Repair of hollow columns in rigid-frame PC bridge damaged by the 2016 Kumamoto Earthquake and examination of performance recovery by monitoring

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Abstract

An intensive earthquake with a magnitude of 7.3 occurred in the Kumamoto area in April 2016. The earthquake severely damaged the Aso Choyo Ohashi bridge (four-span continuous rigid-frame PC box girder). Severe cracks were developed at the mid-height hollow section of the bridge column, some of which penetrated the concrete wall of the hollow section. Previous studies have shown that such cracks deteriorate the shear strength of the hollow column as well as the stiffness. To recover the shear strength of the column, high-fluidity concrete was filled into the void of the overall height of the hollow section. The effects of the concrete filling on an existing foundation and the increase of seismic inertia have been previously analyzed, and for this bridge, it was analytically verified that these effects were less significant.

To numerically clarify the performance recovery of the hollow column, the natural frequencies of the bridge and vibration modes were monitored by both an artificial excitation test and microtremor measurement during the construction stage of concrete filling. A change in these properties may indicate a change in the stiffness of the hollow column.

The natural frequency estimated by the artificial excitation test was found to significantly change when the concrete filled the section with cracks penetrating the concrete wall. Moreover, a similar trend of the natural frequency change was found from the data obtained by microtremor measurement, which suggests that microtremor measurement might be a monitoring tool to examine the performance of a hollow bridge column repaired by filling concrete into the void.

Keywords: hollow bridge column, recovery, monitoring, vibration characteristics

1. Introduction

An intensive earthquake with a magnitude of 7.3 occurred in the Kumamoto area in April 2016. The earthquake severely damaged the Aso Choyo Ohashi bridge (four-span continuous rigid-frame prestressed-concrete box girder). Severe cracks were developed at the mid-height hollow section of the bridge column, and some of the cracks penetrated the concrete wall of the hollow section. Previous studies have shown that such cracks deteriorate the shear strength of the hollow column as well as the stiffness. To recover the shear strength of the column, high-fluidity concrete was filled into the void of the overall height of the hollow section. The effects of the concrete filling on an existing foundation and the increase of seismic inertia have been previously analyzed, and for this bridge, these effects were analytically verified to be less significant.

To numerically clarify the performance recovery of the hollow column, the natural frequency of the bridge and vibration modes were monitored by both an artificial excitation test and microtremor measurement during the concrete filling construction stage. A change in these properties may indicate a change in the stiffness of the hollow column.

As a result, the natural frequency estimated by the artificial excitation test was found to significantly change when the section with cracks penetrating the concrete wall was filled with concrete. Moreover, a similar trend of the natural frequency change was found from data obtained by microtremor measurement, which suggests that microtremor measurement might be a monitoring tool to examine the performance of the hollow bridge column repaired by filling concrete into the void.

This paper introduces the repair of hollow columns and monitoring-based examination of the performance recovery of the Aso Choyo Ohashi bridge.

2. Damage of Aso Choyo Ohashi bridge

The Aso Choyo Ohashi bridge is a four-span continuous prestressed-concrete (PC) box girder rigid-frame structure bridge designed based on 1980 specifications for highway bridges, as shown in Figure 1. Located at a deep V-shaped valley, the height of the P2 pier is 68 m, as shown in Figure 2.

With the 2016 Kumamoto earthquake, the slope around each abutment was deformed. Especially on the A1 abutment side, the slope failed, breaking bearings on A1, accompanied by the subsidence of A1 and a relative vertical displacement between A1 and the end of its box girder. On the A2 abutment side, even though a small level difference and relative transverse direction displacement occurred at the joint, the parapet and bearings were not damaged.

Regarding the piers, investigation was limited to observing the damage of the outside surface using UAV immediately after the earthquake because of the height of the piers and the damage of the access route to the piers. In the UAV investigation, cracks were observed on all piers. Large cracks were especially observed on the mid-height of P3 pier. With the rope access investigation from the outside surface, as shown in Figure 3, gap was observed at the horizontal crack on the mid-height of P3 pier. Although the gap is small, some cracks observed on the outside surface of P3 pier were suspected to penetrate the concrete wall of the hollow section. Because it is important to determine whether the cracks penetrate the concrete wall or not in order to consider how to restore the pier, the damage of the inside surface of the hollow section was investigated using a small camera, through a small hole formed on top of the solid section of the pier. As a result, cracks inside the surface corresponding to cracks outside the surface were observed, as shown in Figure 4, confirming that some cracks penetrated the concrete wall of the hollow section. Figure 5 shows whole cracks on P3 pier obtained from the investigation process described above. Although some cracks were observed at the bottom of the solid section, the extent of the damage was smaller than that of the mid-height of the hollow section.

Some cracks were observed on both the outside and inside surfaces, as well as on the PC box girders. However, based on the positional relationship between cracks observed on the outside and inside surfaces, and the crack depths, obtained using ultrasonography, it was not confirmed that these cracks penetrated the concrete wall of the PC box girder. At the support point of a girder, concrete was peeled off, exposing reinforcing steel as shown in Figure 6.

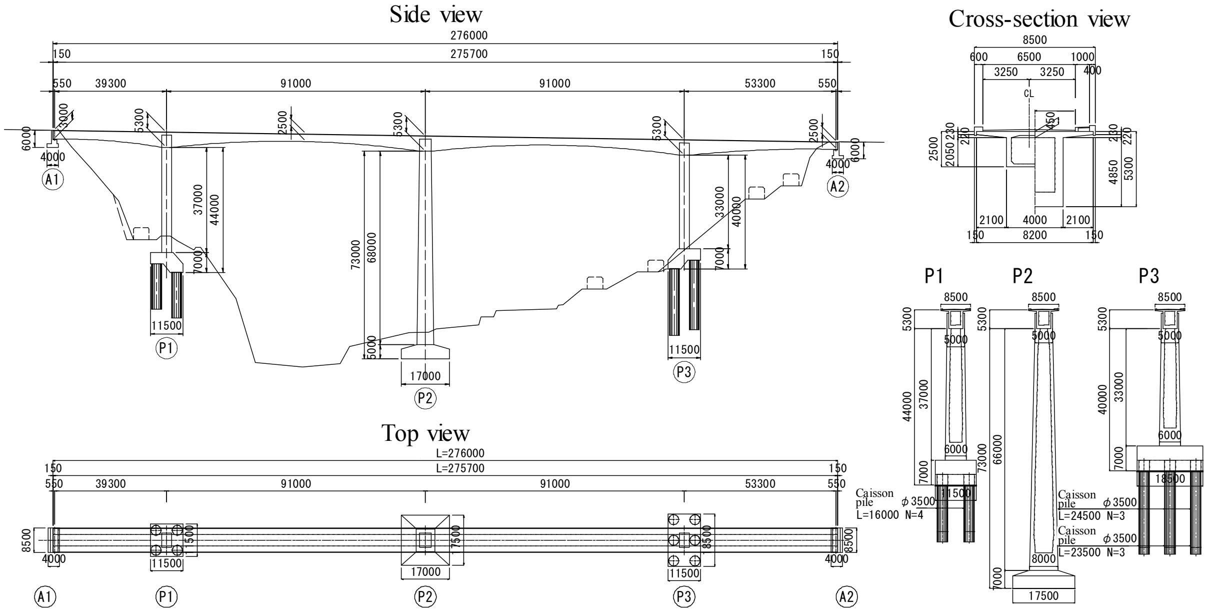

Fig. 1 – Schematic of the Aso Choyo Ohashi bridge (before the 2016 Kumamoto earthquake)

Fig. 2 – Slope failure and subsidence of the A1 abutment of the Aso Choyo Ohashi bridge

Fig. 3 – Rope access investigation

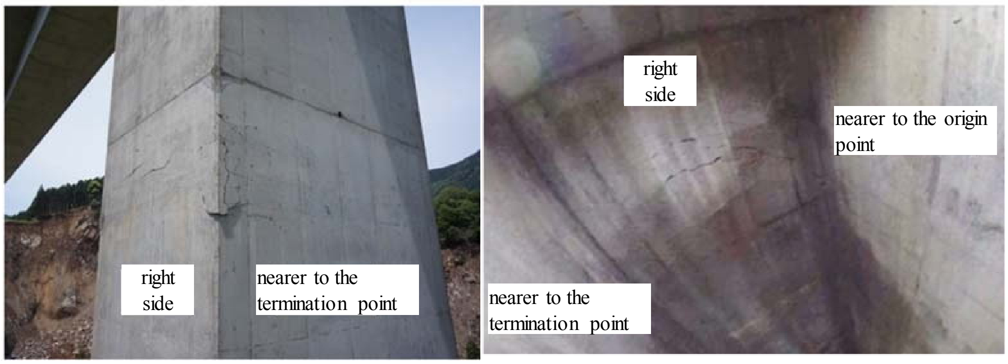

Fig. 4 – Cracks observed on P3 pier outside and inside surfaces

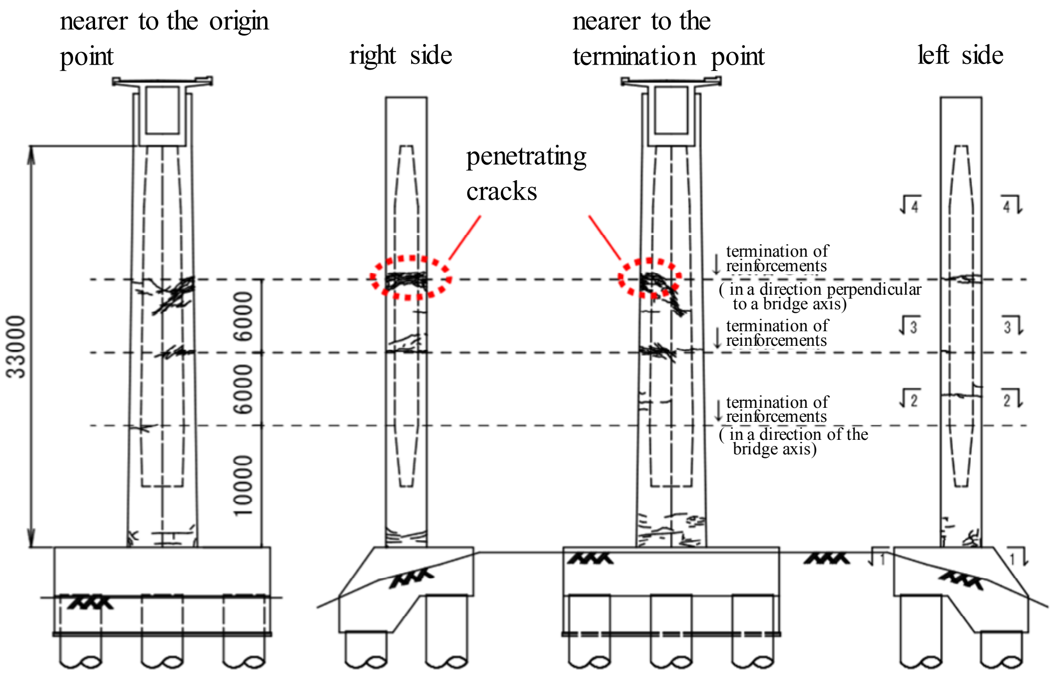

Fig. 5 – Penetrating cracks observed on P3 pier

Fig. 6 – Damage of the PC box girder end supported on the A2 abutment

3. Technical consideration for restoring the Aso Choyo Ohashi bridge

3.1 Technical consideration on restoring P3 pier

As mentioned above, P3 pier lost its function to resist shearing forces at the hollow section given some cracks in the concrete wall. Accordingly, the hollow section was filled with concrete to complement the resistance of the shearing force. However, this would increase the inertial force and dead load on the foundation. To reduce the dead load, one of the options may be to limit the area filled with concrete to the sections with penetrating cracks. However, this would lead to another problem during construction. As a result, the impact on the bridge was analyzed by increasing the dead load of P3 pier and the liquid pressure of the fresh concrete when placing the concrete, on the condition that the overall height of the hollow section was filled with concrete. As a result, the bridge performance can be ensured even if the overall height of the hollow section is filled with concrete. Based on this result, it was decided to fill the hollow section to the overall height with concrete.

To integrate the existing concrete and filling concrete, horizontal reinforcement bars were arranged and post-installed anchors were attached inside the surface at the section with penetrating cracks, as shown in Figures 7 and 8. Anchors were attached at 450 mm intervals in the height direction, as shown in Figure 8, to prevent cutting of the reinforcing bar inside the existing concrete and significant reduction of rigidity when the inner surface of concrete wall is being drilled to attach anchors.

When filling concrete into the hollow section, setting a hole for concrete pouring and conveying pipe, as shown in Figure 9, was necessary. Regarding the hole for concrete pouring, an 80 cm × 100 cm hole was formed on top of the solid section of the pier for the workability of the filling concrete. To minimize the impact on the existing concrete, the following considerations were adopted. First, the location of the hole for concrete pouring was selected; PC steel was avoided, minimizing the impact on the structure strength. Second, the concrete around the hole for concrete pouring was reinforced with carbon fiber sheets. In addition, an inspection hole (void pipe) with a diameter of 20 cm was constructed near the section with penetrating cracks in order to investigate the inside surface after future earthquakes by inserting a small camera. When constructing the inspection hole, the void pipe was reinforced by inserting a vinyl pipe into the void pipe to prevent collapse by the pressure of the fresh concrete. Furthermore, high-fluidity concrete was selected as the filling concrete and was added from the top of the pier into a hollow section because of the difficulty of the concrete compaction due to the tight space. Considering the liquid pressure of the fresh concrete from the inside surface and workability of the inspection hole, the height of filling concrete per lift was 8.5 m at the first lift, and 1.2 m after the first lift, minimizing the impact on the inspection hole (void pipe).

Fig. 7 – Arrangement of bars inside the pier

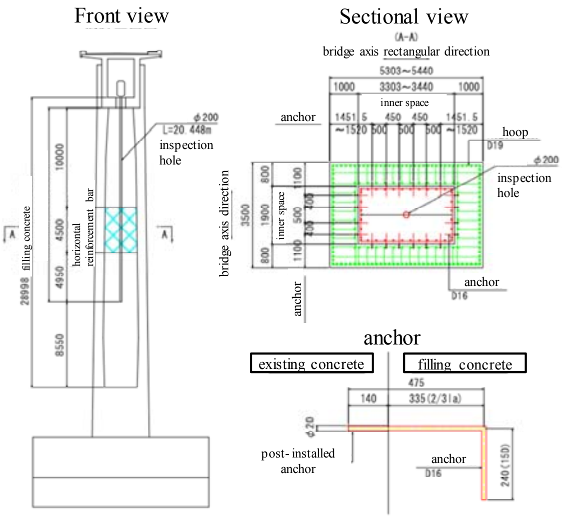

Fig. 8 – Consideration for the integrity of the filling concrete

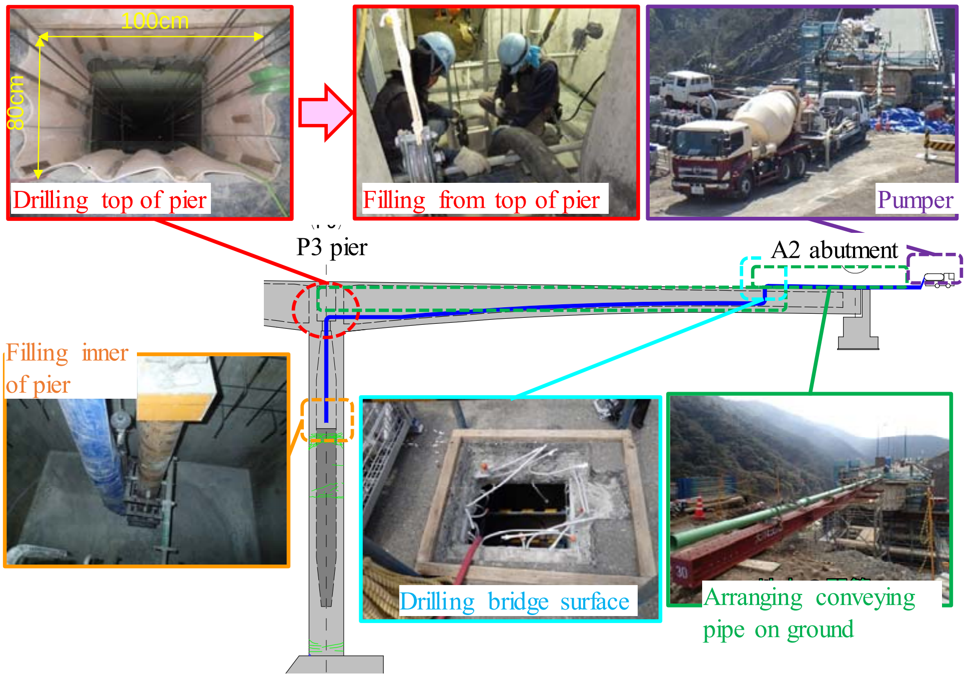

Fig. 9 – Filling concrete into the hollow section of P3 pier

3.2 Technical consideration of restoring other members

The A1 abutment was studied to reconstruct at the location with set back from the original position where the slope failure caused a significant subsidence of A1. However, if the A1 location changes to behind as it was before the earthquake, the length of the PC box girders should extend, that may have issues of structural details of connection between an existing girder end and an additional girder. Because there are few nuggets of such connection details, another strategy was discussed in the design process.

To mitigate the risk of slope deformation, the location of support layer for A1 foundation was reconsidered. At the site with cracks, a geological boring survey was conducted on the ground immediately after the earthquake to three-dimensionally understand the loosening of the ground. Survey points were set in a grid-shape, focusing on the points near the A1 abutment. As a result of the boring survey, some cracks and ground loosening were observed on the original support layer of the design at the time of construction. As a result, the loosening of the ground was eliminated. A new support layer was visually identified, confirming the actual geological condition of the ground on the site.

In addition, to ensure the traffic function of the bridge in case the ground at the front of the A1 abutment fails due to slope deformation around A1, an RC rigid-frame structure was adopted for the new A1 abutment, resulting in extremely high rigidity and integrity. To provide structural redundancy, the back-end of the RC rigid-frame structure was extended over the site in which ground loosening was observed. Additionally, to avoid weakening the joint of the upper floor slab and pier wall, a haunch was set up at the corner to improve rigidity. The size of the haunch was as thick as the pier wall considering the corner may be a plastic hinge. Furthermore, regarding the back side of A1 abutment, the road linearity was reconsidered in the direction apart from the slope to ensure traffic function of the road in case the slope around A1 collapses.

As a result of these considerations, the A1 abutment was restored, adopting a five-span continuous curved RC rigid-frame structure (total length 64.5 m), as shown in Figure 10. To mitigate the risk of the slope failure around A1 abutment, the slope was also stabilized, with techniques such as anchoring, reinforcement steel insertion, and spraying for slope protection.

Regarding cracks on the PC box girders, carbon fiber sheets were added to supplement the decreased loading capacity after pouring and section repair on the cracks. To inspect the progress of cracks after restoration, carbon fiber sheets were adhered only to the outside surface and not to the inside surface, for easier access during inspection. In addition, epoxy resin was used as an intermediate coating material to finish the outside surface of the carbon fibers sheets, followed by acrylic resin.

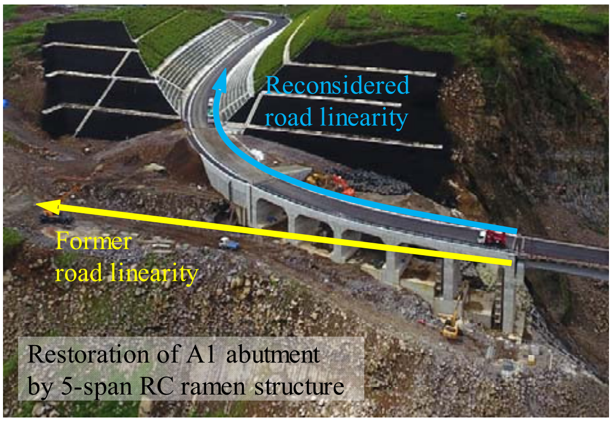

Fig. 10 – Full view of the restoration of the A1 abutment

4. Clarification of performance recovery by monitoring during the construction stage of concrete filling

4.1 Purpose of monitoring in the Aso Choyo Ohashi bridge restoration

When restoring a bridge with special damages like the Aso Choyo Ohashi bridge, technical considerations and uncertainties often arise, unlike when designing or constructing new bridges. Accordingly, the performance recovery is clarified to complement the uncertainty of design or the restoration construction is realized by monitoring the vibration characteristics of the bridge to confirm the validity of the change of vibration characteristics, utilizing the characteristic that structural system changes as the construction advances. In this paper, vibration characteristics are obtained by an artificial excitation test, microtremor measurement, and characteristic value analysis. Comparing the results of the analyses, the ability of microtremor measurement to examine the performance of the hollow bridge column repaired by filling concrete into the void is verified.

Furthermore, in the literature, there are some case examples that confirm the vibration characteristics of concrete before and after damage occurrence or restoration. Kato et al. confirmed the decline of natural frequency due to the advancement of damages; a PC girder of π-shaped rigid-frame bridge with a maximum span of 24 m was subjected to vertical loading until the PC cable reached plastic region. Tanaka et al. and Sho et al. confirmed the decline of natural frequency due to the advancement of damages; RC rigid-frame bridges with a maximum span of 6–8 m were artificially damaged with the aim to demolish. In addition, Seki et al. confirmed the decline of the natural frequency of an RC rigid-frame bridge with a maximum span of 6 m after damage by earthquake. Although there are some case examples that utilize vibration characteristics to confirm changes in the bridge condition, most are focused on bridges with a maximum span of less than 30 m. This case regarding the Aso Choyo Ohashi bridge is distinctive as it involves a long-span PC rigid-frame bridge.

4.2 Strategy of monitoring

As mentioned above, the section stiffness of the P3 hollow column was assumed to be less than that before the earthquake due to cracks penetrating the concrete wall. Accordingly, to confirm the change in stiffness due to the concrete filling (Figure. 11), the changes in the natural frequency and vibration mode by filling concrete were measured using accelerometers.

Figure 12 illustrates representative construction steps of the restoration process. The vibration mode of the whole bridge changes from construction step 2 to construction step 3 as the support condition changes. After construction step 2, accelerometers were set to the whole bridge to measure not only the vibration mode of P3 pier but also the vibration mode of the whole bridge, as depicted in Figure 13. Especially near P3 pier, accelerometers were set densely because the change in vibration characteristic by concrete filling was thought to be relatively large. In addition, four accelerometers (two for transverse direction, two for vertical direction) were set on four cross sections to measure the torsional behavior of box girders. Figure 13 shows the location of accelerometers. Furthermore, the specifications of the devices are also presented in Figure 13; they helped in accurate measurements even for extremely small vibrations.

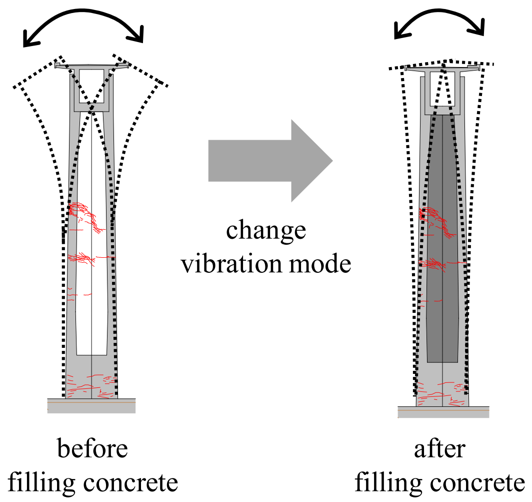

Fig. 11 – Change in vibration mode assumed for P3 pier

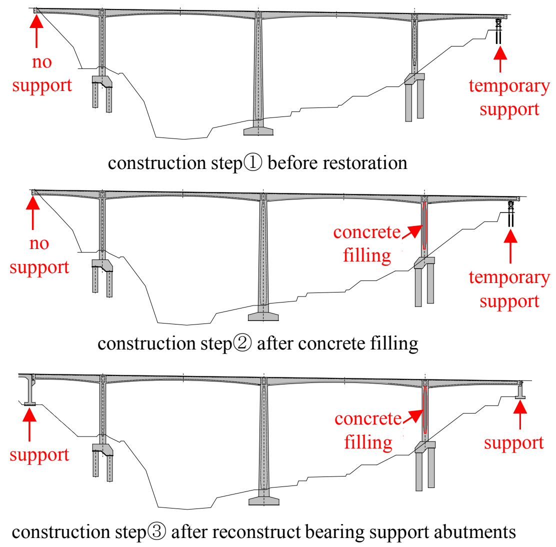

Fig. 12 – Representative construction steps

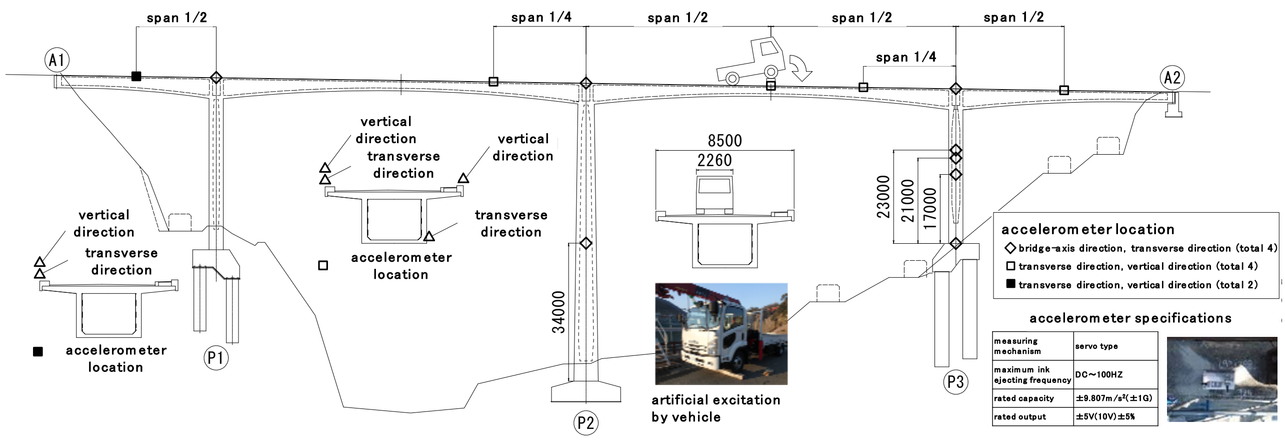

Fig. 13 – Location of accelerometers

4.3 Data measurement methods

4.3.1 Microtremor measurement

Microtremor was measured for 20 min. The sampling frequency was 256 Hz, three significant figures in order to ensure the accuracy of the measured natural frequency. For measured acceleration waveform, the DC-component was eliminated every 20 waves and a window-process was conducted. Afterward, the natural frequency was obtained by fast Fourier transform and moving average at 0.01 Hz width.

4.3.2 Artificial excitation test

As shown in Figure 13, excitation was conducted by dropping a construction vehicle from a 15 cm square timber. The excitation point is also shown in Figure 13. The weight of the construction vehicle was 3530 kg. After measuring the acceleration waveform, the natural frequency was obtained using the same procedure as described above.

4.4 Characteristic value analysis

Characteristic value analysis was conducted to compare the result with the measured vibration. Figure 14 illustrates the model. Main girders and piers were modeled using a fiber element. Each element was divided at the termination of reinforcements into a length less than the side length of the section. The boundary conditions for the respective support points are presented in Table 1. Moving steel bearing supports on A1 and A2 were modeled as "fix," because it was considered that modeling as "fix" could simulate actual behavior, because the external forces on the moving steel bearing on A1 and A2 were extremely small. Cracks were not modeled.

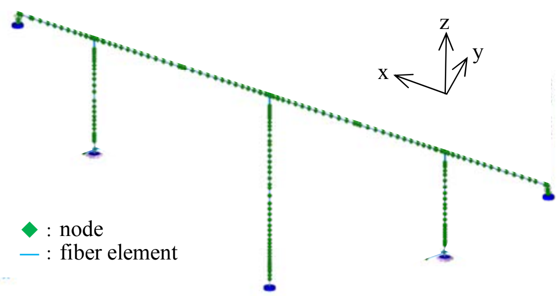

Fig. 14 – Model for characteristic value analysis

Table. 1 – Boundary condition for each support point

Construction step	Support point	Structural condition	Boundary condition					
			x	y	z	Tx	Ty	Tz
①	A1	no support	free	free	free	free	free	free
	A2	temporary support	free	free	fix	free	free	free
②	A1	no support	free	free	free	free	free	free
	A2	temporary support	free	free	fix	free	free	free
③	A1	support	fix	fix	fix	fix	free	fix
	A2	support	fix	fix	fix	fix	free	fix

4.5 Identification of natural frequency by microtremor

Figure 15 compares the natural frequency obtained from microtremor measurement with that obtained from artificial excitation test. Both results correspond well with each other. Figure 16 compares the natural frequency obtained from microtremor measurement with that obtained from characteristic value analysis. The ratio of natural frequency obtained from microtremor measurement to that obtained from characteristic value analysis is 91%–118%, except for the vibration mode that the temporary support point significantly moves (third vibration mode). It was confirmed that the natural frequency could be identified by microtremor measurement for this bridge.

4.6 Change in vibration characteristics of the bridge and performance recovery

Figure 17 shows the change in natural frequency due to the concrete filling. The figure focuses on the vibration in the transverse direction because penetrating cracks were observed on surfaces parallel to the bridge axial direction. As for the secondary vibration mode; P3 pier mainly vibrates, a change in natural frequency was observed due to the concrete filling. The natural frequency is proportional to the square root of the rigidness of the pier and is inversely proportional to the square root of the pier mass as represented in Eq. (1). As indicated in Eq. (1), the increase of the natural frequency of a hollow column represents the increase of the column stiffness since the column mass is increasing by concrete filling into a void section.

$$\omega = \sqrt{K/M} \tag{1}$$

ω: natural frequency, K: spring constant (rigidness), M:mass

Specifically, a significant change to a larger value can be observed after the concrete was filled into the hollow section with penetrating cracks. On the other hand, for the primary vibration mode, no change in natural frequency was observed; only pier P1 mainly vibrates, as shown in Figure 17. This result proves that the change in the rigidness of P3 pier does not induce natural frequency change as for the primary vibration modes; pier P1 mainly vibrates because no concrete was filled into the pier. Furthermore, a similar trend of rigidness change was observed both by dropping the front wheel of a construction vehicle and by microtremor measurement, even if the values of natural frequency were a little different from each other. From the above results, the performance recovery can be clarified by a simple monitoring method, focusing on the structural system change of the whole bridge during the restoration process.

Figure 18 shows the distributions of horizontal displacement in the height direction of P3 pier before and after filling concrete. The displacement was induced by dropping the front wheel of a construction vehicle. The horizontal axis represents the dimensionless ratio of the horizontal displacement induced at each height of P3 pier to the maximum horizontal displacement measured at the road surface, induced by dropping the front wheel of a construction vehicle before filling with concrete. The horizontal displacements induced at specific heights of P3 pier were calculated from the double integral of the horizontal accelerogram measured. From this result, it can be observed that vibration mode starts at the mid-height of P3 pier and the vibration does not induce horizontal displacement at heights lower than the height before concrete filling. It is considered that sectional force was not transmitted to the lower section at the mid-height of P3 pier due to the loss of shear capacity caused by penetrating cracks at the height. On the other hand, it can be observed that the vibration mode changed to start at the base of P3 pier and to vibrate over the overall height of P3 pier after filling with concrete. A jump of horizontal displacement observed before filling concrete is not observed at the section with penetrating cracks. These results clarify that the concrete filling share shear forces and transmit horizontal forces to the lower section. Thus, the change in the vibration mode of P3 pier by filling concrete is confirmed via the artificial excitation test.

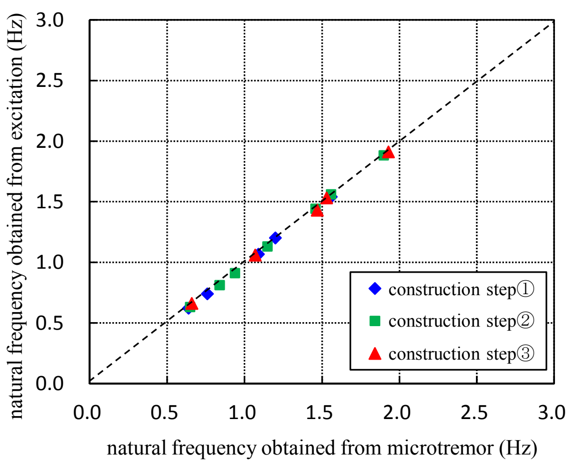

Fig. 15 – Comparison between the natural frequencies obtained from microtremor measurement and from artificial excitation test

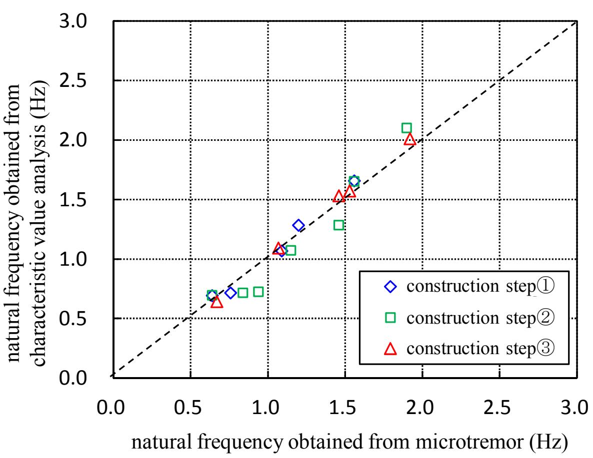

Fig. 16 – Comparison between the natural frequencies obtained from microtremor measurement and from characteristic value analysis

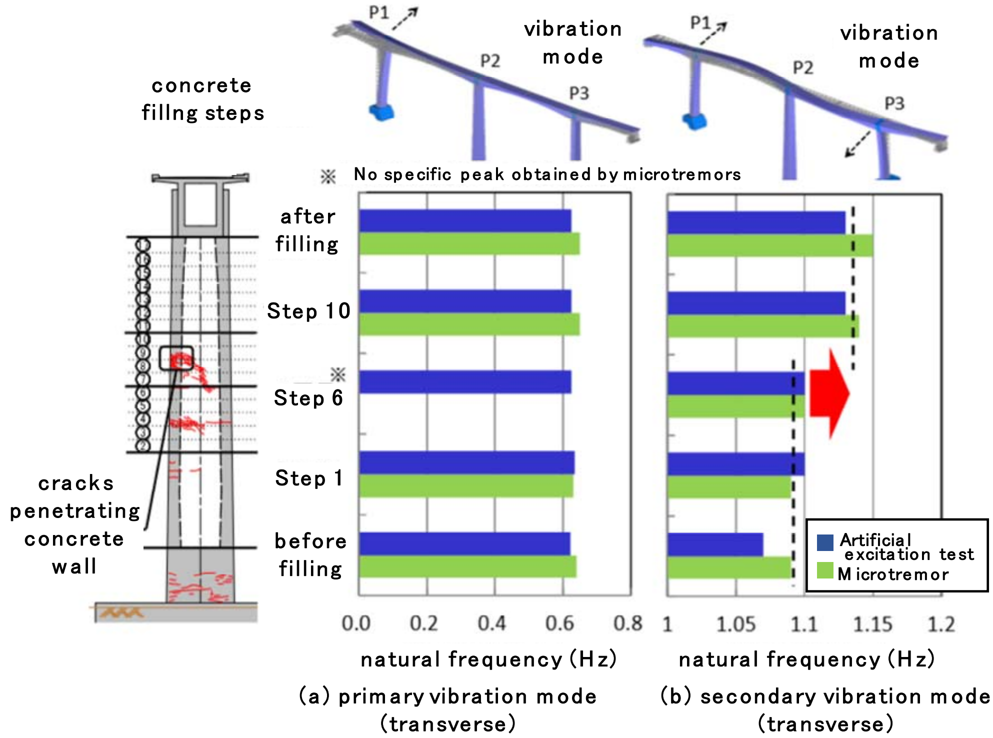

Fig. 17 – Change in natural frequency by filling with concrete

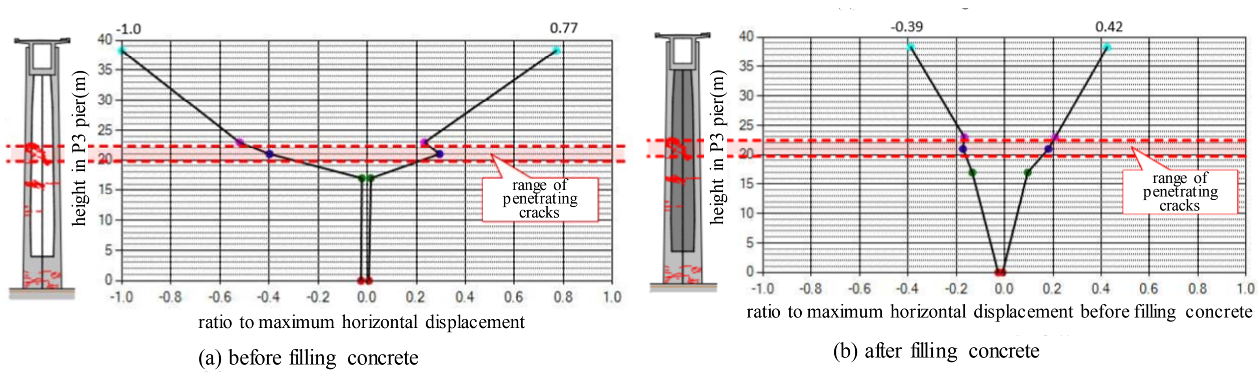

Fig. 18 – Change in vibration mode by filling with concrete

5. Conclusions

In this paper, focusing on the Aso Choyo Ohashi bridge (four-span continuous rigid-frame PC box girder), the effectiveness of microtremor measurement as a monitoring tool to examine the performance of a hollow bridge column was confirmed. Vibration was measured by microtremor and excitation tests, and the results were compared with those of characteristic value analysis conducted through the construction steps. The conclusions of this paper are as follows:

The natural frequency observed after filling concrete into the hollow section with penetrating cracks were found to be significantly greater than that before the concrete filling, determined by microtremor as well as by excitation measurements. Microtremor measurement was confirmed to be effective for examining the performance of the repaired hollow bridge column.

In addition, by excitation test, the vibration mode was observed to start at the base of P3 pier and to vibrate over the overall height of P3 pier after filling with concrete. This result clarifies that the filled concrete share shear forces and transmit horizontal forces to the lower section.

Utilizing the above information, if the vibration data of a bridge before damage (e.g., after construction, after repair) are available, the performance of the bridge when damaged can be easily and rapidly examined by conducting the same vibration measurement. In addition, the reliability of bridge performance examination may be improved by integrating microtremor and excitation measurements.

6. References

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