated with the phase opposite to that of the neighboring columns. It means that the hit column is vibrated locally in the second mode (right of Fig.13(b)).

From the above results, it is found that the first mode of the hit pier or column corresponds to the mode in which the whole structures are vibrated in the same direction. Such mode is expected to appear as the main vibrational mode during earthquakes, so it is found that the damping constant obtained by the curve fitting method is valid.

5. Estimation of damping constants based on all the measurement results

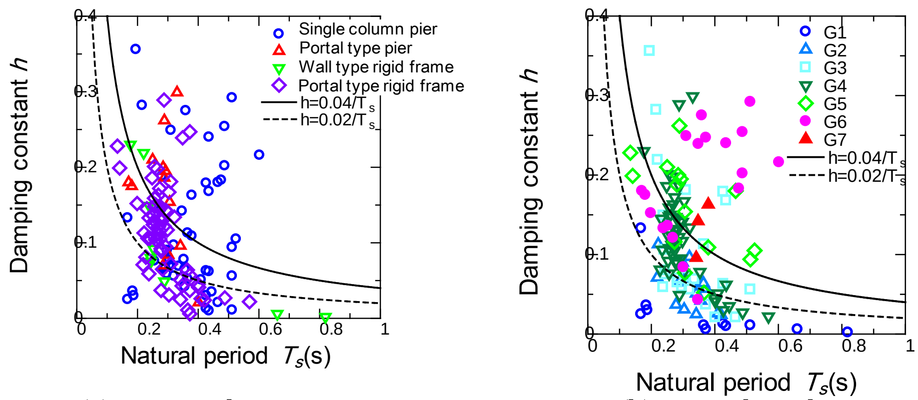
5.1 Relationship between heights and natural periods

Fig.14 shows the relationship between the heights of the measured structures and the natural periods. It is found that the heights have a positive correlation to the natural period. Furthermore, there is not a clear difference among the respective structural types or ground conditions.



(a) Structural types (b) Ground conditions

Fig.14–Relationship between natural periods and heights



(a) Structural types (b) Ground conditions

Fig.15–Relationship between damping constants and natural periods

5.2 Relationship between damping constants and natural periods

Fig.15 shows the relationship between damping constants and natural periods. Here, the damping constant is estimated by the curve fitting method. An empiric formula [8,12] about the damping constant and the natural period (h=0.02/T_s, 0.04/T_s) is also plotted in the figure. We can notice that some dispersion exists between the empiric formula and the measured data. Anyway the natural period and the damping constant are inversely proportional according to the empiric formula.

Damping constants which are larger than the empiric formula's ones are obtained from the data in soft ground condition (ground type G4-G7). On the other hands, most damping constants which are smaller than the empiric formula's ones are obtained from the data in hard ground condition (ground type G1-G3). It seems that the damping constant is dependent on ground stiffness (natural period).

5.3 Relationship between damping constants and amplitude ratios

From the results in chapter 5.2, the damping constant of the whole structure seems to have correlation with the ground's natural period. Past researchs [13,14] pointed out that the ratio of the structural natural period to the ground's one has correlation with the radiational damping. In this study, we define the amplitude ratio α of the upper side to the lower side of the structure (Fig.16). Amplitude ratio α=0 means that there is no ground displacement, in other words, only relative displacement between the upper and the lower part of the structure occurs. In contrast, amplitude ratio α=1 means that there is no relative displacement between the upper and the lower part of the structure, in other words, only ground displacement occurs. That is to say, amplitude ratio α is related to the contribution of ground displacement.

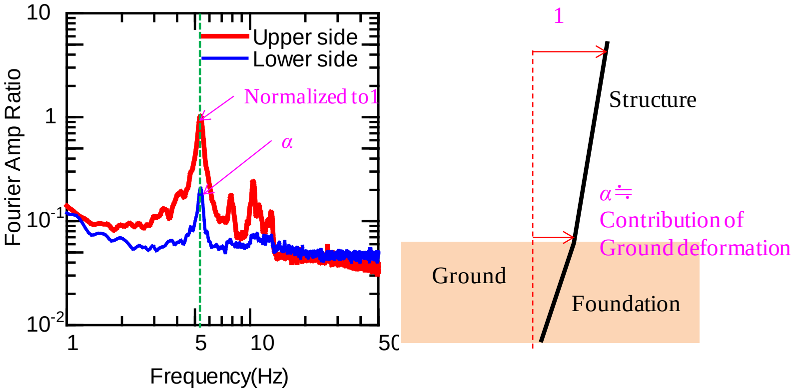


Fig.16–Amplitude ratio α of the upper side to the lower side of the structure

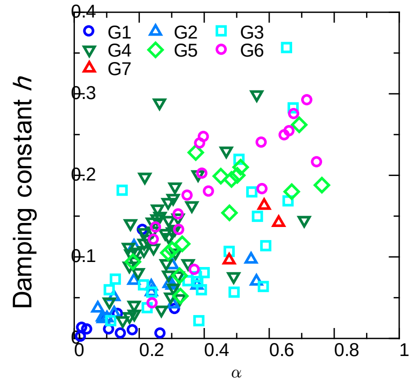


Fig.17–Relationship between damping constants and amplitude ratios

Fig.17 shows the relationship between damping constants and amplitude ratios. As a result, there is a positive correlation between the damping constant and the amplitude ratio. It means that the damping constant is large in the cases of large α (the contribution of ground displacement is large) because of the large contribution of radiational damping, and the damping constant is small in the cases of small α (the contribution of ground displacement is small) because of the large contribution of material damping. Moreover, it seems that amplitude ratio α is large in the soft ground condition (ground type G4-G7).

These results seem that the damping constant of the whole structure is decided according to the weight of structural damping properties and ground's ones, and they are in good agreement with the concept of the strain energy proportional method which is often used in the Japanese seismic designs.

In addition, this study is targeted to the damping constants of the whole structures including the foundations and the ground. This is because the damping constants consist of the internal damping (relevant to structural relative displacement) and radiational damping (relevant to ground's displacement), so the simplest model of single degree of freedom including these damping components is applied to consider the basic characteristic of dynamic response of the structure. Moreover, this study is targeted to the damping constant at small amplitudes (linear behavior) based on impact vibration test and microtremor measurement. In the case of strong motion (non-linear behavior), there is an additional damping of hysteretic damping [15].

6. Conclusions

In this study, we measured the damping constants and the natural periods of railway structures of various structural types and in various ground conditions by using two vibration measurement methods named impact vibration test and microtremor measurement. As a result, we can notice that some dispersion exists between the empiric formula and the measured data. Anyway the natural period and the damping constant are inversely proportional according to the empiric formula. Furthermore, we define the amplitude ratio of the upper side to the lower side of the structure which is related to the contribution of ground deformation, and analyse the relationship between damping constant and amplitude ratio. As a result, there is a positive correlation between the damping constant and the amplitude ratio, and it seems that the damping constant of the whole structure is decided according to the weight of structural damping properties and ground's ones. In addition, the above result is in good agreement with the concept of the strain energy proportional method which is often used in the Japanese seismic designs.

In the future, we will try to clarify some dispersion between the empiric formula and the measured data and propose the estimation method of the damping constant of the structure and that of the ground separately.

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References

[1] Japan Transport Safety Board (2013) : Railway accident and incident reports. (in Japanese)

[2] Siringoringo, D. M. and Fujino, Y. (2006) : Observed dynamic performance of the Yokohama-Bay Bridge from system identification using seismic records, *Structural Control and Health Monitoring*, **13**, 226-244.

[3] Siringoringo, D. M. and Fujino, Y. (2007) : Dynamic characteristics of a curved cable-stayed bridge identified from strong motion records, *Engineering Structures*, **29**, 2001-2017.

[4] Siringoringo, D. M. and Fujino, Y. (2007) : System identification applied to long-span cable-supported bridges using seismic records, *Earthquake Engineering and Structural Dynamics*, **37**, 361-386.

[5] Manabu ITO, Tsuneo KATAYAMA (1965) : Damping of bridge structures, *Proceedings of the Japan Society of Civil Engineers*, **117**, 12-21. (in Japanese)

[6] Masafumi KATO, Shizuo SHIMADA (1981) : Statistical analysis on the measured bride vibration data, *Proceedings of the Japan Society of Civil Engineers*, **311**, 49-58. (in Japanese)

[7] Masafumi KATO, Shizuo SHIMADA (1983) : Statistical analysis on the pier vibrational characteristics by the measured data, *Proceedings of the Japan Society of Civil Engineers*, **338**, 229-232. (in Japanese)

[8] Eiichi KURIBAYASHI, Toshio IWASAKI (1970) : Earthquake resistant design of bridges (III) – experimental studies on vibrational damping of bridges **–** , *Research Report of Public Works Research Institute*, **139**, 1-12. (in Japanese)

[9] Tetsuo TAKEDA, Shinichi YAMANOBE and Yuji NIIHARA (1998) : Damping property of prestressed concrete cable-stayed bridges, *Journal of Japan Society of Civil Engineers*, **626(I-48)**, 147-161. (in Japanese)

[10] Yuji NIIHARA, Tetsuya KOHNO, Shinichi YAMANOBE and Yoshihiro HISHIKI (2001) : Study on damping characteristics of PC extradosed bridges, *Journal of structural engineering*, **47A**, 489-500. (in Japanese)

[11] Meguru TSUNOMOTO and Yasuo KAJIKAWA (1999) : Estimation of damping ratio of stress ribbon bridges, *Journal of Japan Society of Civil Engineers*, **612(I-46)**, 337-348. (in Japanese)

[12] Railway technical research institute (2012) : Design Standards for Railway Structures and Commentary (Seismic Design), Supervised by Ministory of Land, Infrastucture and Transportaion, Maruzen. (in Japanese)

[13] Toshihiro WAKAHARA and Kazuhiko KAWASHIMA (1991) : Radiational damping of road bridge piers supported by rigid foundation, *Journal of Japan Society of Civil Engineers*, **432(I-16)**, 145-154. (in Japanese)

[14] Kenzo TOKI (1978) : Seismic response and aseismic design of bridge foundation, *Disaster Prevention Research Institute Annals*, **21A**, 1-24. (in Japanese)

[15] Moaveni B, He X, Conte J, J Restrepo and M Panagiotou, (2011) : System Identification Study of a 7-Story Full-Scale Building Slice Tested on the UCSD-NEES Shake Table, *Journal of Structural Engineering*, **137**, No 6, 705-717.