PRESTRESSED CONCRETE BEAM END REPAIR (INTERIM REPORT)

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16. Abstract

This report details the development of Michigan Department of Transportation's prestressed concrete I-beam end repair procedure and verifies its effectiveness by experimenting with a 114-mm prestressed I-beam. Due to the limited existing repair methods, various Michigan Department of Transportation personnel, along with the Federal Highway Administration (FHWA), drafted a repair procedure to be used on prestressed concrete I-beams when the reinforcing steel is exposed or unsound concrete is present. This procedure was performed in our laboratory on a salvaged prestressed I-beam. To match the in-service deteriorated prestressed I-beams, concrete was removed from around the strands and stirrups at the end of the beam. Vibrating wire strain gages were attached to the beam end to measure long term static strain and foil gages were attached at the beam end to measure dynamic loading strain. After the concrete was removed, the beam end repair was performed and the beam was load tested. From this experiment, we determined that exposing up to 305 mm of prestressing strands and 1 stirrup at the beam end does not result in significant prestress or shear strength loss.

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Prestressed Concrete Beam End Repair
(*Interim Report)*

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Testing and Research Section
Construction and Technology Division
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INTRODUCTION

The Michigan Department of Transportation's Structural Research Unit recently investigated the condition of Michigan's prestressed concrete bridge beams, as reported in Research Report R-1348, *Investigation of Condition of Prestressed Concrete Bridges in Michigan* dated 1997. From this, we discovered that most of the prestressed beams are in "good" condition with one common problem; the beam ends of prestressed I-beams are experiencing more deterioration when compared with the remainder of the beam. This condition concerns bridge maintenance engineers since roughly 60 percent (and rising) of the bridges built today are constructed with prestressed beams. Therefore, the Structural Research Unit initiated a research project to develop a prestressed concrete I-beam end repair procedure and verify its effectiveness by experimenting with a 114 mm prestressed I-beam.

For this project, we established three goals.

- The first goal was to determine how much concrete could be removed from around the strands at the beam ends before a loss in prestressing force occurs.
- The second goal was to determine how much prestressing force would be lost as a result of removing the beam end's concrete.
- The third goal was to determine if the strands should be re-stressed after repairing the concrete, and if so, how could strand re-stressing be accomplished and what force would be required.

During the first stage of this project, we performed a literature review of prior work performed by other departments of transportation. Through this review we only found one other agency, Ontario Ministry of Transportation, that has performed a similar repair. Therefore, we formed a committee of various Michigan Department of Transportation (MDOT) personnel, along with the Federal Highway Administration (FHWA), and drafted our own repair procedure for prestressed I-beams (PCI-beams) due to the limited existing repair methods.

This prestressed concrete I-beam (PCI-beam) end repair was developed for prestressed I-beam ends deteriorated to a point where the reinforcing steel is exposed. Refer to Appendix A for the standard plan sheets for PCI-beam end repair. It should be noted that sheet one involves a beam without an end block and sheet two involves a beam with an end block. If the beam end is not severely deteriorated, then (at this time) a concrete sealer will be applied to the beam at a distance not less than twice the beam depth from the end. We are currently investigating other methods that will address beam ends that are not severely deteriorated. MDOT is currently looking into applying various passive cathodic protection systems to PCI-beams.

PCI - BEAM END REPAIR

The following is a description of the PCI-beam end repair detail (shown in Appendix A) and a general step by step repair method.

The first steps in the repair are to install temporary supports and remove the existing end diaphragms to provide greater access to the beam end. With the beam end exposed, the limits of the deteriorated concrete must be saw cut at the bottom flange and the deteriorated concrete removed. Saw cutting will improve the performance of the patch material by preventing a weak feather edge. During the removal, care must be taken to avoid damaging the prestressing strands. To ensure minimal damage to the strands the following note was added to the plan sheets, “Use hand held 15 kg max. pneumatic hammer to chip beam end and remove diaphragms, except around the strands where only hand mauls or 7 kg pneumatic hammer will be allowed”. Other methods of removal are allowed if approved by the Engineer. The next step is to lightly roughen the existing sound concrete that falls within the patch limits. As previously stated, either a 15-kg maximum pneumatic hammer or other approved methods are allowed to perform this work. This will provide a better bonding surface for the Grade D Latex Modified concrete patch. To further improve the shear performance of the existing concrete to the concrete patch interface, create three 13 mm by 25 mm keyways in each side of the existing beam end. Once the existing beam end surface is prepared, core three 38 mm holes at the beam end (holes are only required if the concrete in the specified area has not been removed) to provide continuity for the diaphragms and anchorage for the reinforcing steel. With the reinforcing steel in place for both the patch and the new end diaphragms, the concrete forms can be set. The final step in the repair procedure is to place the proposed oversized repair patch and the new end block. These items should be placed monolithically to eliminate cold joints. The new end diaphragms are placed at the beam end to provide additional corrosion protection. In the future if the bridge deck expansion joint leaks, there will be a significant amount of new concrete protecting the beam’s reinforcing steel. The PCI-beam end repair detail as shown in Appendix A shows the end diaphragms 90 degrees to the beam. If the bridge is skewed, the only detail that would be modified is the degree of bend in the EB19 reinforcing bars that extend from the beam patch to the end diaphragm.

TEST BEAM DETAILS

The prestressed I-beam used for experimentation was salvaged from a high load hit at Nixon Road over US-23 (S04 of 81103) in Washtenaw County, Michigan. This type III, 114-mm prestressed beam was placed into service in 1991 and removed from service in 1994, refer to Figure 1. The in-service span length of the beam was 21 m, however we only salvaged 12 m due to damage at mid-span. In the beam, there were a total of sixteen, 13 mm, 7-wire, low relaxation prestressing strands. Eight of the sixteen strands were debonded (using two plastic tubes around the strands) at the beam end, four were debonded 1370 mm and the remaining four were debonded 2440 mm. Grade 300 MPa, #13, stirrups were spaced from the beam ends at 305 mm for the first 1120 mm and at 460 mm thereafter. During fabrication, the initial prestress was 138 kN/strand and the specified concrete compressive strength (f'c) was 35 MPa. At the time of our experiment, we found that the concrete compressive strength was closer to 61 Mpa.

TEST PROCEDURE

The first step in testing our PCI-beam end repair procedure was to apply various strain gages to the prestressed I-beam. All of the strain gages were applied to the surface of the concrete using industrial strength adhesives.

Two types of strain gages (vibrating wire and foil gages) were used. The vibrating wire strain gage is beneficial in situations requiring long term static strain. On the other hand, the foil gages are beneficial for dynamic loading. Note: the accuracy of the vibrating wire gages is ± 10 microstrain and the accuracy of the foil gages is ± 0.5 microstrain.

The first set of gages were 150 mm vibrating wire strain gages. Six gages were equally spaced between 535 mm and 1550 mm from the beam end on both sides to monitor change in the transfer length. The transfer length is the length required for a strand at the end of a prestressed member to develop the prestress force in the concrete. For the test beam, the calculated transfer length based on subsection 5.11.4.1 of the 1998 American Association of State Highway and Transportation Officials LRFD Bridge Design Specifications (AASHTO LRFD) was located 760 mm from the beam end at 60 times the strand diameter. From these vibrating wire strain gages, we monitored long term strain change in the concrete surface due to concrete removal around the strands.

In addition to the vibrating wire strain gages placed at the beam end, vibrating wire strain gages were placed at the mid-portion of the beam. This was our control point. Since no repair work was to be performed at this location, there should not be any change in strain.

The second set of strain gages were 100 mm foil gages. These were applied to monitor change in the development length of the prestressing strands. The development length is the minimum embedment length required to develop the yield force of the strand. For the test beam, the calculated development length based on subsection 5.11.4.2 of AASHTO LRFD was 1930 mm, $l_d \geq (0.15f_{ps}-0.097f_{pe})d_b$. The purpose of these gages was to monitor the continuously ramped test load response before and after the concrete repairs. Four foil gages were equally spaced between 1450 mm and 2060 mm from the point load on both sides of the beam.

Refer to Figure 2 for the locations of the vibrating wire and foil strain gages.

We also placed linear variable differential transformers (LVDTs) at the mid-portion of the beam to monitor changes in deflection of the beam as a result of prestress loss and test load.

During this study, our intent was to determine how the prestressing force within the beam related to the PCI-beam end repair procedure and how the beam responded to load after performing the repairs. To determine these relationships, the beam end underwent two repairs and three sets of load testing as described below.

INITIAL LOAD TEST

With all of the vibrating wire and foil strain gages attached, the beam was loaded using a four point loading scheme as displayed in Figure 3. Two load tests were performed to establish base line readings. The first load test was continuously ramped from 0 kN to 445 kN and returned to 0 kN, roughly half the anticipated service loading for this beam. The load rate was 445 N/sec based on American Society for Testing and Materials (ASTM) C78. This test ramped to only 445 kN due to the limitations of our load ram.

For the second initial load test, a maximum load of 670 kN, roughly 80 percent of the anticipated in-service loading for this beam, was applied in steps of 90 kN. To obtain a loading greater than 445 kN, additional load rams were used. Since these rams were hand operated, they were not included in the continuously ramped test due to the difficulty of obtaining a uniform load rate.

Both tests were used to determine if the development length was affected. By using the strain obtained from the foil gages as the base line, we can compare strain from the successive load tests. A shift in strain will relate to a shift in the development length.

FIRST BEAM END REPAIR

Once the initial tests were performed, portions of the beam end were removed to match similar in service deteriorated PCI-beams, refer to Figure 4. For this test, 130 mm of the prestressing strands and web were exposed at the beam end by removing the concrete with a 7-kg jack hammer. The 7-kg jack hammer was used to minimize the damage to the prestressing strands. Then the surface of concrete within the patch limits was lightly roughened with the 7-kg jack hammer so that the new concrete would bond to the exposed aggregates. Two different jack hammer bits were used so we could compare the effects of one to the other. The north half of the beam end was roughened with a chisel bit, while the south half was roughened with a bush hammer bit. Prior to placing the repair patch the exposed surface was blasted with oil-free compressed air to remove surface laitance and prewetted in preparation for the Grade D latex-modified material. After the patch concrete cured to the design strength, the beam was load tested as previously described for the initial test.

Upon the completion of the two load tests, we performed a bond tension test between the existing concrete surface and the Grade D latex-modified patch. On each side of the beam end, two 50 mm diameter cores through the patch were taken to determine the bond strength.

SECOND BEAM END REPAIR

The second beam end repair began after the removal of the first patch. Other than having the concrete surface roughened, our second test procedure was similar to the first with the one exception being that 305 mm of the prestressing strands and web were exposed, refer to Figure 5.

RESULTS

During the initial load test, two shear cracks formed at 150 kN of applied load and prior to reaching the design shear strength of the concrete according to AASHTO LRFD subsection 5.8.3.3. One crack width, measured under 150 kN of load applied, was 0.8 mm on the north side of the beam and 1.3 mm on the south side. These cracks formed at an angle of 32 degrees and 40 degrees, respectively, both which are greater than the calculated crack angle of 27 degrees. An increase in the crack angle relates to a decrease in shear strength because as the crack angle increases, the applied force is distributed over a smaller area.

An additional shear crack developed during the first beam end repair load test. The location and width of the crack were recorded and measured. In addition, the shear cracks that were present from the initial load test opened wider, 1.3 mm on the north side and 1.5 mm on the south side under 150 kN applied load, during the first beam end repair load test. For the second beam end repair load test, another shear crack developed near the previous cracks and the shear cracks that were present from the initial load test were 1.3 mm on the north side and 1.5 mm on the south side of the beam under 150 kN applied load. Refer to Figure 6 for the location of the cracks.

Analyzing the beam's shear strength at 2440 mm from the beam end according to AASHTO LRFD section 5.8.3.3, the beam has an ultimate shear strength of 700 kN and a concrete shear strength of 530 kN, both are more than the total applied shear load of 390 kN determined at 2440 mm from the beam end. The total applied load includes the weight of the steel I-beam used for loading as well as the dead load weight of the prestressed I-beam. Since the beam theoretically should not have cracked given the applied load, we investigated factors that could explain the cracks. Using the measured crack angles, the total losses that would have to occur to reduce the shear strength of the concrete to 390 kN (the total applied load) would need to be 520 MPa not the 301 MPa used during design. The increase in prestressing force loss could be explained by the high load hit, which caused extensive damage. During the high load hit, the vehicle's impact severed several prestressing strands in the bottom flange. The large vibrations imposed into the beam and the lateral force pulling the prestressing strands could have caused portions of the prestressing strands to debond and lose prestressing force.

Using the design prestressing force in the strands and AASHTO LRFD, it was determined that at 2130 mm from the beam end, d_v, there is inadequate longitudinal reinforcement to resist the applied force and keep a shear crack closed once it forms. This was evidenced by our laboratory tests. The location of d_v was chosen according to AASHTO LRFD section 5.8.3.2 because at this distance from the beam end the reaction force in the direction of the applied shear will not introduce compression into the region.

Throughout this experiment, no measurable long term static strain change occurred at the beam end. Figures 7 and 8 displays the variability of the 14 vibrating wire gages. From this, it can be seen that the range of readings remained fairly constant throughout the monitoring until the temporary support was moved on August 7, 1998. The vibrating wire gage readings as shown in Figures 7 and 8 were adjusted from their original readings to account for inadvertent debonding of the gages and unexplainable erratic readings. The base lines for each gage were also adjusted so all of the gages could be shown on the same graph for comparison.

Minor changes in strain were observed in the foil gages during the continuously ramped load test. These changes were discovered by comparing the slope of the strain verses load graph, refer to Figure 9. The initial test slope was flatter than the first and second beam end repair test slope. This slope change may relate to a prestress loss. During the initial load test, the instrumented area had a high prestress (compressive) force and as the beam is loaded this compressive force restrains some tensile force. As prestress (compressive) force decreases, there is less compressive force to restrain the tensile force. Assuming that strain is uniform through uncracked concrete, the strain at the concrete surface can be correlated to the strain in the prestressing steel. Following this line of thought, in Figure 9 we see a prestress loss of about 14 micro strain, which translates to roughly a 29 MPa (½ percent loss) at the location of foil gage #1. Refer to Table 1 for change of strain at different gage locations.

From the LVDTs placed at the mid-portion of the beam, we found that the beam deflected 7.6 mm more after the placement of the first patch. The beam deflected an additional 1.3 mm after the placement of the second patch. The deflection measurements were taken under an applied load of 150 kN. The increase in deflection can be explained by formation of cracks in the beam during the load tests. As more cracks form, the beam becomes less stiff resulting in more deflection.

Tensile bond pull-off tests were performed between the existing concrete and the patch material. The first patch had two pull-off values, 752 kPa and 959 kPa, with an average bond strength equal to 856 kPa for the north side roughened with a 7-kg jack hammer and a chisel bit. We were unable to get accurate pull-off values from the south patch prepared with a 7-kg jack hammer and a bush hammer bit. For the second patch, we obtained two equal pull-off values of 841 kPa for the north side and two pull-off values, 1317 kN and 1669 kN, with an average bond strength equal to 1493 kPa for the south side. During all tests, the bond broke within the existing concrete. However, the depth of the break was just beneath the bond line. We had the fracture zone analyzed by our Petrography Unit and from this analysis it was determined that the bond broke within the micro cracked (bruised) area of the existing concrete. This area became bruised due to the hammering of the surface with the jack hammer. From our results, it appears that the bush hammer bit created less bruising than the chisel bit. Our desired bond strength was 1380 kPa.

PCI-I BEAM REPAIR

Performing the repair procedure as detailed in Appendix A, we discovered the following items worth noting.

The stirrups at the end of the beam may be cut as a result of coring the three holes for the reinforcing steel. This should not be of major concern because the amount of concrete and steel being placed for the patch will account for any loss if the end stirrup were cut. Additionally, the reaction at the beam end introduces compression into the end region of the member. Therefore, the tensile requirements of the end stirrup are negligible.

The Grade D latex modified concrete had a compressive strength (f'c) of 55 MPa after 22 days. This strength exceeds the existing PCI-beams that have a f'c of 34 MPa to 48 MPa.

When the existing concrete surface is lightly roughened using a 7-kg jack hammer the concrete just below the surface becomes bruised. Therefore, it may be advantageous to require the contractor to use other methods to lightly roughen the surface, i.e., hydro blast.

CONCLUSION

We determined that exposing up to 305 mm of prestressed strands at the beam end does not result in significant prestress loss. Therefore, there is no need to re-stress the strands prior to placing the repair.

FUTURE WORK

The Structural Research Unit will perform detailed field reviews of three structures for a period of one year after the PCI-beam end repair procedure has been performed. The repair will be closely inspected throughout construction and then bimonthly thereafter. During our field inspections, we will review the constructibility of the repair detail and look for the initiation and/or growth of cracks along with patch delaminations. The structures to be monitored are S05 and S11 of 33171 (NB and SB US-127 over Vine Street) and S07 of 47014 (NB and SB US-23 over Center Road). Upon completion of the field monitoring, a final report will be prepared summarizing our findings.

FIGURES

APPENDIX