Cause and repair of detrimentally cracked beam in reinforced concrete bridge pier

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ABSTRACT: Routine inspection detected a crack up to 10 mm in width in reinforced concrete (RC) pier C1-1034 of the Circular 1 Route (C1) of the Tokyo Metropolitan Expressway. As an emergency repair, the overhanging beam was removed and rebuilt. A subsequent inspection into the cause of the crack found that the anchor bolts used to install the brackets after the construction of the bridge had cut the reinforcing bars of the beam. Thus, for those piers to which a bracket was installed during the same installation project, further inspections of the reinforcing bars were conducted to look for defects. As a result, defects were confirmed in the reinforcing bar of many of the inspected piers. Reinforcement using outside cables was added to one pier (C1-1039), as 80% of the cross-section of its reinforcing bar was defective.

1 INSTRUCTIONS

Tokyo Metropolitan Expressway is a major network of expressways supporting the automobile traffic in the Tokyo Metropolitan Area. Approximately 1.1 million vehicles use the network each day. Approximately 80% of the network’s total road length of 286.8 km is an elevated bridge. It is one of the largest urban expressway systems in Japan. (Fig. 1)

Routine inspection (close visual inspection) of the Circular 1 Route (C1), located at the center of the Tokyo Metropolitan Expressway network, revealed a crack measuring up to 10 mm in width in the overhanging part of the beam on reinforced concrete (RC) pier C1-1034 (Photo 1). The crack ran continuously along the upper side of the beam, as well as on both lateral sides. Since the crack was very wide, bents were immediately installed as a temporary measure to support the damaged pier beam. After jacking it up, we conducted emergency repairs involving removal and rebuilding the beam portion. The entire repair project took two months, during which a working zone was set up below one of the three lanes of the elevated expressway leading to central Tokyo. The cause of the crack in C1-1034 was found to be the anchor bolts, which were applied after the pier was constructed in order to install a bracket. These bolts cut the reinforcing bars of the beam.

In response to this finding, we conducted Non-Destructive Tests (NDT) of the piers that had undergone the same bracket installation work to examine the condition of the reinforcing bars of the beams. The inspection, conducted from the upper side of a beam, measured the thickness of the covering concrete and confirmed the interference of the reinforcing bars and the post-construction anchor bolts. As a result, many of the piers that underwent the same installation work were found to have defective reinforcing bars. One response to the inspections involved reinforcement with outside cables (as was applied to pier C1-1039), which had defects in approximately 80% of the cross sections of the reinforcing bars.

This report describes the emergency repairs conducted on C1-1034, the NDT for defects in the reinforcing bars of piers to which the same brackets were installed, as well as the design and installation work required for the outside cable reinforcement applied to C1-1039.

2 CONSTRUCTION AND REBUILDING

HISTORY OF PIER C1-1034

Construction work on pier C1-1034 began in January 1960 and was completed in July of the same year.
Figure 1. Tokyo Metropolitan Expressway Network.

Figure 2 shows the pier in cross section. The pier has a RC structure and is 4.8 m high. The pier is of a rectangular shape, 1.2 m wide in the direction of the expressway axis and 1 m in the across direction. The beam is 16 m long (with an overhang of 3.5 m on each side), 1.2 m wide, and 0.6 to 1.2 m high. The upper structure is composed of simple steel composite I girders.

The section of the expressway in question was opened in December 1962. The marked increase in the traffic on the system because of the Tokyo Olympics in 1964 and other reasons meant that another lane was added to each side in 1972 to alleviate traffic congestion.

More recently, from 1995 to 1997, the steel bearings of the movable side were replaced with rubber bearings to handle the increase in vehicle sizes. Brackets were also installed to broaden the outside edges. At approximately the same time, in response to the “1995 Kobe Earthquake”, steel plate wrapping was applied to the pier as earthquake-proof reinforcement. Furthermore, the steel bearings of the fixed side were also replaced with rubber bearings between 1997 and 1999, and devices were installed to prevent displacement of the upper ends of the pier.

3 CAUSE OF THE CRACK

The crack, measuring up to 10 mm in width, appeared on the upper surface of the beam and across the entire width of the beam, which measured 1,150 mm. On the lateral sides of the beam, the crack was found around the border of the brackets used to broaden the outside edges and also on the lower side. On the Edobashi and Haneda sides, the maximum crack width was 2.0 mm and 3.0 mm, respectively, and its length was 950 mm on both sides, reaching the lower surface of the beam.

We removed some of the concrete where the crack crossed the reinforcing bars on the upper face of the beam, and visually inspected the reinforcing bars and...
the post-construction anchor bolts. As a result, we confirmed that two reinforcing bars had been completely cut by the post-construction anchor bolts (Photo 2).

In addition, the part of the beam that had been cut was transferred to the Haneda yard for further analysis where we removed the concrete to examine the reinforcing bars inside. As a result, we found that 9 of the 24 reinforcing bars (ø 32 mm) had been similarly cut by the post-construction anchor bolts, which were added when installing the steel bracket. We also discovered that an additional reinforcing bar had been cut by multiple post-construction anchor bolts.

Next, we collected a concrete core from the area surrounding the crack and subjected it to indoor tests. The tests revealed that the average crushing strength of the concrete to be 42.5 N/mm² with a maximum carbonation depth of 5.4 mm, indicating that the concrete was in good condition. The average yield strength of the reinforcing bars was 338 N/mm², the average rupture strength 507 N/mm², and the average elongation 30%, which demonstrated that the reinforcing bars were sufficiently strong. The reinforcing bars were round steel bars (ø 32 mm) that were equivalent to SR30 (SS50).

Since no degradation or lack of strength was observed in the concrete, we determined that the crack in question occurred primarily because the reinforcing bars were cut by the post-construction anchor bolts.

**4 REPAIR WORK ON PIER C1-1034**

We considered and compared the four options shown in Fig. 3 for the repair of the C1-1034 pier. 1) Removal and rebuilding of the damaged section of the beam, 2) Reinforcement with pre-stressed concrete (PC) cables and steel plates, 3) Reinforcement with steel plates, 4) Reinforcement with fiber sheets.

We considered all four options in terms of the reinforcement effects, the feasibility of the installation work including its required duration, and maintenance. Plan 1, the removal and rebuilding of the damaged beam section, was selected because this option has three advantages: 1) it facilitates highly reliable repair work, since it workers are able to check the inside the damaged beam section; 2) work duration would be short; 3) inspection after the repair is straightforward. Furthermore, we selected RC brackets to broaden the outside edges of the girder 1 (G1), and applied steel reinforcing bars and some cast concrete while repairing the beam. After installing bents to support the girders, we cut the beam of the existing pier and, simultaneously with the reinforcement using steel bars and the concrete casting, we examined the reinforcing bars in the cut-off part of the beam and conducted concrete-mixing trials. After the concrete casting, the bearings were installed and the girder-supporting bents were removed. Installation of the reinforcing bars and cast concrete are presented below.

4.1 Connecting the reinforcing bar

After cutting the existing reinforcing bars, we removed the concrete to expose the existing reinforcing bars to 1 m in order to connect new reinforcing bars. We examined the exposed reinforcing bars for damage using ultrasonic testing and the impact-echo method. As a result, we confirmed that one reinforcing bar
had a crack at approximately 0.6 m from the cutting surface.

We selected highly reliable pressure gas welding to replace the existing reinforcing bars by connecting new reinforcing bars. Where it was difficult to create sufficient space between two reinforcing bars for this welding method, we used enclosed welding. Since the existing reinforcing bars were round steel bars (SR30), we conducted tension tests to determine whether the joints with the new reinforcing bars, made of steel bars of a different shape (D32), had sufficient strength. Tension tests confirmed that sufficient strength was obtained through both the pressure gas and enclosed welding methods. We simultaneously conducted ultrasonic testing to examine the damage from the side of the new reinforcing bars to confirm that the welding was of a suitable standard. The joint positions were arranged in a zigzag pattern at an interval of 800 mm was ensured, which was 25 times greater than the diameter of the reinforcing bar, ø 32 mm.

Furthermore, to compensate for the loss of reinforcing bars resulting from cutting the reinforcing bars, we decided to attach anchor-reinforcing bars down to the edge of the pier. However, bearings in the positions that we originally selected for drilling for the anchor reinforcing bars, bearing anchor bolts for the girder 2 (G2) bearings, reinforcing bars of the pier, the G2 bracket anchors (horizontal) for broadening the outer ends complicated the selection of sites for drilling. To select optimal positions for drilling the additional anchor reinforcing bars, we drilled 28 holes (ø 16 mm) to examine what was inside. Although most of these holes showed interference/interfered with a reinforcing bar, after careful inspection with a fiberscope we managed to select four positions for drilling ø 40 mm holes.

4.2 Concrete casting

In order to secure early-stage strength, we selected rapid-strength Portland cement concrete and set its nominal strength at 40 N/mm². As the compensation concrete for shrinking, we used an expanding material in order to secure the union between the new beam portion and the existing pier, as well as to reduce cracking. Furthermore, we mixed in 0.05 vol% of polypropylene fibers to prevent the concrete from chipping.

Concrete casting began at 11:00 pm, when traffic could be kept out of two lanes of the road below the elevated expressway. To reduce the duration of the work, we modified the existing beam-supporting bents for formwork and scaffolding work. However, it was discovered that these bents were connected to girder-supporting bents and that they carried/conducted numerous vibrations, caused by vehicles traveling along the expressway. We therefore decided to cut the joint between the girder-supporting bent and the scaffolding work material to reduce these vibrations while the concrete was cast and cured.

5 INSPECTION OF REINFORCING BARS ON THE PIERS

As stated above, after cutting the reinforcing bars, the cause of the crack in pier C1-1034 was found to be the post-construction anchor bolts, which were installed in brackets in order to broaden the outside ends. We then conducted NDT of the damage to reinforcing bars of the piers that underwent the same bracket installation work.

In these inspections, we first searched for reinforcing bars from the upper face of the beam as well as from its two lateral sides using RC radar (Photo 3), and assessed the possibility of damage based on the offset distance between the reinforcing bars and post-construction anchor bolts. Next, for those post-construction anchor bolts judged to be potentially defective by the reinforcing bar inspection, we exposed the crossing sections of the reinforcing bar and the post-construction anchor bolt by boring cores (ø 150 mm) from the upper face of the beam. Then we visually checked the reinforcing bar for defects. In addition, at those portions of the beam where the visual check revealed serious damage to the reinforcing bar, we measured the covering depth of the reinforcing bar by small-diameter drilling, and determined a damage ratio for the cross-section of the reinforcing bar caused by post-construction anchor bolts.

6 DEFECTS WITH THE REINFORCING BARS OF THE C1-1039 PIER

For pier C1-1039, which is another RC pier, our inspection for reinforcing bar damage confirmed that the
damage ratio of the cross section of the reinforcing bar caused by post-construction anchor bolts was approximately 80% for the mountain-side overhanging beam and approximately 70% for the sea-side overhanging beam. The crack had not been discovered in the bracket neighborhood in the sea-side beam though the crack that expanded from the bracket of 380 mm in length and 0.1 mm in width in the mountain-side beam had been discovered in the externals visually check of this pier. Described below are the results of our visual inspection of the mountain-side overhanging beam, conducted by boring cores and measuring the covering depth of the reinforcing bar through small-diameter drilling. The interference of the post-construction anchor bolts is also mentioned.

6.1 Results of the visual inspection through core boring

For a mountain-side overhanging beam, we bored a core of ø 150 mm from the upper face where the bracket anchor bolt crossed the reinforcing bar of the beam and visually inspected the area of where they crossed. Figure 4 and Photo 4 show the covering depth of the reinforcing bar and the anchor bolt measured from the upper face of the beam, as well as the interference condition.

Boring two cores confirmed that four main reinforcing bars had been cut.

6.2 Measuring the covering depth through small-diameter drilling

As stated above, the core boring in two positions confirmed that four reinforcing bars had been cut. Due to a variety of restrictions, however, such as obstacles and the limited working space on the upper face of the beam, it was difficult to conduct any further core boring. We therefore measured the covering depth of the reinforcing bars by drilling small holes with a small-diameter drill of ø 15 mm at the positions where radar inspection from the upper face of the beam confirmed the existence of the reinforcing bars.

Figure 5 summarizes the results of the covering-depth measurements through small-diameter drilling and measurement through core boring. For the mountain-side overhanging beam of the C1-1039, some 80% of the cross-section areas of the reinforcing bars inspected were defective and we decided to reinforce them immediately.

7 DESIGNING AND APPLYING REINFORCEMENT TO THE C1-1039

7.1 Designing reinforcement with outside cables

For reinforcement, we selected the outside cable method, which has proven to be effective in the flexural strengthening of pier beams.

The most common method for applying tension to a PC cable is to fix one end of the cable and draw the other end with a center-hole jack. With C1-1039, however, there were broadening girders on both sides of the expressway, which did not permit sufficient space within which to work. We therefore decided to use a drawing jack, to apply tension to the cable.

Furthermore, at 700 mm, the height of the end of the beam was small. Thus, to maximize the eccentric bending of the outside cable, we decided to use one cable on each side, 250 mm from the upper end of the beam (Fig. 6).

Since approximately 80% of the reinforcing bars of the beam were found to be damaged, we assumed that the bottom sections of the beam attached to the piers were made of unreinforced concrete and decided to apply tension of 1,200 kN per cable, so that the
Figure 6. Reinforcement with outside cables.

pre-stress of the outside cable would offset the upper edge stretching stress of the concrete generated by the B-live load.

7.2 Procedure for the work

After setting up the scaffold, we examined the reinforcing bars in the beam by core boring and drilling. As described earlier, a large portion of the cross-sectional area on the reinforcing bars was damaged. For this reason, after designing the reinforcement using PC cables, we prepared the PC cables and fixtures and placed them on the scaffold. After installing the drawing jack, we tensioned the PC cables while measuring dynamic strain.

As a daily control until the reinforcement with the PC cables was completed, we installed a gauge in the crack on the upper face of the beam to check for changes in the width of the crack.

7.3 Tensioning the outside cables

We installed concrete and reinforcing bar gauges, etc., to check the effectiveness of the reinforcement by the tensioning of the outside cables and to check the state of the concrete on the end face of the beam during the cable tensioning work.

To confirm the effectiveness of this reinforcement, we measured the dynamic strain continuously for 24 hours: 1) before tensioning, 2) after applying 50% of full tension (600 kN), and 3) after applying full tension (1,200 kN). Also, the tensioning process proceeded in a stepwise manner, with 120 kN of tension applied for each step, while we checked the appearance of the concrete and recorded the gauge values.

7.4 Reinforcing effect

Table 1 shows example values measured before and after tensioning. In the values in the table, there are fluctuations in the strain and stress generated in the reinforcing bars and concrete in the sections where the beam is attached to the pier.

The minimum values measured during the 24 hours before tensioning are considered to be those measured with no live load on the expressway. Thus, if the maximum values generated by the in-service load after reinforcement with the outside cables are below the minimum values before tensioning, we can conclude that our intended objective of offsetting the stretching stress caused by a live load has been achieved.

Because the maximum values after applying 50% of the full tension were less than the minimum values before tensioning, reinforcement with 50% of the full tension was sufficient to offset the stretching stress caused by an in-service load.

Nevertheless, we applied the full 100% tension. We can, therefore, conclude that the reinforcement was sufficient for the service expected of the expressway.

Table 1. Stress of reinforcing bar and concrete.

<table>
<thead>
<tr>
<th>Stress (MPa)</th>
<th>Reinforcing bar</th>
<th>Concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max.</td>
<td>Min.</td>
<td>Max.</td>
</tr>
<tr>
<td>Before tensioning</td>
<td>6.8</td>
<td>0.76</td>
</tr>
<tr>
<td>50% tension</td>
<td>−4.4</td>
<td>−0.28</td>
</tr>
<tr>
<td>Full tension</td>
<td>−11.6</td>
<td>−1.06</td>
</tr>
</tbody>
</table>

Photo 5. C1-1034 after reinforcement.

Photo 6. C1-1039 after reinforcement.
We removed a overhanging beam of C1-1034 and rebuilt it. We also inspected the reinforcing bars of the other piers that underwent the same anchor bolt installation work to survey the damage caused by the post-construction anchor bolts on the reinforcing bars.

For other piers located in the geographical section where some damage to a reinforcing bar was found (although not as severe as the damage to C1-1039), we decided to measure and organize the degree of stress generated by in-service loads on each of those piers, and we plan to continue monitoring those piers. Instead of regular inspections twice a year, we decided to conduct visual inspections below the expressway three times each year.

Rebuilding work of various kinds in the future will require the use of post-construction anchor bolts. In designing such work, it is very important to minimize the diameter and length of drilling holes as much as possible. It is also important for the bolts to be placed in positions where they do not interfere with any existing reinforcing bars.

To conclude, in repairing and reinforcement designs such as in this project, it is difficult to restore the original condition perfectly. In future repair and reinforcement design projects that similar to this one, setting the optimal degree of reinforcement for each actual situation will be very important.

At last, Photos 5 and 6 show C1-1034 and C1-1039 after reinforcement.