# DESIGN FOR WIDENING OF EXISTING BRIDGE PIERS AFFECTED BY ALKALI-SILICA REACTION (ASR)

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**Abstract**

As a part of a project to add extension girders to an expressway bridge, it was planned to widen existing T-shaped concrete pier beams and install external tendons to support the new girders with an enhanced flexural capacity. However, numerous cracks were found in the surface of the beams after removal of protective coatings, suggesting a possibility of deterioration by alkali silica reaction (ASR). Detailed investigations demonstrated that the concrete was actually damaged by ASR. Since the beams were to be prestressed by the external tendons over the whole length, it was necessary to check the soundness of concrete inside the beams. Ultrasonic tomography was applied to evaluate the properties of the inner concrete. This paper reports the detailed investigations and strengthening designs for the ASR-affected piers.

Keywords: widening, visual inspection, external tendon, ultrasonic tomography

## 1 INTRODUCTION

The Hanshin Expressway, which was originally a 2.3 km section from Tosabori to Minatomachi in Osaka opened to traffic in June 1964, now forms an important urban expressway network of a total length of 259.1 km in the Kyoto-Osaka-Kobe region. Deterioration by alkali silica reaction (ASR) was first found in reinforced concrete (RC) piers in 1982, not long after being put in service. Immediate actions were taken by 1984 to develop quality standards for aggregates and implement various measures, including proper selection of aggregates, in new construction projects to control ASR. The first guidelines for systematic maintenance practice on ASR-affected structures were established in 1984. Preventive measures such as surface coatings for deteriorated structures were taken to mitigate ASR, and follow-up inspections have been carried out continuously.

The protective coatings are applied to the surfaces of RC structures for the purpose of preventing penetration of water which is the major cause of ASR. Cracks occurring in them are easily visible, but those occurring beneath them cannot be readily detected.

With the recent decrease in the number of new construction projects since 2000, many rehabilitation projects have been planned and conducted on existing structures to connect existing routes and enhance the network of the urban expressways.

The project in this report was planned to connect two existing expressway routes by widening the superstructure, and the beams of T-shaped RC piers were to be extended by additional concrete casting. This paper reports the investigations carried out in relation to ASR and strengthening designs selected for the existing T-shaped RC piers to be rehabilitated.

## 2 GENERAL PLAN

The plan to widen the bridge superstructure or construct extension girders required additional concrete casting on the existing pier beams as shown in Figure 1. Since the stress in the main reinforcement of the beams was estimated to exceed the allowable value under the load conditions upon completion, it was planned to strengthen the beams with external tendons for an increased flexural capacity.

## 3 INVESTIGATIONS

### 3. 1 Background

A total of 17 piers were included in the the general plan, and 10 of them which were considered to have been affected by ASR were repaired by surface protection in 1980s. The widening

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project was contracted as planned, and field survey was carried out in which readily visible large cracks attributable to ASR were found in the protective coatings. The 17 piers were categorized into some groups based on the shape and appearance of the cracks observed in the protective coatings, and Piers I to III were selected as representative piers to be investigated in detail.

### 3. 2 Piers investigated

*Structure types and years of construction*

Piers I, II and III selected for the investigations were T-shaped RC piers as shown in Figure 2. They were built in a similar period: Pier I was built in 1970, and Piers II and III were built in 1972.

The in-house criteria of Hanshin Expressway define ASR-affected RC piers as those having the presence of alkali silica gel in the beam and a total length of cracks wider than 0.3 mm exceeding 100 m [1]. According to the in-house standard, none of these three piers was determined to be an ASR-affected pier, with the total length of the cracks found in the protective coatings falling within the criterion.

*Repair and inspection histories*

Table 1 shows the repair history of the piers. Repairs were made in 1996 on Pier I, in 1985 and 1996 on Pier II, and in 1980, 1985 and 1996 on Pier III. Same techniques were used in each repair: crack injection and surface protection. All structures of the Hanshin Expressway are visually inspected periodically every five years in accordance with the in-house inspection manual [2]. The three piers in focus have undergone periodic visual inspections three times, in 2001, 2006 and 2011, since the latest repair in 1996, being determined to be unaffected by ASR based on the surface observation from above the protective coatings.

### 3. 3 Detailed ASR investigations

*External inspection*

The existing protective coatings applied to the beams in 1996 were removed before the placing of additional concrete as described in the general plan. After the removal of surface coatings, many cracks were found, which could not be detected by the periodic visual inspections. Figures 3 to 5 and Table 2 show the results of visual inspections of cracks before and after the removal of surface coatings. The total length of cracks by the visual inspection before the removal of surface coatings was 8.1 m in Pier I, 13.4 m in Pier II, and 12.9 m in Pier III, showing no significant differences (Table 2). However, the total crack length after the surface coating removal was 91.7 m in Pier I, 134.5 m in Pier II, and 298.1 m in Pier III, revealing significant development of cracks beneath the coatings. The total crack length exceeding 100 m in Piers II and III suggested the possibility of ASR deterioration according to the criteria of the Hanshin Expressway.

*Tests using concrete cores*

The external inspection revealed that the total length of cracks wider than 0.3-mm exceeded 100 m in Piers II and III. In order to determine the degree of deterioration of concrete, concrete core specimens were taken from the piers, and various tests were carried out to determine compressive strength, static elastic modulus, total expansion and presence of alkali silica gel. Core specimens taken from Pier I were also tested for comparison. Table 3 shows the test results.

*Compressive strength*

It is known from previous researches that deterioration by ASR affects compressive strength of concrete. Pier I exhibited a compressive strength of 38.4 N/mm$^2$ which was well above the design strength, $\sigma_{ck}$ = 27 N/mm$^2$. Compressive strength of Pier II was 25.5 N/mm$^2$, which was 94% of the design strength. Pier III which had the highest crack density, 5.79 m/m$^2$, exhibited the lowest compressive strength, 14.6 N/mm$^2$, which was only 54% of the design strength.

*Static elastic modulus*

It is a common understanding that ASR-affected concrete generally exhibits a more significant reduction in static elastic modulus than in compressive strength. Pier I had a static elastic modulus of 33.6 kN/mm$^2$ which was well above the design level, $E$ = 26.5 kN/mm$^2$. Pier II showed a value of 23.3 kN/mm$^2$, which was 88% of the design level. The value of Pier III was the lowest, 7.7 kN/mm$^2$, which was only 29% of the design level. Although these tendencies were similar to those in compressive strength, the reduction in static elastic modulus was more significant than that in compressive strength. This implies that damage could be more severe than suggested by compressive strength.

*Total expansion and alkali silica gel*

Accelerated test of residual expansion capacity (the JCI-DD2 method) was carried out using the cores to estimate future expansion of concrete. Total expansion was found to be 0.023% in Pier I, 0.023% in Pier II, and 0.027% in Pier III, being less than 0.1% which was the criterion for the risk of harmful effects in future. Alkali silica gel was not observed in the cores of Pier I, while detected in those of Piers II and III.

*Discussion*

Pier I which was used as the control for comparison was determined to be sound by the core tests, with the values of compressive strength, static elastic modulus and total expansion satisfying the design levels. Piers II and III which showed reductions in compressive strength and static elastic modulus in addition to the presence of alkali silica gel were determined to be affected by ASR. Total residual expansion was estimated to be less than 0.1% in all three piers, suggesting low risk of harmful effects in future. A comparison between Piers II and III suggested more significant ASR deterioration in Pier III than in Pier II, with larger decreases in compressive strength and static elastic modulus as well as larger total expansion as compared to Pier II.

### 3. 4 Internal concrete inspection

*Necessity of internal concrete inspection*

The external inspection can determine damage in the surface region only. The concrete core tests can provide local measurements of compressive strength and static elastic modulus at the measured points but no determination about the damage across the entire cross-section of a beam. The girder widening project in this report required determination of the degree and extent of deterioration in the internal concrete of the beams because external tendons were to be used to provide prestress to the beam concrete.

*Internal inspection by ultrasonic tomography*

It was found in a previous study [3] that there was a certain level of correlation between the velocity of an ultrasonic wave propagated in concrete and compressive strength or static elastic modulus. That is, the higher the propagation velocity in concrete, the larger the compressive strength or static elastic modulus is, suggesting soundness of the concrete.

In order to determine the degree and extent of deterioration of the concrete across the cross-section of the beams, internal concrete inspection using ultrasonic tomography was carried out.

Figure 6 shows a schematic of ultrasonic tomography, and Figure 7 shows a view of the inspection using the technique. The transmitter attached to one end of an object emits ultrasonic waves, and the receiver attached to the other end of the same object detects the propagated waves. The time to reach the receiver and the distance between the transmitter and the receiver are measured, and propagation velocity is calculated from them. The object is divided into some blocks, and the mean of the obtained ultrasonic wave propagation velocities along the wave lines passing through each block is calculated as a representative ultrasonic wave propagation velocity for the block.

Ultrasonic wave propagation velocity was 4100 to 4400 m/s in Pier I, 3900 to 3600 m/s in Pier II, and 3300 to 3600 m/s in Pier III. Many cases have been reported in which ultrasonic wave propagation velocity was mostly in a range from 4000 to 4500 m/s in sound concrete and decreased to about 70% in ASR-affected concrete, with some variations depending on the quality of concrete [4]. Decrease in propagation velocity due to ASR was found in Pier III.

Frequency analysis was made on the propagation velocity for frequency-based tomographic analysis. This analysis takes advantage of the fact that a prominent frequency appears at the resonance frequency of a probe in the waveforms in case of propagation in sound concrete but shifts to the lower frequency band in case of propagation in deteriorated concrete.

The probes used in this study had a resonance frequency of 40 kHz. If the concrete is sound, a prominent frequency should appear around 40 kHz. If a pier has significant cracks due to ASR deterioration, frequency levels should be low for the inside of the beam. Figure 8 shows the tomographic analysis results for the concrete inside the beams of Piers I to III. Pier I (left in Figure 8) was found to be sound, showing frequency levels of 40 kHz or above for the entire cross-section of the beam. Pier II (middle in Figure 8) was found to be deteriorated near the top end of the cross section of the beam but to be mostly sound, showing frequency levels of 40 kHz or above for most of the inside of the beam. On the other hand, Pier III (right in Figure 8) was found to have advanced deterioration in the concrete in the top half of the cross-section of the beam.

# 4 STRENGTHENING DESIGNS

Pier I was not classified as an ASR-affected pier and showed good results in the core tests and internal concrete inspection. Therefore, construction was carried out as originally planned.

Detailed examination was carried out on Pier II using the values of compressive strength and static elastic modulus obtained by the concrete core tests (Table 3). The results showed that the originally planned design could satisfy the allowable stress limits. Pier II was classified as an ASR-affected RC pier due to the presence of alkali silica gel and a total length of cracks wider than 0.3-mm exceeding 100 m. However, ultrasonic wave velocity and tomographic analysis shown in Table 4 allowed to determine that internal concrete deterioration by ASR was not advanced in the pier. Consequently, strengthening design for Pier II was finalized by taking into account the facts that the pier should be classified as an ASR-affected pier according to the criteria of the Hanshin Expressway, and that external tendons were to be used for introducing prestress to the concrete beam. In addition to the general plan shown in Figure 1, the existing concrete in the bearing region at the end of the beam was replaced with new concrete to ensure transmission of the applied prestressing force to the concrete cross-section of the beam. It was also decided to apply transparent protective coatings to the beam for easier external inspection and detection of further deterioration by ASR, if any, after the strengthening. Figure 9 shows a schematic of the strengthening design for Pier II.

Detailed examination was also carried out on Pier III using the values of compressive strength and static elastic modulus obtained by the core tests (Table 3). The results showed that the stress due to the external prestressing force would not meet the allowable level for the concrete calculated from the measured compressive strength. The concrete inside the beam was considered to have advanced deterioration by ASR as shown in Table 4, suggesting that it would be difficult to strengthen the beam. Consequently, it was decided to remove the existing beam and construct a new one.

# 5 MONITORING

## 5. 1 Necessity of monitoring

The strengthening design for Pier II is very unique in that prestressing force is to be applied to an ASR-affected beam by using external tendons. Long-term monitoring should be carried out on this unprecedented case in terms of maintenance, in addition to monitoring during prestressing. The following sections describe the monitoring during prestressing and long-term measurement.

*Monitoring during prestressing*

Longitudinal prestressing force was expected to cause compression in the beam in the same direction, forcing the beam to expand in the vertical direction. This tendency could be significant with the decrease in static elastic modulus due to ASR. It was possible that such behaviors would result in the loss of prestressing force or development of cracks occurring in the longitudinal direction during the prestressing process. In order to ensure safety during prestressing, a tension meter, strain gauges and contact strain gauges were installed as shown in Figure 10, and tensile force of the external tendons, reinforcing steel stress and crack width were measured.

The measurement results demonstrated that there was no decrease in tensile force due to the loss of prestress or no development of cracks, and that reinforcing steel stress was below the design value.

*Long-term measurement*

Continuous follow-up inspection is necessary to monitor long-term stress release in the external tendons and behaviors of the beam. Items to be measured are tensile force in the external tendons, reinforcing steel stress, change in the length of the beam and crack behaviors. The pier will continue to be placed under such measurement for proper maintenance in future. Figure 10 shows the measurement instruments installed on Pier II, and Table 5 shows the items, instruments and purposes of the measurement.

# 6 CONCLUSIONS

- Existing piers were inspected after removing the protective coatings to examine cumulative cracks since the time of construction. Significant development of cracks were found to have occurred beneath the coatings.

- Soundness of concrete inside the beams was investigated by using ultrasonic tomography. The analytical results were mostly consistent with the external inspection and core tests (unconfined compressive strength, static elastic modulus, residual expansion and presence of alkali silica gel).
- The use of ultrasonic tomography for analysis allowed for a nondestructive determination of internal concrete deterioration, without damaging existing structures.
- The external tendon system was found applicable to strengthening of ASR-affected concrete beams as long as the internal concrete was sound. The pier strengthened in this project will be placed under continuous monitoring for proper maintenance in future.

## 7 REFERENCES


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[3] Kuzume, K, Manabe, H, Ogura, N, Yamamoto, T, and Miyagawa, T (2013): A study for determination of internal properties of ASR-affected concrete using ultrasonic tomography. Proceedings of the Concrete Structure Scenarios, JSMS, Vol. 13. (in Japanese).

[4] Chubu Cement and Concrete Researchers (1990): Alkali-silica Aggregate Reaction in Concrete Structures. (in Japanese).


TABLE 1: Repair history.

| Year | Pier I | Pier II | Pier III |
|---|---|---|---|
| 1970 | Built as a T-shaped RC pier. | -- | -- |
| 1972 | -- | Built as a T-shaped RC pier. | Built as a T-shaped RC pier. |
| 1980 | -- | -- | Repaired by crack injection and surface protection. |
| 1985 | -- | Repaired by crack injection and surface protection. | Repaired by crack injection and surface protection. |
| 1996 | Repaired by crack injection and surface protection. | Repaired by crack injection and surface protection. | Repaired by crack injection and surface protection. |

TABLE 2: Crack inspection results.

| | | Pier I | Pier II | Pier III |
|---|---|---|---|---|
| Total crack length (m) | Before | 8.1 | 13.4 | 12.9 |
| | Surface coating removal | ↓ | ↓ | ↓ |
| | After | 91.7 | 134.5 | 298.1 |
| Maximum crack width (mm) | Before | 0.1 | 0.2 | 2.0 |
| | Surface coating removal | ↓ | ↓ | ↓ |
| | After | 0.5 | 1.0 | 10.0 |

TABLE 3: Detailed investigation results.

| Evaluation items | | Design or threshold value | Unit | Pier I | Pier II | Pier III |
|---|---|---|---|---|---|---|
| Mechanical properties | Compressive strength | 27 | $N/mm^2$ | 38.4 | 25.5 | 14.6 |
| | Static elastic modulus | 26.5 | $kN/mm^2$ | 33.6 | 23.3 | 7.7 |
| Total expansion | JCI-DD2 | 0.1 | % | 0.023 | 0.023 | 0.027 |
| Alkali silica gel | Present/absent | -- | -- | Absent | Present | Present |
| Cracks | Crack density | -- | $m/m^2$ | 1.64 | 2.14 | 5.79 |
| | Maximum crack width | -- | mm | 0.5 | 1.0 | 10.0 |

TABLE 4: Ultrasonic test results.

| | Threshold | Unit | Pier I | Pier II | Pier III |
|---|---|---|---|---|---|
| Ultrasonic wave velocity | 4000 | m/s | 4500～4600 | 4100～4400 | 3300～3600 |
| Tomographic analysis | 40 | kHz | Sound for the entire cross-section. | Deteriorated only in the top end of the beam. | Deteriorated across the cross-section of the beam. |

TABLE 5: Measurement items.

| Measurement items | Measurement instruments | Purposes |
|---|---|---|
| Tensile force in the external tendons | Tension meter | Measurement of change in tensile force of the external tendons with time |
| Reinforcing steel stress | Strain gauges | Measurement of main reinforcement stress with time |
| Dimensions of the beam | Contact strain gauge | Measurement of change in the length of the beam with time |
| Crack width | Contact strain gauge | Measurement of change in crack width with time |

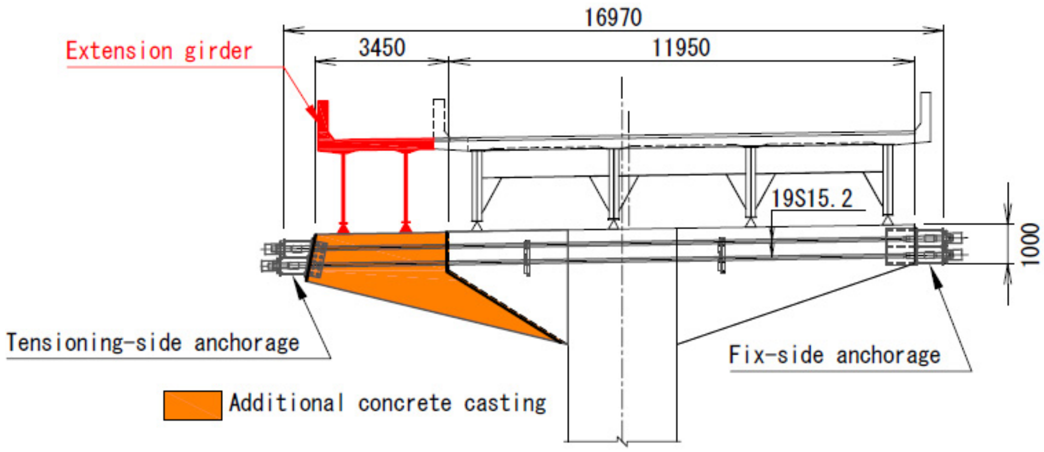


FIGURE 1: General plan

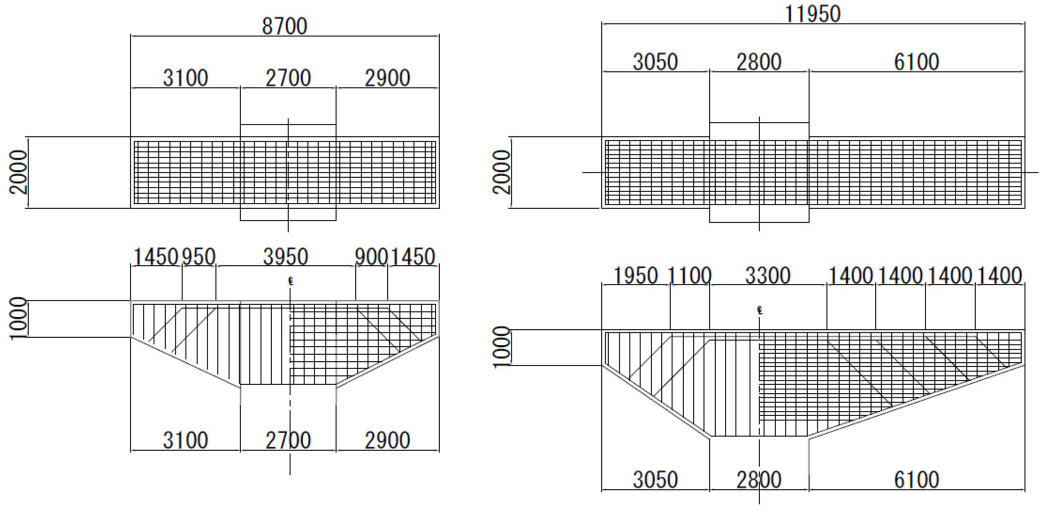


FIGURE 2: Structural drawings of Pier I (left) and Piers II and III (right)

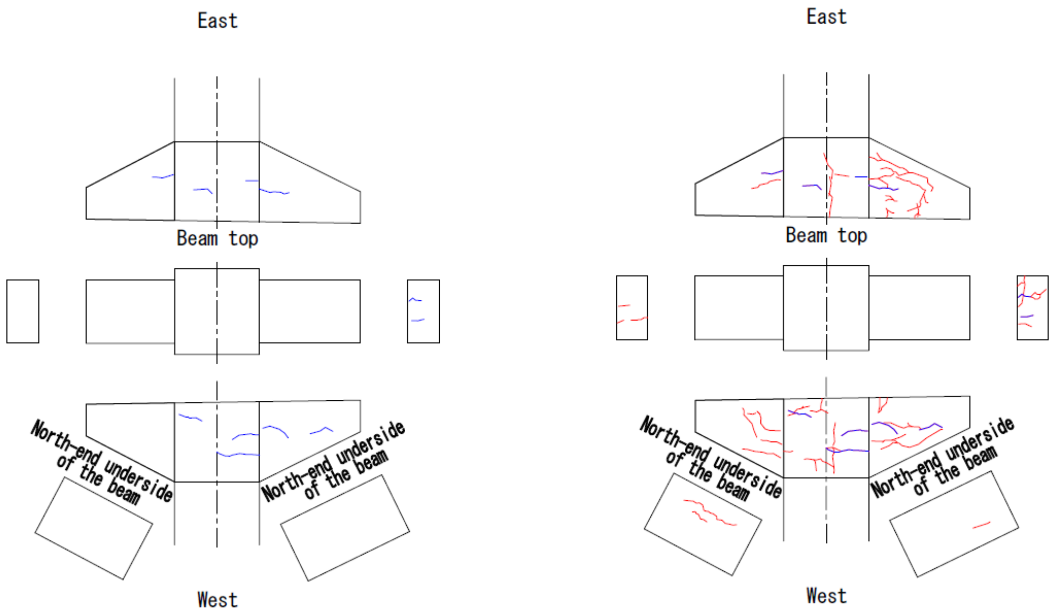


FIGURE 3: Sketches of cracks in Pier I in the surface coatings (left) and after surface coating removal (right)

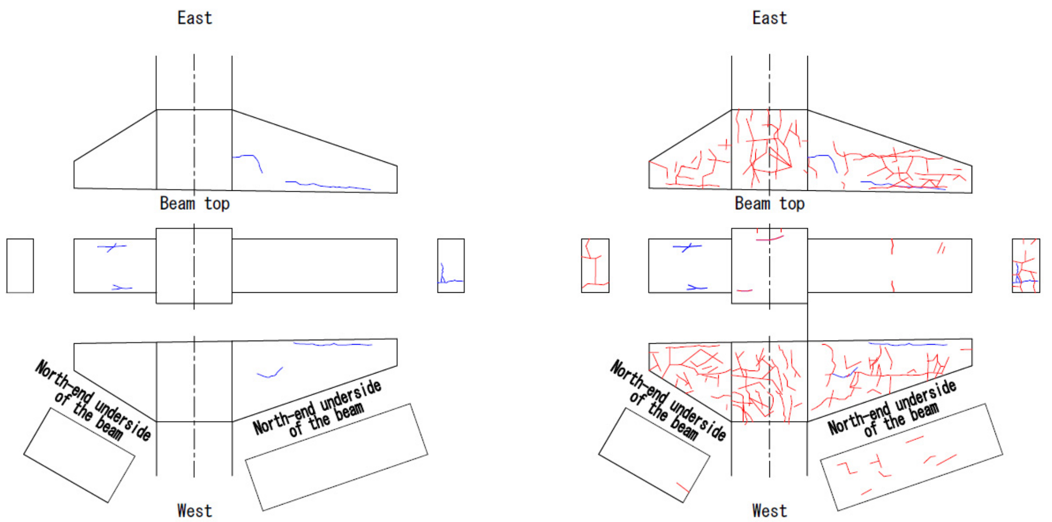


FIGURE 4: Sketches of cracks in Pier II in the surface coatings (left) and after surface coating removal (right)

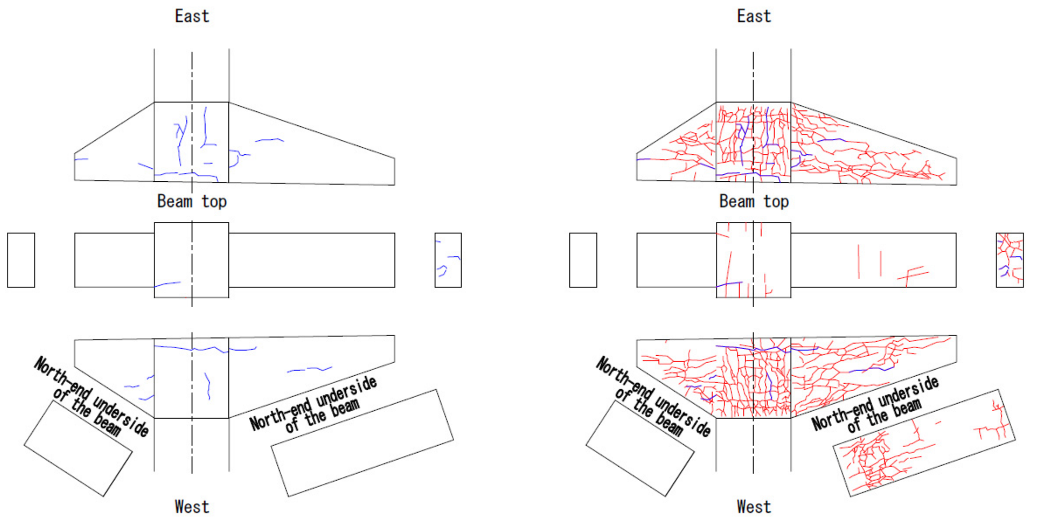


FIGURE 5: Sketches of cracks in Pier III in the surface coatings (left) and after surface coating removal (right)

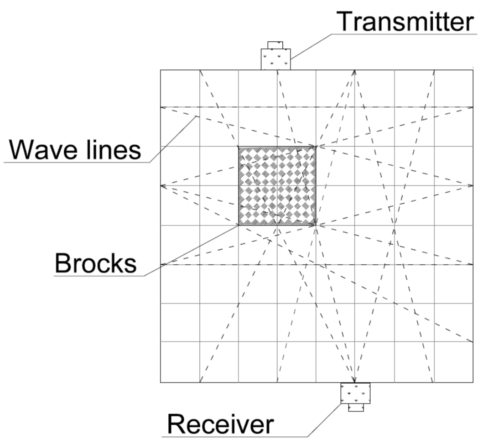


FIGURE 6: Schematic of ultrasonic test

FIGURE 7: View of ultrasonic measurement

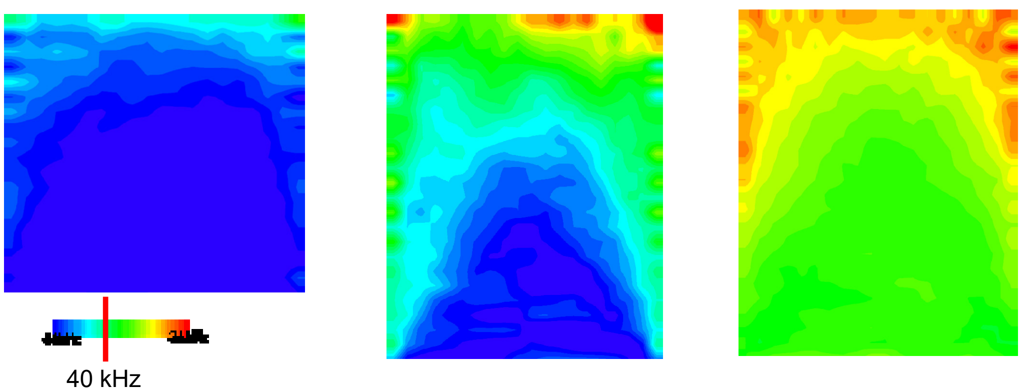


FIGURE 8: Tomographic analysis results for Pier I (left), Pier II (middle) and Pier III (right)

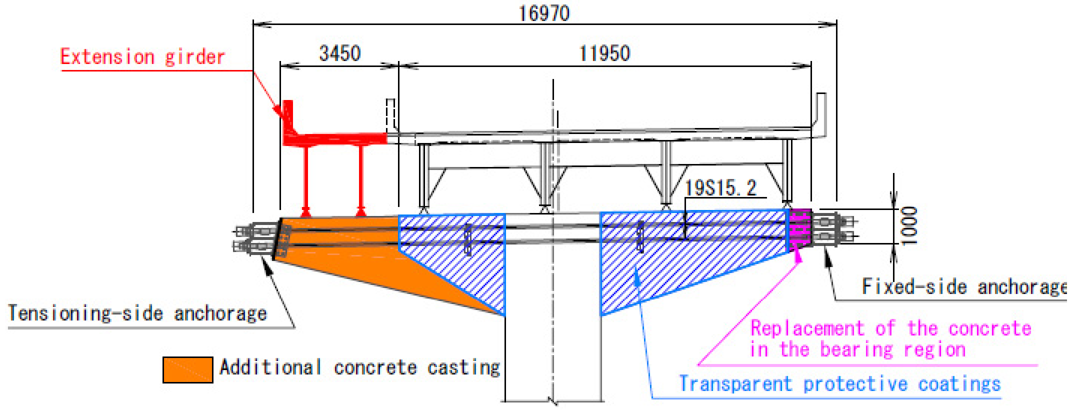


FIGURE 9: Schematic of the strengthening design for Pier II

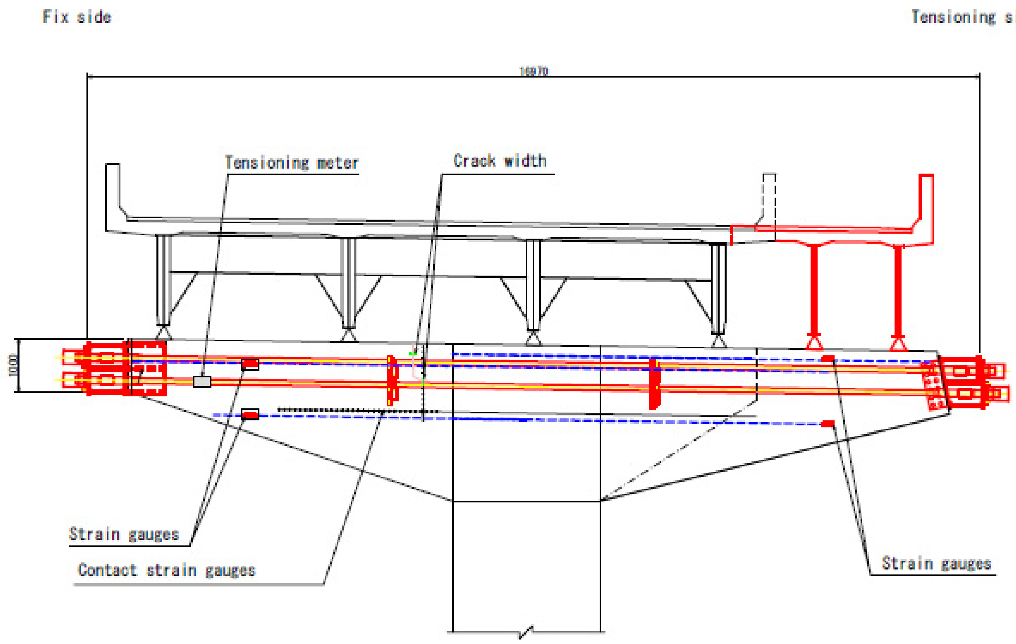


FIGURE 10: Schematic of the measurement plan