A PROPOSAL FOR REHABILITATION OF ASR-AFFECTED BRIDGE PIERS WITH FRACTURED STEEL BARS

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Abstract
The brittle fracture of steel bars in ASR-affected bridge piers has recently been reported in many places in Japan. At present, the maintenance and rehabilitation techniques for such severely damaged concrete piers due to ASR have become a great concern. The authors carried out the series of investigation on both the reduction in compressive strength of concrete and the degree of fracture of steel bars in concrete piers in which the severe deterioration occurred again after the surface coating had been applied. This report describes the details of survey on the re-deterioration of ASR-affected bridge piers and their rehabilitation method.

Keywords: ASR, re-deterioration, fracture of steel bars, pessimum percentage, rehabilitation

1 INTRODUCTION

It is well known that the alkali-silica gel formed around aggregates leads to the expansion of concrete and thereby the cracking of structure. The brittle fracture of steel bars has also been found out in severely deteriorated bridge piers in Japan [1]. At present, the development of rehabilitation and maintenance techniques for such ASR-damaged bridge piers becomes an important subject. The Japan Society of Civil Engineers, JSCE, established a task work which has compiled a report about the mechanism of fracture of steel bars and the load-bearing capacity of members with the fracture of steel bars in August 2005. However, there are very few reports concerning the deterioration of bridge piers with fracture of steel bars and their rehabilitation methods [2]. The authors obtained the chance in carrying out an investigation on bridge piers with the loss of integrity of structure due to the fracture of steel bars in bridge piers. Based on the results of the investigation including the strength of cores and the degree of fracture of steel bars, the bridge piers were reconstructed in 2006 and 2007.

In the first part of the paper, the investigation on ASR-affected bridge piers, which were severely deteriorated again immediately after the coating on surface and the injection into cracks had been applied, is described. In the second part of the paper, both the rehabilitation technique and the execution process for such bridge piers across the Jinzu River are introduced.

2 OUTLINE OF ASR-AFFECTED BRIDGE PIERS

The Arisawa Bridge, which was completed in 1972, is the Warren-type steel structure one having seven spans across the Jinzu River in Toyama prefecture in Japan. The super structure is supported by seven reinforced concrete hammerhead piers with an oval cross sectional shape across the river. Although the time when the cracks have initiated due to ASR is not reported, the piers were subjected to the coating on surface and partially injection into large cracks in 1989 at the time when 17 years have passed after construction. The surface coating was carried out in accordance with the technology development project supported by the Ministry of Construction, which included injection into cracks with an epoxy resin; the category of soft and thick type coating of A-1 was applied. Table 1 presents the specifications for the soft and thick type coating materials for ASR-affected structures at that time. However, in intermediate piers P_2 and P_4, the visible cracking on coating was observed within 5 years after repairing. This is attributable mainly to the drainage water going into the upper portion of pier from the joint of decks. In this region, during the winter, a lot of deicing salts, mostly sodium chloride (NaCl), are used on the bridge decks in order to assure the safe traffic conditions. Thereafter, the cracking of bridge piers rapidly extended to all coated surfaces. Figure 1 shows the external appearance of P_2 bridge pier.

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3 SURFACE CRACKING OF BRIDGE PIER

3.1 External appearance of cracking of P_2 bridge pier

In the hammerhead beam of P_2 pier, the cracks pierced through the 0.5 mm thick surface coating, where the horizontal continuous cracks with the width of 1 to 2 mm were the most prominent. On the other hand, in the column, the cracks with the width of 2 mm occurred along the longitudinal steel bars as well as the small map cracking. Moreover, the water leakage from the gap of expansion joints was observed near the beam soffits and on the side surface of the column. This may suggest that the deterioration of concrete due to ASR is accelerated especially at the portion of bridge piers where the water is always supplied from the deck. Figure 2 shows the cracking along the lower edge of hammerhead beam of P_2 pier. Surprisingly, the very large cracks of 20 mm width and gap of 10 mm occurred, where the partial spalling near large cracks was also observed. Figure 3 shows the comparison between the cracking inspected in 1989 and in 2004, respectively. The location of cracking at the inspection did not necessarily coincide, presumably because the cracks injected by the epoxy resin in 1989 had widened and extended in 2004. Table 2 presents the density of cracks of 0.4 mm and larger in width at various portions of P_2 and P_4 piers. In the column of pier P_4, the crack density on the ground was higher than that in the air atmosphere, where large cracks of 1 to 2 mm and larger in width occurred. It is therefore inferred that the deterioration of bridge piers in the river may be strongly affected by the water supply from the river itself.

3.2 Deterioration of surface coating

In the P_2 and P_4 piers, the surface coating itself was deteriorated more significantly by the external actions, mainly the water supply from the rainfall and the wetting and drying by sunshine, where the coating was swelling with water and partially peering off. Although the surface coating material was prepared according to the specifications at that time, the elastic property of the coating was totally lost and its adhesive property was also reduced to a significant extent. From the results, it has been found out that the waterproofing effect of coating material is insufficient especially in the Hokuriku region. The protective measure for water seepage, particularly the leakage from the joint of deck, is required to improve the durability of waterproofing for the coating material itself.

4 PROPERTIES OF CORES FROM BRIDGE PIERS

4.1 Visual observation of cores

A lot of full-length Cores, 55 mm in diameter and 1,500 mm in length, were drilled from the side of hammerhead beam of P_2 pier, where the water streaks on the side surface of pier were observed. Figure 4 shows the degree of deterioration of internal concrete of cores. The cores were easily broken out at the time of drilling, becoming small pieces of 50 mm to 100 mm. This phenomenon may indicate that the concrete is damaged not only in the surface portion, but also in the interior due to the excessive expansion of concrete in the progress of ASR. It is also observed at the broken portions of cores that a lot of white deposits of alkali silica gel were produced around the andesite particles contained in both the river sand and river gravel.

4.2 Mechanical properties of concrete

Figure 5 shows the relationship between the compressive strength and the static modulus of elasticity in cores drilled from the hammerhead beam, column and caisson of piers. As shown in Figure 5, the reduction in the elastic modulus of cores taken from ASR-deteriorated piers was more significant compared with that in the compressive strength, all values obtained being under the theoretical curve estimated for the sound concrete specified in the Standard Specifications for Road Bridges. In particular, the elastic modulus of cores in the hammerhead beam was particularly lower than those of columns, presumably because of the low steel reinforcement ratio for confinement and the high water supply from the superstructure. Figure 6 shows changes in the compressive strength of cores drilled from the beam. A significant reduction in both, the compressive strength and the elastic modulus of cores was observed even after repairing.

Figure 7 shows the expansion behaviors of cores in accelerated curing test immersed into a saturated NaCl solution at 50°C, Danish method. In this accelerated method, the expansion of more than 0.1 % is judged as the core has a residual expansion capacity [3]. The cores taken from the beam or the column of bridge piers continued to expand, while those from the caisson were very low and almost constant, even in the accelerated curing condition.

4.3 Chloride content and alkali content of concrete

Figure 8 shows the total chloride ion content of cores drilled from the hammerhead beam of P_4. The chloride ion content of concrete remarkably increased from 1990 to 2001, where the value at surface was as higher as 2.1 kg/m^3. Judging from the fact that in Japan, the restriction on studded tires was enforced in 1990, this can be attributed to the influence of deicers sprayed over the road decks during the winter. It is also necessary to investigate the degree of steel corrosion in bridge piers, as the chloride content is beyond the limit in which the steel corrosion can onset at the location of steel bars.

The water soluble alkali content, Na_2O equivalent, in concrete was 2.6 and 2.9 kg/m^3 for the hammerhead beam and the column, respectively. When calculating according to the method specified by the MLIT, these values correspond to a total alkali content of 4.3 kg/m^3. Judging from the fact that the cement with an alkali content of about 1% was locally used in the 1970s and 1980s, the occurrence of ASR is also related to the high alkali content of cement used [4].

5 FRACTURE OF STEEL BARS IN BRIDGE PIERS

5.1 Degree of fracture of steel bars

It has been pointed out that the fracture of steel bars may be a phenomenon resulting from an excessive expansion force due to ASR, where the concentration of stress occurs at the fissure inside the bending steel bars, leading to the brittle fracture in the process of expansion of concrete due to ASR [5]. Table 3 gives the percentage of fractured or cracked steel bars based on the survey when demolishing the hammerhead beams of P_2 and P_4 piers. The numbers of fractured steel bars also include those broken out at the time of demolition. The crack and flaw of steel bars were visually observed by means of the fluorescence dye liquid method or the magnetic particle method. According to the results of chemical compositions and mechanical properties of steel bars used, the steel bar corresponds to SD 295A specified in JIS G 3112, which is a normal steel bar produced at the electric furnace. The fracture was also found at the weld point of D32 mm main steel bars in P_4 pier. The smooth surface of steel bars suggests that this fracture is caused by the inadequate welding at construction.

5.2 Feature of fractured steel bars

Figure 9 and Figure 10 show the external appearance of fracture of steel bars and the typical fractured surfaces of a D16 mm stirrup, respectively. The fracture of steel bars is concentrated on the beam soffits which are geometrically vulnerable to the rainwater or leakage water. In the hammerhead beam of P_2 pier, the percentages of fractured bars were as high as 43% and 63% for the stirrups and main bars, respectively. Interestingly, the fracture of steel bars was more remarkable on the compression side of beams. Steel corrosion was also observed on the surface of fractured steel bars. This indicates that time has already passed since they fractured. It is inferred that the fracture of steel bars in the beam with a low reinforcement ratio may weaken the confinement by steel reinforcement, which leads to a successive fracture of steel bars along the entire beam. It is also inferred that the stirrup in the center of beam has been easily fractured by the stress relaxation due to the removal of cover concrete or the vibration of breaker at demolition. On the other hand, the fracture of transverse steel bars was not found at all because they do not have the bending corner of 90 °. Although the fracture of steel bars has occurred mostly at the bending corner of 90° or 135 °, for the curved steel bars, the probability of fracture is inferred to be very low. Figure 11 shows the typical appearance of cracked steel bar at the inside part of bending. The location of cracked steel bars tended to scatter, which has no relation with the cracking on the concrete surface. Thus, the presence of cracked steel bars is also considered to be an important problem for the load-bearing capacity of structures at the earthquake.

The Maintenance Guidelines require a detailed investigation; (1) when continuous cracks with a width of not less than 2 mm, (2) when continuous cracks with a width of not less than 1 mm at the top edges of the vertical surfaces of hammerhead piers, (3) when level differences of not less than 2 mm at cracks are observed, as this indicate a possibility of bar fracture. The results of this survey totally agreed with the requirements of the Maintenance Guidelines. Therefore, the Maintenance Guidelines can also be applied to the ASR-affected bridge pier coated on surface. However, it should be paid attention to the fact that the fracture of steel bars is possibly present even if the visible crack is not detected on the coated surface.

6 MINERALOGICAL CHARACTERISTICS AND ALKALI-SILICA REACTIVITY OF AGGREGATES

6.1 Mineralogical characteristics

The aggregates used in concrete were river sand and river gravel supplied from the Jouganji river in Toyama prefecture, which contains a certain amount of volcanic stones. Figure 12 shows the result of polarization microscopy. A representative view shows that there are multiple cracks near the andesite particles, and that reaction rims exist around andesite particles in both river sand and river gravel. It is observed that the andesite particle is the most reactive one, which contains the cristobalite, opal and volcanic glass as reactive components. The high alkali-silica reactivity of river sand used is very similar to that of river gravel, presumably resulting in the severe deterioration of bridge piers due to the successive expansion of concrete.

6.2 Rock compositions and pessimum percentage

Table 4 gives the rock compositions determined by the visual observation of core photograph. Andesite particles were calculated at the percentage of 20 and 42 % for the river sand and the gravel, respectively. On the other hand, the percentage of other reactive stones such as the rhyolite, green tuff and chert was marginal, being observed with little reaction by polarization spectroscopy. In the previous study, the mortar bar with the andesite particle content of 40% showed the maximum expansion, indicating the pessimum percentage of andesite particle to be about 40 % [6]. It is considered that the andesite particle content of fine and coarse aggregates in concrete is as same as the pessimum percentage. This is also another reason for the occurrence of significant damage in these bridge piers.

7 REHABILITATION METHOD

7.1 Selection of reconstruction of bridge pier

A precise prediction of deterioration of ASR-affected structures in future is currently difficult. Furthermore, the rehabilitation method is not yet established for bridge pier with the fracture of steel bars. For this reason, the reconstruction of the beam alone was initially planned. However, when demolishing the beam of bridge pier, the concrete with lots of internal cracks was significantly fragile, although its compressive strength was satisfied with the designed strength.

Actually, ASR gel was observed in all the concrete fragments, and the concrete was readily breakable. Since it was judged that the concrete deterioration of the column was similar to that of the beam, it was also decided to reconstruct the column from the standpoint of preventive maintenance.

7.2 Reconstruction process

In order to demolish the existing pier in service, a temporary bent was required to support the superstructure of steel structure. The temporary support points were selected on the bottom chords at the location of 2 m inward from the existing supports. The bent columns under the temporary support points were connected with horizontal H section steel beams and stiffened with diagonal bracing. Figures 13 and 14 show the bend planning draw and the state of its construction, respectively. Since approximately 3 000 kN of concentrated load acts on the bottom chord for each temporary support point, truss stiffeners were temporarily applied between panel points to bear the reaction forces from support points. Laminated rubber bearings were installed on each support point. After setting a 500-ton hydraulic jack at each of the four support points, the superstructure was jacked up 50mm to substitute the supports. As for the foundations of the temporary bent, H sections were placed on the existing caissons to substitute the bearing, while transferring the loads to the existing caissons, thereby preventing settlement of the bent in use.

The hammerhead beam of the existing pier was completely removed, but the D25mm main bars of the columns were retained. When replacing the column concrete of bridge pier, the soundness of the main bars in the existing column is a key issue. Despite surface cracking as wide as 2 mm, no steel corrosion was observed on the main bars of the column, presumably because cracks were filled with ASR gel. The main bars were reused by carefully removing the adherent debris by a water jet in order to ensure the bond with new concrete. The D41mm longitudinal bars were additionally erected in holes drilled in the top slab of the caisson to increase the flexural capacity so as to achieve the seismic performance required by the Standard Specifications for Road Bridges in 2002. Figure 15 shows the overview of the reconstructed bridge pier.

8 CONCLUSIONS

The piers in Arisawa bridge that are severely deteriorated due to ASR, were investigated in detail. Based on the results of a survey on the fracture of steel bars in bridge piers, both the beam and the column in bridge piers were completely reconstructed.

The main results obtained in this survey are summarized as follows:

(1) The deterioration occurred again on the surfaces of bridge piers within 5 years after repairing, because the expansion of concrete due to ASR continued.

(2) Both the elastic modulus and compressive strength of concrete were significantly reduced even after repairing, indicating that the deterioration due to ASR progressed even after repairing.

(3) The stirrup steel bars fractured at the percentage of 43 % on the compression side of beam in bridge piers, while the bent-up steel bars similarly at the percentage of 63 %. Based on the results of survey, the reconstruction for all bridge piers except for the footing was adopted as a rehabilitation method.

(4) The evaluation whether or not the fracture of steel bars actually may occur in bridge pier coincided as a whole with the maintenance guideline proposed by the MLIT.

(5) Large numbers of steel bars with cracks at the bending portion also existed in the interior portion of bridge piers, most of them readily fracturing at the demolition work.

(6) The content of andesite particles in the river sand and river gravel from the Jouganji River ranged from 30 % to 40 %, which was considered to be around its pessimum content.

9 ACKNOWLEDGMENTS

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10 REFERENCES

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Figure 1: Cracking of P_2 pier after repair

Figure 2: Cracking along beam soffits

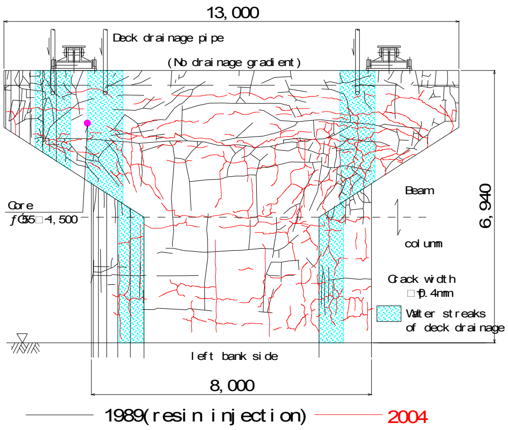

Figure 3: Comparison of cracking in P_2 pier in 1989 and 2004

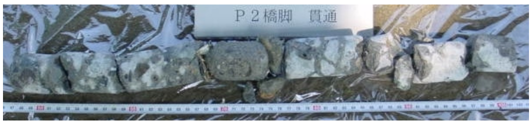

Figure 4: Failure of cores drilled from beam "ϕ55×1,500 mm"

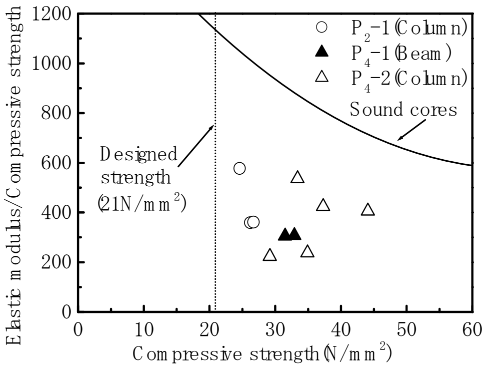

Figure 5: Relationships between compressive strength and elastic modulus of cores taken from RC piers

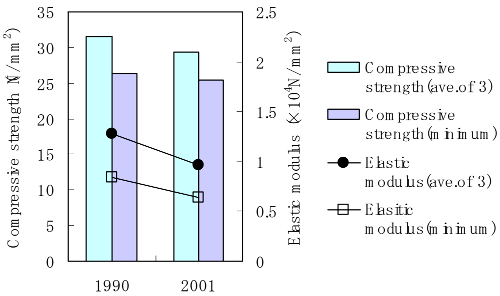

Figure 6: Changes in compressive strength of cores with time

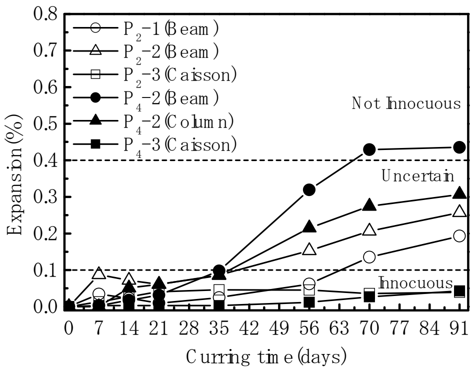

Figure 7: Expansion behavior of cores stored in saturated NaCl solution

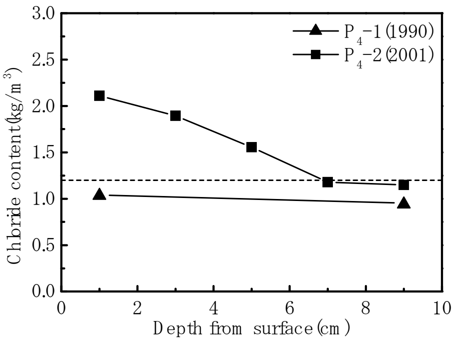

Figure 8: Chloride content of concrete

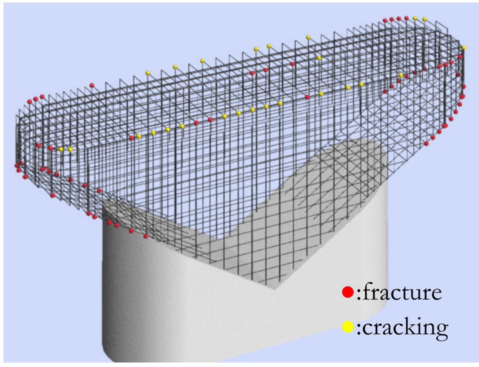

Figure 9: Portion of fracture of steel bars in the beam of P_2 pier

Figure 10: Surfaces of fracture of steel bars

Figure 11: Side view of steel bar with crack at bending

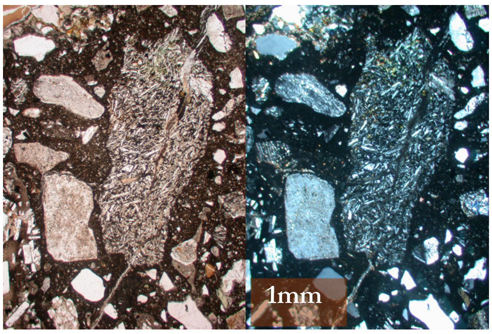

Figure 12: Observation of thin section of core by polarization microscopy (left: with plane polarized light, right: with crossed polars)

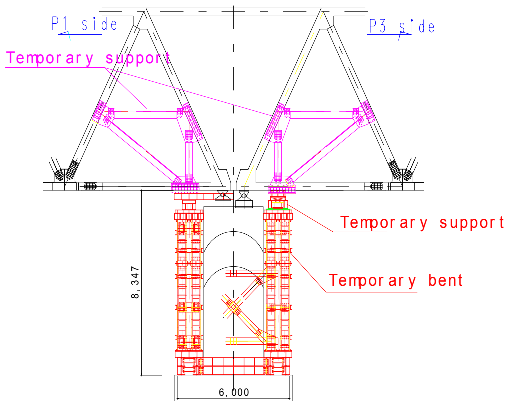

Figure 13: Design of temporary bent

Figure 14: Supporting by temporary bent

Figure 15: Total view of re-constructed P_2 pier