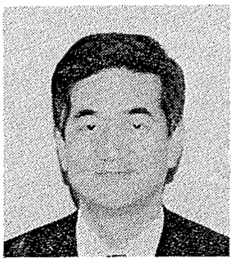
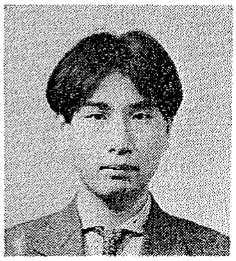
CONCRETE LIBRARY OF JSCE NO. 30, DECEMBER 1997

A STUDY ON THE VIBRATIONAL CHARACTERISTICS OF RIGID-FRAME BRIDGES AFTER THE HYOGO-KEN NAMBU EARTHQUAKE

(Translation from Proceedings of JSCE, No.550/V-33,November 1996)



Masaki Seki



Kenta Mizutani

Akihiko Nishimura

Satoshi Nakano

Some of the elevated rigid-frame bridges on the New Tokaido line were damaged by the Hyogo-ken Nambu Earthquake. After restoration, the dynamic percussion test was performed by the authors on the Shinkansen's rigid-frame bridges throughout the disaster area.

These tests were intended to ascertain the bridges' natural frequencies in order to check their serviceability. Some of the bridges tested had suffered a decline in the natural frequencies as a result of the earthquake. Suspecting that this was caused by either a decrease in the column rigidity of the bridge or the bearing capacity of the soil, eigen-value analysis was conducted by the authors to determine the cause of the decline.

Additionally, the authors selected a rigid-frame bridge that had an increased natural frequency as a result of the restoration work, and ascertained the cause of this increase. Then it was found that the serviceability of rigid-frame bridges could be evaluated through the use of natural frequencies by the dynamic percussion test.

Keywords: earthquake damage, rigid-frame bridge, natural frequency, dynamic percussion test, serviceability

---

Masaki Seki is a General Manager in the tracks and structure department of Shinkansen Operations Division at Central Japan Railway Company. He is a member of JSCE.

Kenta Mizutani is a Chief Assistant in the construction department at Central Japan Railway Company. He is a member of JSCE.

Akihiko Nishimura is a General Manager in the foundation engineering group at Railway Technical Research Institute. He obtained his Dr. of Eng. from Tokai University in 1992. He is a member of JSCE.

Satoshi Nakano is a Section Manager in the structural design department at Fukuyama Consultants Co.,Ltd. He is a member of JSCE.

1. INTRODUCTION

The Hyogo-ken Nambu Earthquake, the epicenter of which was located at the northern part of Awaji Island, occurred at 5:46 a.m. on January 17, 1995. In addition to claiming many lives, the earthquake caused major damages of roads, railways, port facilities, subways, and other infrastructures. The railway infrastructures that had major damage were bridges, embankments and underground structures of the Sanyo Shinkansen, Japan Railways conventional lines, private railways, and subways, primarily within the area in which the seismic intensity was 7 [1]. On the New Tokaido Line as well, the columns of rigid-frame bridges had damages in Takatsuki area between Kyoto and Shin Osaka.

Since 1991, Central Japan Railways Co. has performed the dynamic percussion test (explained below) as a method of investigating the serviceability of concrete structures on the New Tokaido line. This method was adopted to continually investigate and monitor the fatigue and deterioration of the Tokaido Shinkansen's concrete structures that have been in service for more than 30 years, since the line commenced operation in 1964. Central Japan Railway Co. is accumulating sequential measurement data obtained from the dynamic percussion test.

To ascertain the serviceability of concrete structures after the earthquake, Central Japan Railway Co. performed the dynamic percussion test on structures in earthquake-stricken areas. The test was performed on structures that suffered no visible damage; structures where the soil had loosened around the foundations; and structures on which restoration work had been performed.

The dynamic percussion test is used to judge the serviceability of a structure based on changes in its vibrational properties. This test was performed on rigid-frame bridges that did not require restoration work after the earthquake, obtaining valuable measured data showing that the bridges natural frequency declined after the earthquake. We also confirmed that the natural frequency of certain rigid-frame bridges has increased as a result of restoration or repair work.

The research described in this paper, was conducted through analyses based on actual measured values using a spring-mass model. By doing so, focus was placed on the rigidity of RC columns and ground spring constants as factors in the decline of the natural frequency of rigid-frame bridges as a result of the earthquake. The authors were thus able to evaluate the decline in column rigidity and ground spring constant, and accordingly propose their method as a technique for future evaluation of post earthquake serviceability of concrete structures.

2. OVERVIEW OF DAMAGE AND RESTORATION WORK

The damage of rigid-frame bridges on the New Tokaido line occurred along an approximately 12 km stretch near Takatsuki City, which is located between Kyoto and Shin Osaka (Fig. 1). The damage was mainly in the columns of rigid-frame bridges. In many cases, concrete on the upper or lower part of the columns had cracked and fallen off. A total of 172 columns in 30 bridge blocks received damage. Their damage classified by degree is shown in Table 1.

The columns that received Class-A, -B, or -C damage were restored with steel-plate covers (Fig. 2-(a),(b)). When even one column in a rigid-frame bridge block required steel-plate restoration, the columns of the entire block were reinforced with steel plates, in order to maintain equilibrium in each column's rigidity.

As for the restoration work of foundations, cement paste was poured to fill the open space between the soil and the footings of columns where separation had occurred. In addition, the columns that received Class-D damage were reinforced with polymer mortar grouting.

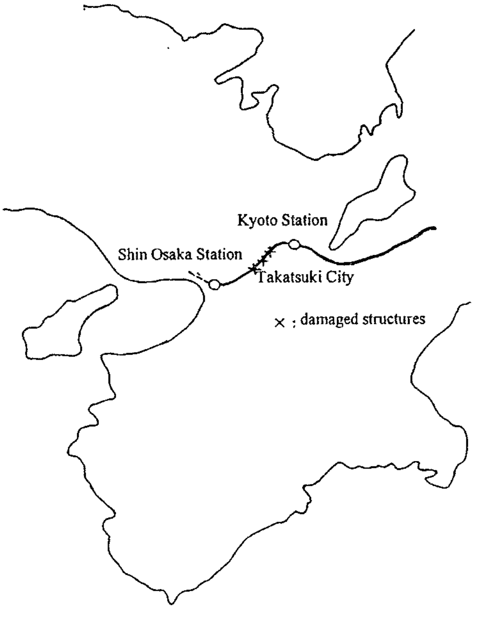


Fig. 1: Location of Damaged Structures

Table 1: Classification by Degree of Damage

Degree of Damage		Countermeasures
A	Partial destruction of concrete	Main reinforcement exposed Reinforced with steel plates
B	Separation of covering concrete	
C	Cracks	
D	Slight cracks	Polymer mortar grouting

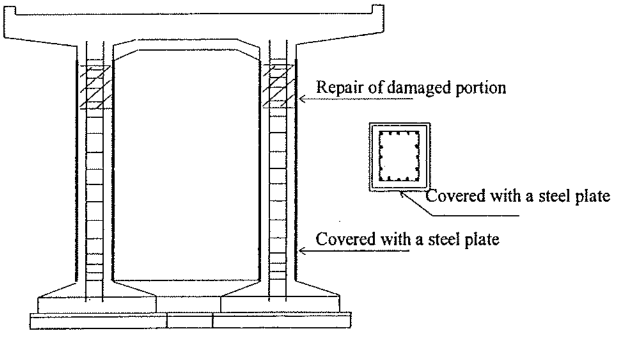


Fig. 2(a) Diagram of the Restoration of a Rigid-Frame Bridge (Class A, B)

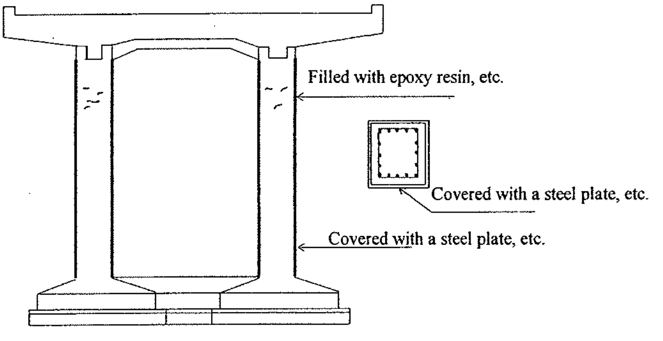


Fig. 2(b) Diagram of the Restoration of a Rigid-Frame Bridge (Class C)

3. DYNAMIC PERCUSSION TEST

3.1 Research Results Due to The Dynamic Percussion Test and The Role of This Research

Dynamic percussion testing is conducted to evaluate the serviceability of railway bridge foundations where direct visual inspection is not possible [2]. It is performed to measure the bridge's natural frequency based on the response when a structure's member is struck with a heavy bob. The structure's serviceability is judged from the values thus obtained.

Various studies have previously been conducted to evaluate the serviceability of substructures by means of the dynamic percussion test.

The main objective of these studies is the confirmation of two factors, first, that the natural frequency of a structure changes due to a diminution in soil-bearing capacity resulting from scouring of soil around the foundation, and second, that this change can be ascertained with sufficient accuracy using the data obtained with the dynamic percussion test [3]. The natural frequency used in these studies was invariably that of the first vibration mode, which is a vibrational property similar to that of rigid-body vibration.

Moreover, the structural stiffness ascertainable through reverse analysis using the measured values of natural-frequency were primarily ground spring constants. Additionally, the measurement of the first-mode natural frequency of the rigid-body vibration of a structure can be made through the use of not only the dynamic percussion test, but also techniques such as microtremor measurement, depending on the scale of the structure and the underlying soil conditions [4].

In the case of a rigid-frame bridge, the dynamic percussion test is used primarily to measure the bridge's natural frequency (referred to below as the first natural frequency of the overall rigid-frame bridge) from the properties of the response generated by striking a heavy bob to the top of the bridge at a right angle to the railroad track, as shown in Fig. 3.

In contrast to piers, however, which exhibit vibrational properties similar to rigid-body vibration, rigid-frame bridges predominantly exhibit bending vibration. In this case, by conducting reverse analysis using the measured values, two types of stiffnesses can be ascertained: ground spring constants and the flexural rigidity of the bridge's columns. In fact, a study in which the column rigidity of a rigid frame bridge was examined, reported that the condition of the column's construction joints was the cause decreased rigidity [5].

In terms of the deterioration of rigid-frame bridges, the factors generally cited are a decrease column or foundation rigidity due to cracking or deterioration of the concrete, and a decrease in the spring constant of the foundation due to local ground subsidence or other causes. As a long-term countermeasure against damage, Central Japan Railway Co. has been investigating serviceability based on yearly changes in the vibrational properties of individual rigid-frame bridges. The natural frequency of a rigid-frame bridge is determined by the mass of columns and members on them, the flexural rigidity of the columns, and the ground spring constants. Because a decrease in column rigidity or soil-bearing capacity will manifest itself as a decrease in natural frequency [6]; the serviceability of rigid-frame bridges is evaluated by ascertaining the changes in natural frequency.

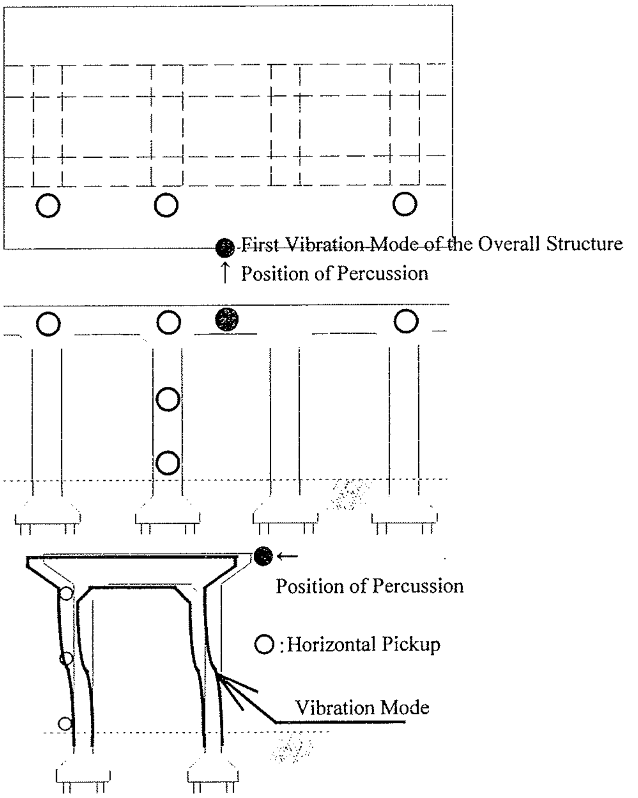


Fig. 3: Diagram of the Dynamic Percussion Test Performed to the Top of a Bridge (First Vibration Mode of the Bridge's Overall Structure)

In this paper, the authors decided to attempt to identify the primary factors that cause changes in the natural frequencies of rigid-frame bridges by ascertaining the column rigidity and ground spring constants of the bridges. To evaluate column rigidity more accurately, one must measure the higher-order natural frequencies (the second natural frequency or more) and vibration modes, which significantly exhibit the effects of changes in rigidity [5][7]. In this study, the natural frequencies and vibration modes caused by striking the columns of rigid-frame bridges at mid-height were also measured (hereinafter referred to as the "second natural frequency of the column").

None of the previous studies concerning evaluation of the serviceability of structures by the dynamic percussion test have actually compared measured values before and after the occurrence of damage. Rather, in all these studies, the decline in soil-bearing capacity is estimated based on experiments conducted on actual structures with the periphery of their foundations artificially excavated to simulate scouring. Therefore, the results are based on the comparison of bearing capacity in normal conditions with that subsequent to soil scouring. In addition, the decrease in column rigidity was derived either through comparison with other normal columns or through evaluation of rigidity by reverse analysis.

There are also many previous studies that confirmed the state of damages of structures following an earthquake by means of vibration-table or alternate-loading tests [8][9]. In this study, the dynamic percussion test was used to check the degree of damage to structures both before and after they encountered an actual earthquake. Results obtained from this study suggest that the dynamic percussion test is extremely effective in evaluating the serviceability of structures after an earthquake.

3.2 Matters Investigated

The specific dynamic percussion tests for evaluating the serviceability of rigid-frame bridges are described below.

a) First Natural Frequency of the Overall Structure
As shown in Fig. 3, we ascertained the natural frequency and modal form of the bridge by striking a heavy bob to the top of its center at a right angle to the railway tracks, and measuring the horizontal vibration velocity at three locations on the top of the bridge and two locations on the column. The form of the overall structure's first vibration mode is shown in Fig. 3.

b) Second Natural Frequency of Columns
As shown in Fig. 4, we measured the natural frequency of the bridge by using a beetle to strike the column at mid-height at a right angle to the railway tracks, and measuring the horizontal vibration velocity at three locations on two columns, one to the percussion side and one across therefrom. The shape of the columns' second vibration mode is shown in Fig. 4.

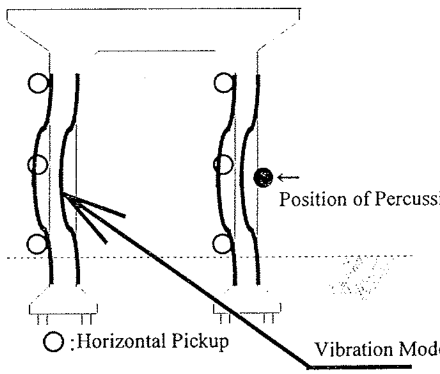


Fig. 4: Diagram of the Dynamic Percussion Test Performed on the Midheight of a Bridge's Column (Second Vibration Mode of the column)

3.3 Bridges Inspected and Results

For this study, the dynamic percussion test was performed to ascertain the serviceability of structures in the quake-stricken area. In the process, a detailed analysis was conducted of the three rigid-frame bridges described below.

a) Bridge A
Bridge A is a three-span (3m + 3 x 6m+3m) standard rigid-frame bridge (Fig. 5), with footings connected by underground link beams. It sustained no discernible outward damage by the earthquake. Bridge A had also been subjected to the dynamic percussion test (in 1991) prior to the earthquake.

Figure 5(a) shows the Fourier amplitude spectrum transformed from the impact-response waveforms measured prior to the earthquake. The peak on the left side of the Fourier spectrum (marked with an arrow in the diagram) is the first natural frequency of the overall structure. While the waveform of the response generated by the dynamic percussion test is a damped free vibration, this response is excited by an artificial external force. Hence, the natural frequency of the waveform determined based on the response velocity is not only confirmed as the peak of the Fourier amplitude, but also as the frequency indicating a phase-difference angle of 180o from the moment when external force is applied to the bridge, in the phase-difference response spectrum.

Likewise, the peak at the right side of the Fourier amplitude spectrum is the second natural frequency of the columns. When the Fourier spectral amplitude corresponding to the frequency of this peak was used to draw a diagram of the column's vibration mode, the resultant modal form exhibited its maximum amplitude in the middle of the column, as shown in Fig. 5(c). Therefore, it was concluded that the peak on the right side of the Fourier spectrum is the second natural frequency of the column as shown in Fig. 4.

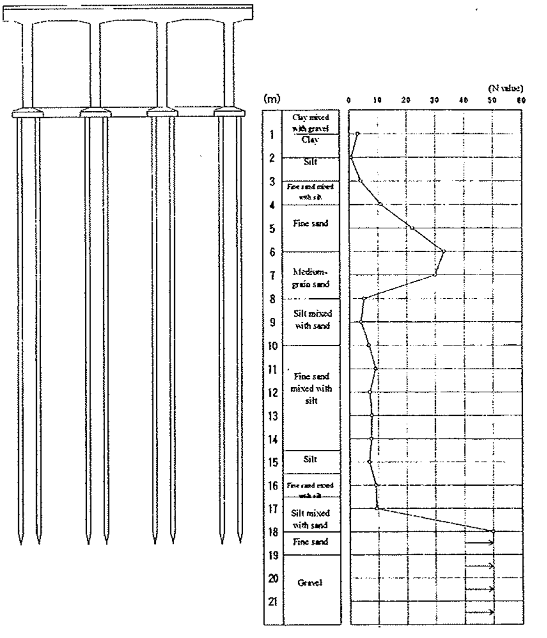


Fig. 5: Bridge A

In addition, the phases of the left and right columns at this moment were synchronous (confirmed based on the phase-difference spectrum and the phase of the impact waveform). This finding confirmed that the columns' second vibration mode would assume a modal form like that shown in Fig. 4.

Figure 5(b) is the Fourier amplitude spectrum of the response waveform measured after the earthquake. Comparing Figures 5(a) and 5(b), we can see that after the earthquake, the first natural frequency of the overall structure and the second natural frequency of the columns had declined by 0.4 Hz and 12 Hz, respectively.

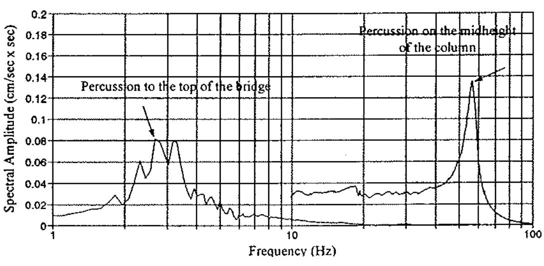


Fig. 5(a): Bridge A pre-earthquake
(For the percussion on the midheight of the column, only the frequency band of 10 Hz - 100 Hz is shown.)

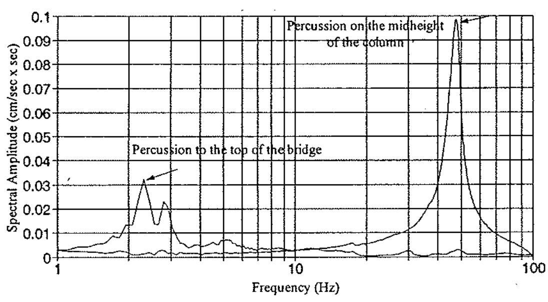


Fig. 5(b): Bridge A post-earthquake

b) Bridge B

Bridge B is a three-span (3m + 3 x 6m + 3m) standard rigid-frame bridge (Fig. 6) with isolated footings. After the earthquake, openings were found in the bottom section of its columns. These openings were subsequently filled with cement paste. We performed the dynamic percussion test before and after this repair work.

Figure 6(a) is the Fourier amplitude spectrum prior to the repair work, while 6(b) is the Fourier amplitude spectrum after the repair work. The natural frequency of the overall structure increased by 0.2 Hz following the repair work.

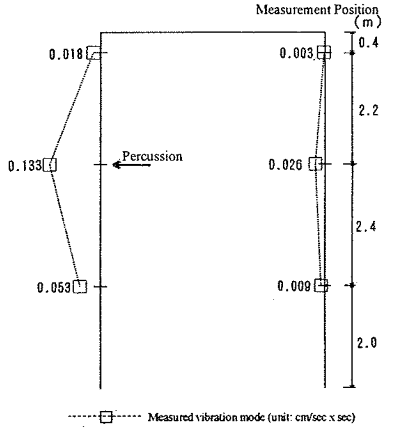


Fig. 5(c): Measured Vibration Mode of the Columns of Bridge A

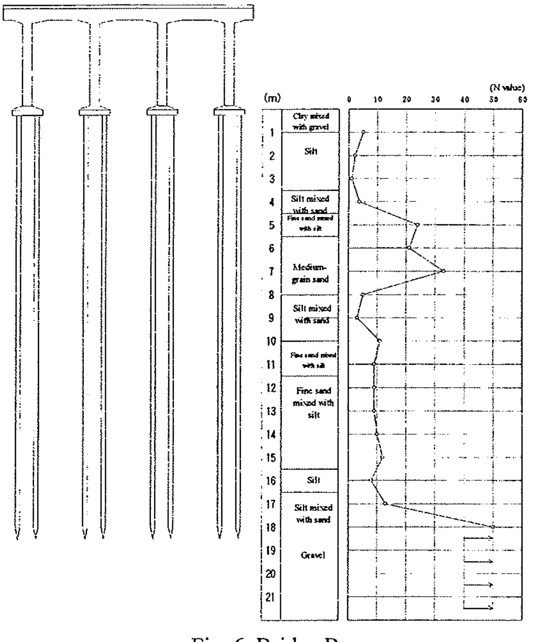


Fig. 6: Bridge B

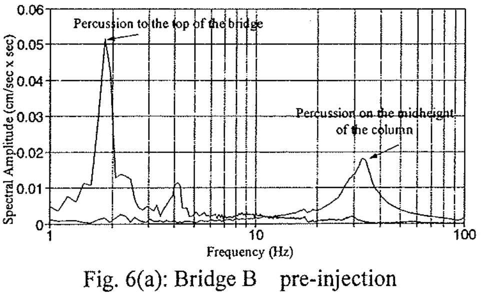


Fig. 6(a): Bridge B pre-injection

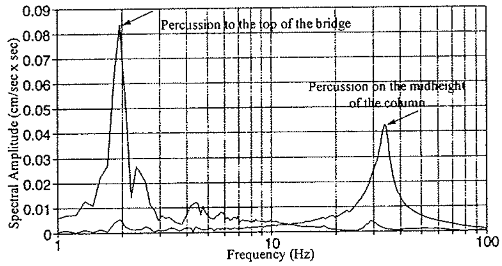


Fig. 6(b): Bridge B post-injection

c) Bridge C

Bridge C is a three-span (3m + 3 x 6m + 3m) standard rigid-frame bridge (Fig. 7), with footings connected by underground link beams. Its columns were damaged by the earthquake and reinforced with steel plates. The dynamic percussion test results revealed that the natural frequencies of Bridge C, after restoration, (Fig. 7-1) are higher than those of an undamaged rigid-frame bridge of the same type (Fig. 5(a)).

On the basis of the above results, we made a model for each rigid-frame bridge and practiced modal analysis.

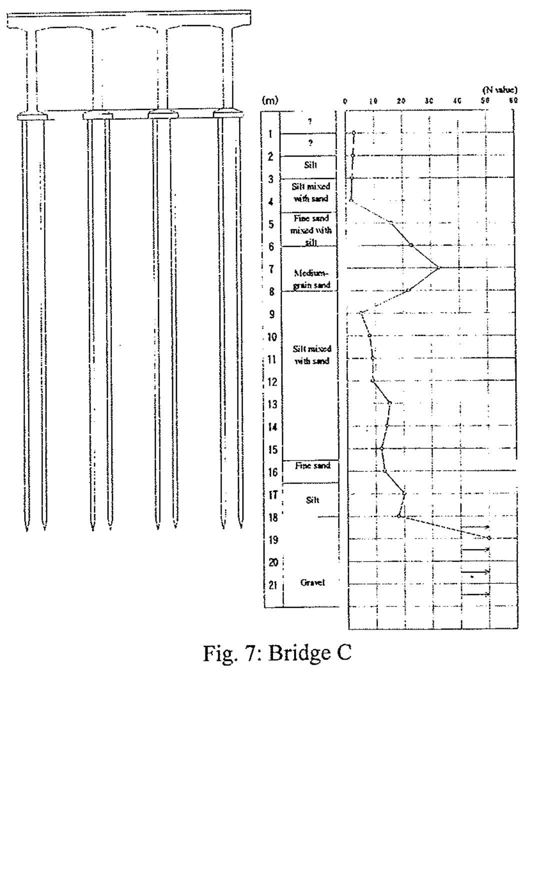


Fig. 7: Bridge C

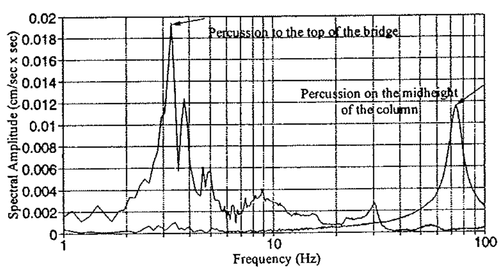


Fig. 7-1: Bridge C after Reinforcing

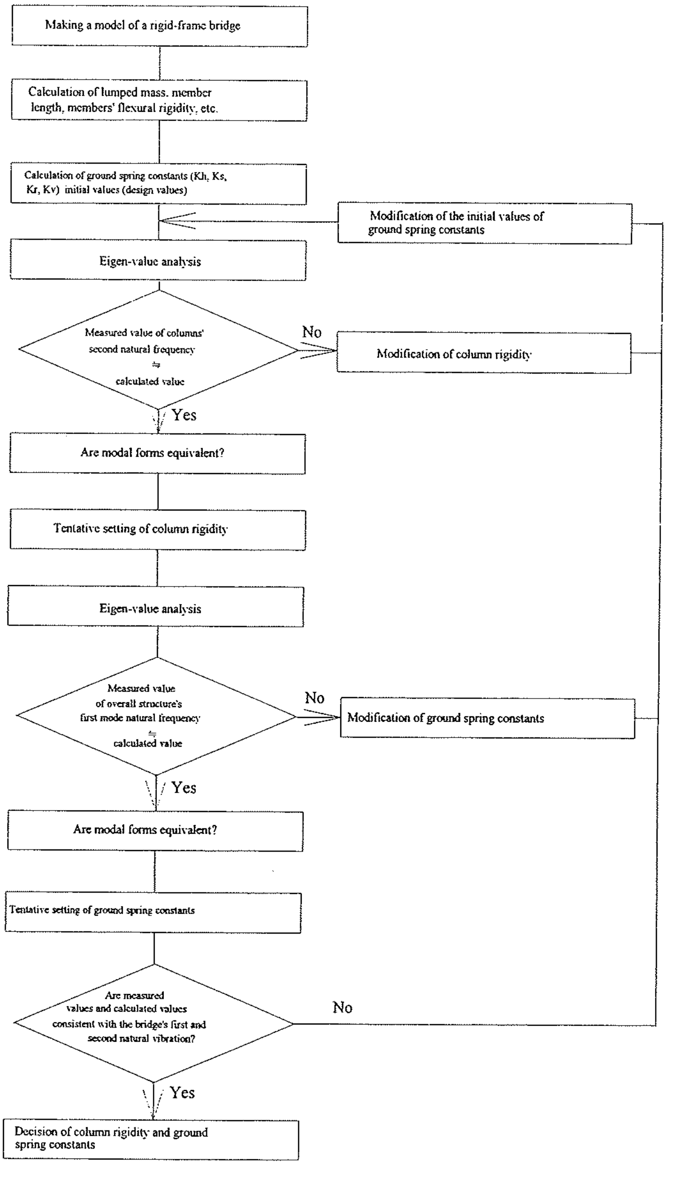


Fig. 8: Flow of Analysis

4. EIGEN-VALUE ANALYSIS

4.1 Analytic Model and Procedure

Vibrational characteristics of rigid-frame bridges are primarily represented by natural frequency and vibration mode. The factors that determine the natural frequency of a rigid-frame bridge include the mass of its skeleton and members, the flexural rigidity of its columns, and the bearing capacity of the surrounding soil.

Simulation analysis of the results of the dynamic percussion test by eigen-value analysis is conducted by estimating the rigidity or ground spring constant of a structure, based on its natural frequency and natural vibration mode. Through this process, the factors that cause a change in natural frequency can be investigated by ascertaining the flexural rigidity and ground spring constants of columns, provided that the structure's mass remains constant. In terms of damages considered attributable to earthquakes, decreases in rigidity due to cracks in columns or foundation damage (loosening of soil, damage to foundation piles, etc.) is evaluated.

In eigen-value analysis, column rigidity and ground spring constant are calculated based on the measured natural frequencies and vibration mode of a bridge. In the case of rigid-frame bridges, however, a change in column rigidity has virtually no effect on the first vibration mode. Therefore, the second natural frequency of the column was incorporated into this analysis. The flow of the eigen-value analysis we conducted is shown in Fig. 8. The analytic model of a rigid-frame bridge used for analysis is depicted in Fig. 9.

First, lumped mass, member length, and the flexural rigidity of members was calculated from by the existing design calculation sheets. Next, the ground spring constant was determined based on the coefficient of subgrade reaction obtained from boring data. In this case, to calculate the coefficient of subgrade reaction, the following equations 10) of the foundation design standard were applied.

$$k_h = 0.2\alpha E_0 D^{-\frac{3}{4}} \qquad (1)$$

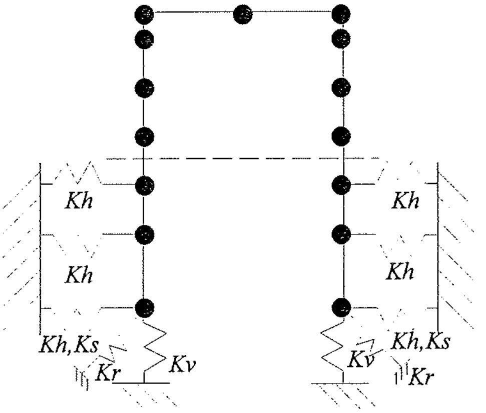
$$E_0 = 25N \qquad (2)$$

where, a: correction coefficient (=2); E_0: coefficient of ground deformation; D: pile diameter;
N: N value; K_h: horizontal coefficient of subgrade reaction

After calculating the above mentioned initial values, eigen-value analysis can then be conducted. In conducting eigen-value analysis of a rigid-frame bridge, the column rigidity and the ground spring constant is adjusted so that the measured natural frequencies and vibration modes obtained through the dynamic percussion test are equal to their analytic values. The measured natural frequencies are obtained from the spectra (amplitude spectra and phase-difference spectra), which is derived from Fourier transformation of the bridges' response waveform at the time of the heavy bob's percussion.

For the vibration modes, the results calculated as amplitude ratios of spectral amplitudes of the frequency of the applicable order were used. These ratios were calculated from the Fourier spectra (amplitude spectra) of the response waveforms obtained from various survey points of the bridge (at the time when the percussion was applied at the indicated point).

First, the column rigidity was set so that the form of the columns' second vibration mode and the amplitude ratios for the top, center, and bottom of the columns coincided with the measured values. Using the column rigidity thus calculated, the ground spring constant was set so that the first natural frequency and vibration mode of the overall structure became consistent with the measured values.



Kh: Horizontal spring constant; Ks: Shearing spring constant
Kv: Vertical spring constant; Kr: Rotational spring constant

Fig. 9: Diagram of an Analytic Model

Table 2: Analytic Results: Bridge A

	Pre-earthquake	Post-earthquake
EI	4.586×10^8 N·m^2	3.273×10^8 N·m^2
K_h	6.262×10^8 N/m	3.131×10^8 N/m
K_v	4.396×10^9 N/m	4.396×10^9 N/m
K_r	6.330×10^9 N·m/rad	6.330×10^9 N·m/rad

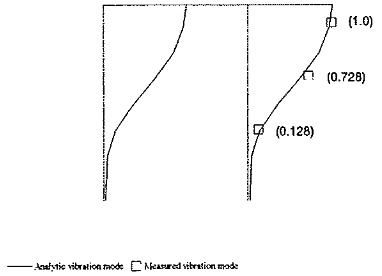


Fig. 10-(a: Pre-earthquake): Vibration Mode Shape

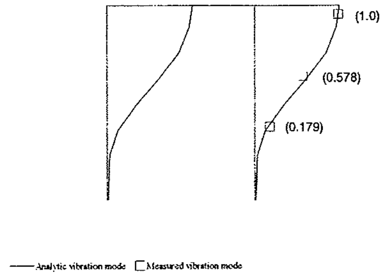


Fig. 10-(b: Post-earthquake): Vibration Mode Shape

4.2 Analytic Results: Bridge A

The analytic results are shown in Table 2 and the vibration mode shapes by the simulation are shown in Fig. 10-(a: Post-earthquake) and Fig. 10-(b: Post-earthquake).

a) Prior to the Earthquake

Based on the analytic results, the column rigidity was assumed to be equal to full-section effective rigidity prior to the earthquake, and set Young's modulus (E) of the concrete at 2.30 x 10^{10} N/m^2. In light of the vibrational displacement by the dynamic percussion test (0.01 mm displacement at the top of the bridge and 0.0015 mm at the column base; ground strain on the order of 10^{-6} - 10^{-7}) by one percussion, we evaluated the ground spring constants were eight times the values shown in the Foundation Design Standard for an earthquake.

b) After the Earthquake

Based on the analytic results, column rigidity after the earthquake decreased to approximately 70% of its pre-earthquake level. This magnitude of the decrease is consistent with a state of slight cracks on the column surfaces.

On comparing the ground spring constants before and after the earthquake, the horizontal spring constant was found to have decreased by 50%. This decrease is consistent with the results of an inspection after the earthquake, which confirmed that the earthquake caused separation of approximately one centimeter between the columns and the soil. The mode shape of the lower portion of the column (see Fig. 10(a),(b)) exhibits this phenomenon.

Table 3: Analytic Results: Bridge B

	Pre-injection	Post-injection
EI	3.332×10^8 N·m^2	3.332×10^8 N·m^2
K_h	2.600×10^8 N/m	2.600×10^9 N/m
K_v	1.013×10^9 N/m	1.013×10^9 N/m
K_r	1.459×10^9 N·m/rad	1.459×10^9 N·m/rad

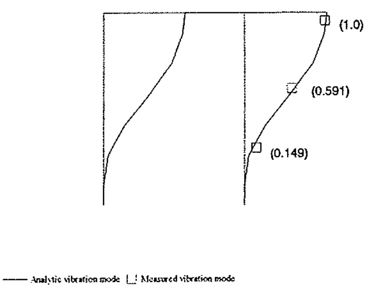


Fig. 10-(c: Post-injection): Vibration Mode Shape

4.3 Analytic Results: Bridge B

The analytic results are shown in Table 3, and the vibration mode shapes generated from the simulation are shown in Fig. 10-(c: Post-injection).

On comparing the values for the horizontal ground spring constant subsequent to the earthquake with those after the injection, the latter was increased by tenfold. This increase is attributable to the cement paste that was poured into the openings around the bottom of columns, which had been found, by a visual inspection conducted shortly after the earthquake. As a result, the horizontal spring constant of the ground was increased. In other words, because the soil was loose shortly after the earthquake, cement paste was injected into the ground and spread to the openings. Consequently the looseness of the soil was rectified, and the horizontal spring constant was increased.

The vertical and rotational spring constants of the ground showed no change between immediately after the earthquake and following the injection. The bridge's piles were excavated beneath the footings and in a visual inspection of the pile heads found to be free from damage.

On the basis of the above analytic results, the increase in the natural frequency of the bridge from 1.8 Hz to 2.0 Hz is considered attributable to the increase in the horizontal spring constant of the ground that resulted from the injection of cement paste shortly after the earthquake.

Column rigidity showed no change between immediately after the earthquake and after the injection. However, in comparison to values derived from design calculations and the standard rigidity values of columns (σ_c=23.5 Mpa; E=2.45 x 10^{10} N/m^2; I: full-section effective) obtained through the dynamic percussion test on rigid-frame bridges of the same standard type as Bridge B, the column rigidity had declined to approximately 78%.

Table 4: Analytic Results: Bridge C

	After Steel-Plate Reinforcement
EI	9.947×10^8 N·m²
K_h	7.792×10^7 N/m
K_v	7.049×10^8 N/m
K_r	7.771×10^9 N·m/rad

4.4 Analytic Results: Bridge C

The analytic results are shown in Table 4.
The dynamic percussion test had not been performed on Bridge C prior to the earthquake. Bridge C had Class-A damage, and all columns were reinforced with steel plates. Although Bridges A and C do not have the same soil conditions, in terms of the type of rigid-frame bridge, they are identical. Accordingly, by performing a comparison based on the first natural frequency of their overall structures, it is possible to relatively evaluate the effect of reinforcing the columns with steel plates.

The difference between the natural frequencies of Bridge C following reinforcement and that of Bridge A prior to the earthquake is 0.5 Hz. In light of the effect of the soil of Bridges A and C (as a comparison of Tables 2 and 4 shows, the ground spring constant of Bridge C is generally low), it is clear that this difference in natural frequency is attributable to the increased rigidity due to the restoration of the columns with steel plates.

5. STUDY RESULTS

Through both the dynamic percussion tests and the eigen-value analysis the effects of earthquakes on rigid-frame bridges were ascertained in terms of the ground spring constants and column rigidity of the bridges. The Hyogo-ken Nambu Earthquake, which caused major damage to rigid-frame bridges, had a greater effect on the decrease in column rigidity than it did on the decrease in ground spring constants due to such causes as soil loosening. This was clear from the results of visual inspections that confirmed that the foundation piles of other damaged bridges suffered no damages.

Table 5 summarizes the results of this study.

To counter soil loosening, cement paste was injected into the openings around the footings, resulting in a tenfold increase in the horizontal spring constant. This is considered to be due to damages characterized by visible openings between the footings and the ground. As a repair measure, compacting and grouting of the soil around the foundation is clearly effective. This does not necessarily mean, however, that soil strengthening is effective as a prior countermeasure against earthquakes. The improvement in safety resulting from soil strengthening must be studied through techniques such as dynamic analysis.

The results of the dynamic percussion test performed as part of this study, revealed that column reinforcement is amply effective. In addition, by virtue of having obtained measured values through the dynamic percussion test of rigid-frame bridges both before and after the earthquake, it was possible to explain the causes of the decreased natural frequencies of rigid-frame bridges, and the subsequent increase in natural frequency due to reinforcing work.

The influence of column rigidity and ground spring constants on eigen-value analysis has been reported by Nishimura [6]. Nishimura states that in terms of the results of numerical analysis, a change in column rigidity affects the first natural frequency of the overall structure, but results in little change in the vibration modes. Conversely, a change in the ground spring constants has a greater effect on the vibration mode than it does on natural frequency. These findings were corroborated by the measured values obtained through the dynamic percussion test.

Additionally, it was found that more accurate evaluation of column rigidity is possible by focusing on the second natural frequency and vibration mode of columns. This procedure increased the accuracy of analysis, and evaluation of serviceability. By employing design calculations based on design loads and structural conditions, including the bridge's column rigidity and ground spring constants thus calculated, one can evaluate the serviceability of a rigid frame bridge with respect to strength and deformation.

Table 5: Summary of Study Results

Bridge	Natural Frequency			Reinforcing
	Before Earthquake	Immediately After Earthquake	After Repairs	
Bridge A First Mode of the Overall Structure:	2.8 Hz	2.4 Hz		
	Decline in soil-bearing capacity			
Second Mode of the Columns:	59.0 Hz	47.3 Hz		
	Decrease in Rigidity			
Bridge B First Mode of the Overall Structure:		1.8 Hz	2.0 Hz	Mortar grouting
		Increase in soil-bearing capacity		
Bridge C First Mode of the Overall Structure:			3.3 Hz	
Second Mode of the Columns:		Columns damaged	80.3 Hz	Restoration with steel plates
	(Decrease in rigidity)			

6. CONCLUSION

Based on the research described in this paper, it was shown that the serviceability of bridges can be evaluated based on measured natural frequencies. This is because the column rigidity and soil-bearing capacity of rigid-frame bridges are quantitatively evaluated by eigen-value analysis based on measured frequencies and vibration modes.

It is possible to evaluate the serviceability of rigid-frame bridges as an absolute value by checking safety through loading of the design load on a structural model of the bridge, using analytically obtained column rigidity and ground spring constants.

However, the most accurate index for judging serviceability is the change in the individual vibrational characteristics of a bridge. Once measured, these characteristics serve as base data for countermeasures against fatigue and deterioration due to the flow of time. In addition, it is clear that this approach is amply effective in the evaluation of serviceability in cases where it is difficult to determine through visual inspection whether a bridge has earthquake damage, or how a structure is affected by adjacent construction work.

Lastly, the proposal that it is preferable to investigate the vibration modes of more of a bridge's columns in order, to more accurately find changes in the column rigidity and ground spring constants of bridges.

References

[1] Great Hanshin Earthquake Damage Survey: Emergency Meeting Papers, The Japan Society of Civil Engineers, 1995

[2] NISHIMURA AKIHIKO and SHIRO TANAMURA, Research Concerning Techniques for Judging the Serviceability of Existing Bridges' Piers, RTRI Report, Vol. 3, No. 8, Sept. 1989

[3] TSUTSUMI, Yoji, Kazutoshi Yamashita, and Satoshi Nakano, Using Natural Frequency to Investigate the Serviceability of Bridge Piers During River Swelling, 50th Annual JCSE Academic Symposium, I-150, 1995

[4] NAKAMURA, Yutaka, Muneyuki Taboshin, and Toshio Kobayashi, Changes in the Vibrational Characteristics of Bridge Piers Resulting from Scouring: A Comparison of Microtremors and Dynamic Percussion Test, 48th Annual JCSE Academic Symposium, I-107, 1993

[5] SUZUKI, Shigeru, Yoji Tsutsumi, and Akihiko Nishimura, Evaluation of the Serviceability of Bridges by the Dynamic Percussion Test, 49th Annual JCSE Academic Symposium, V-85, 1994

[6] NISHIMURA, Akihiko, Research on a Method of Evaluating the Serviceability of Rigid-Frame Bridges, RTRI Report, Vol. 4, No. 9, Sept. 1990

[7] IMAI, Kenichi and Akihiko Nishimura, A Detailed Investigation Method for Bridge Foundations by the Dynamic Percussion Test, 48th Annual JCSE Academic Symposium, V-238, 1993

[8] KAWASHIMA, Kazuhiko and Tatsuhiko Ogawa, Effects of the Number of Loading Cycles on the Dynamic Characteristics of RC Bridge Piers, Proceedings of the JSCE, No. 392, April 1989

[9] ISHIBASHI, Tadayoshi and Shinichi Yoshino, Research on the Deformability of RC Bridge Piers During Earthquakes, Proceedings of the JSCE, No. 390, Feb. 1989

[10] The Japan National Railway, Explanation of Structural Design Standards: Foundations and Pile Earth-Retaining Structures, pp. 182-189, March 1986