Destructive Testing of Deteriorated Prestressed Box Bridge Beam

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A deteriorated prestressed box bridge beam was tested to destruction to determine the effects of deterioration on prestressed beam performance. Three prestressing tendons in one corner of the beam had corroded causing spalling of that corner. One of the tendons had broken and the other two were badly corroded, so only 15 tendons were effective. The resulting tendon pattern was asymmetric. A destructive test was conducted by loading the beam with two point loads. For comparison purposes, an undamaged beam with all 18 tendons intact was cast and tested. The undamaged beam held an applied moment of 2720 kN-m (2005 kip-ft) and did not fail. The deteriorated beam failed suddenly at a total applied moment of 1805 kN-m (1310 kip-ft). This reduction in moment capacity is not totally attributable to loss of tendons or cross section. The deteriorated beam also showed less deflection capacity at the midspan [270 mm (10.7 in.) versus 432 mm (17 in.)], more lateral deflection [28 mm (1.1 in.) versus 0 mm] and more web cracking than the undamaged beam. The final failure of the deteriorated beam appeared to be a lateral instability, which resulted in the sudden collapse of the beam. This lateral instability was caused by the lateral bending and yielding of the steel. It was also found that the AASHTO Code was not conservative for the deteriorated beam. The applied failure moment was 8 percent lower than that predicted by the AASHTO Code.

Bridge elements can be damaged or show signs of deterioration, or both, as a result of traffic and environmental conditions. Engineers frequently are required to determine whether a damaged or deteriorated element may be left in service or should be repaired or replaced. It is difficult to evaluate the strength and serviceability of deteriorated members because clear guidelines and methods often do not exist.

Determination of the strength of a deteriorated prestressed beam is particularly difficult. Deterioration or damage often causes a lack of symmetry in both the cross section and the steel tendon pattern. Under load, the lack of symmetry may cause lateral bending or torsion, or both, which may induce undesirable stresses. The asymmetrical strand pattern of a damaged or deteriorated beam makes evaluation more difficult because there are no standard or simple methods for analysis of asymmetrical prestressed beams. All the usual methods of analysis for prestressed members assume symmetry of the tendon pattern because prestressed beams are almost always manufactured as symmetrical sections to avoid the out-of-plane bowing caused by asymmetry.

There may also be a loss of capacity due to deterioration of the concrete. There is little information on the effect of material damage on the behavior of prestressed members. Therefore any loss of capacity due to material degradation cannot be easily quantified.

One way to determine the effect of deterioration on a prestressed concrete beam is to test a deteriorated beam to destruction while carefully monitoring the response. Information from such a test can then be used to evaluate the various methods of determining the strength and behavior of deteriorated prestressed concrete beams.

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SIGNIFICANCE OF RESEARCH

Prestressed box beams can be damaged by vehicle impact or deterioration mechanisms such as corrosion. Often, the damage will cause a loss of cross section and there may be broken or damaged tendons. One consequence of the damage is that broken tendons result in a loss of prestressing force, which may cause the beam to crack under service loads. There also will be a reduction in ultimate moment capacity due to loss of cross section and tendons. The damaged beam will have an asymmetrical cross section and tendon pattern, and this lack of symmetry may cause lateral bending stresses or torsional stresses under vertical load, or both, which may further reduce the member capacity. Finally, material damage may cause a loss of capacity by limiting material response. By load testing an asymmetrically deteriorated box beam it is possible to evaluate the effect of the damage, loss of cross section, loss of tendons, and loss of symmetry on the beam behavior.

PREVIOUS RESEARCH

Little work has been done on load testing damaged or deteriorated prestressed beams. Shenoy and Frantz (*1*) tested prestressed box beams removed from a bridge. However, these beams were only lightly deteriorated and no tendons were broken or damaged so the beams remained symmetrical. Shenoy and Frantz concluded that, even though slightly damaged, the beams remained sufficiently strong and ductile and that current analysis methods were adequate.

Olson (*2*) tested four 20-year-old AASHTO Type III girders that had been removed from a bridge in Minnesota. These beams were not damaged when removed but were damaged as part of the experimental program to test repair techniques. One beam was tested undamaged, and another was left damaged and was tested without repair as a baseline. (The remaining two beams were damaged, repaired, and then tested.) The undamaged beam was tested under fatigue loading and then tested to failure. Testing of the damaged beam consisted of severing two (of 30) strands on one side of the beam (creating an asymmetrical section) and applying fatigue loads. Fatigue tests were repeated after severing a third and then a fourth strand on the same side of the beam. The beam was then load tested to destruction. The results of Olson's study have three important points. (*a*) Olson observed that the final static load applied at failure was 29 percent lower than that of the undamaged beam. [The flexural capacity was calculated using the 1989 AASHTO Code (*3*) and ignoring the asymmetry in the beam. It was found that loss of tendons results only in a 15 percent calculated reduction in failure load capacity. Clearly the loss of strength was not only a result of tendon loss.] (*b*) Olson noted that on the damaged side, cracks formed and these cracks propagated back toward the supports. (*c*) Olson also noted that the bottom flange of beam on the damaged side "peeled away" from the web. The impact of these results on this study will be discussed later.

DESCRIPTION OF BEAM

The deteriorated test beam had been a sidewalk support beam in a bridge over the Maumee River in Defiance, Ohio. Because it was a sidewalk beam protected by a high curb, it is unlikely to have seen significant service loads beyond its own dead load. Cast in 1980, the beam was a box section 23.3 m (76 ft 6 in.) long, 0.91 m (36 in.) wide, and 0.84 m (33 in.) high with walls 127 mm (5 in.) thick. (Figure 1). Originally, the beam had 18 prestressing tendons 13 mm (½ in.) in diameter with each tendon having an area of 99 mm^2 (0.154 $in.^2$). At the time of the tests, the prestressing tendons in one corner of the beam had corroded (Figure 2) causing spalling of that corner. In the deteriorated areas, the damage was not uniform along the length and the worst visible damage to the beam occurred 7.6 m (25 ft) from one end of the beam (Figure 2). Examination of the deteriorated corner revealed that one tendon was broken and was missing along almost the entire length of the beam. Two other tendons were still present but were badly corroded. In one corroded tendon, the individual strands were broken at various places. The other corroded tendon was still substantially intact and was still embedded in the concrete in some places. It is not known if either of these corroded tendons was effective. Therefore, it is possible to definitely assume that only 15 of the tendons are still effective, although the test results showed that perhaps one of the corroded tendons also carried some load.

The damage to the beam was first noted during a routine annual inspection of the bridge in the summer of 1989. Since the damaged tendons were in one corner of the box, the remaining tendons in the beam had an asymmetric pattern and the prestressing force became eccentric, which caused a lateral moment in the beam. The deteriorated beam had been tied to another sidewalk support beam as required in the original plans. Before removal, the lateral bending would have been restrained by the attachment to the other sidewalk beam. Therefore, the amount of lateral bending and the associated stresses before removal are not known. Removal of the beam from the bridge occurred in summer 1990. After being removed, the beam was stored until it was tested in summer 1992. The presence of the lateral moment caused by prestressing force eccentricity caused an out-of-plane bowing of the beam that was measured to be about 13 mm (½ in.) at the time of testing.

Originally the beam was designed using 38.5 MPa (5,500 psi) of concrete and 1890 MPa (270 ksi) of prestressing steel. At the time of testing, the concrete was approximately 12 years old. Cores taken after the destructive static test indicated that the concrete strength was approximately 56 MPa (8,000 psi). Tests on the prestressing tendons showed a yield strength of 1645 MPa (235 ksi) and an ultimate strength of 1800 MPa (260 ksi).

To accurately assess the effect of damage on the beam, it was desirable to test an undamaged version of the test beam. Because no such beam was available, an undamaged beam having the same length as the deteriorated beam and the undamaged cross section shown in Figure 1 was cast. Cylinder tests indicated that this beam also had a concrete compressive strength of 56 MPa (8,000 psi) at the time of the test, 21 days after casting. The prestressing steel had a yield strength of 1740 MPa (250 ksi) and an ultimate strength of 1880 MPa (270 ksi).

TESTING SYSTEM

The static, destructive test was conducted on the beams at the ESSROC Prestressed Concrete Products manufacturing facility in Melbourne, Kentucky. This site was chosen because of the presence

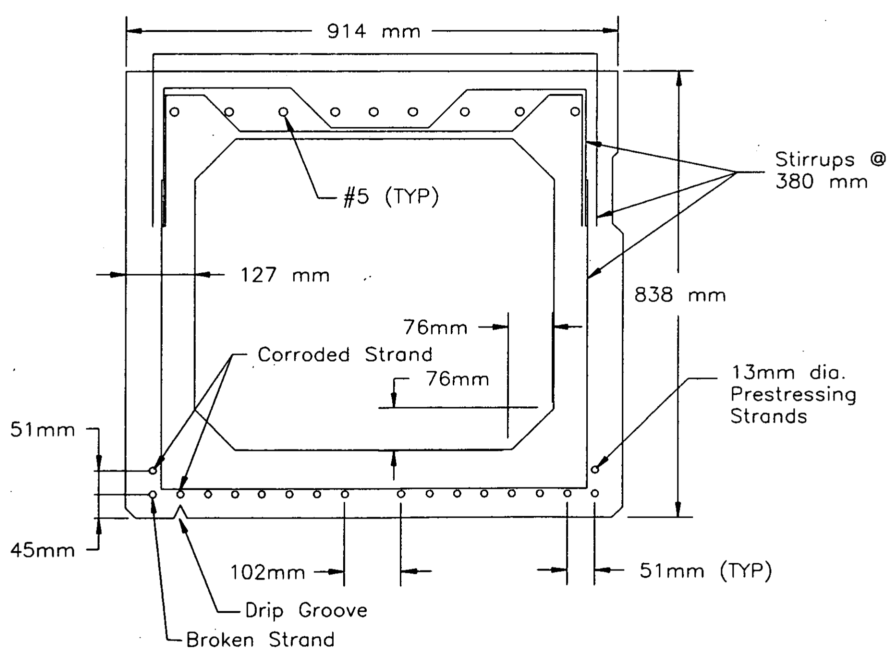

FIGURE 1 Cross section of original beam.

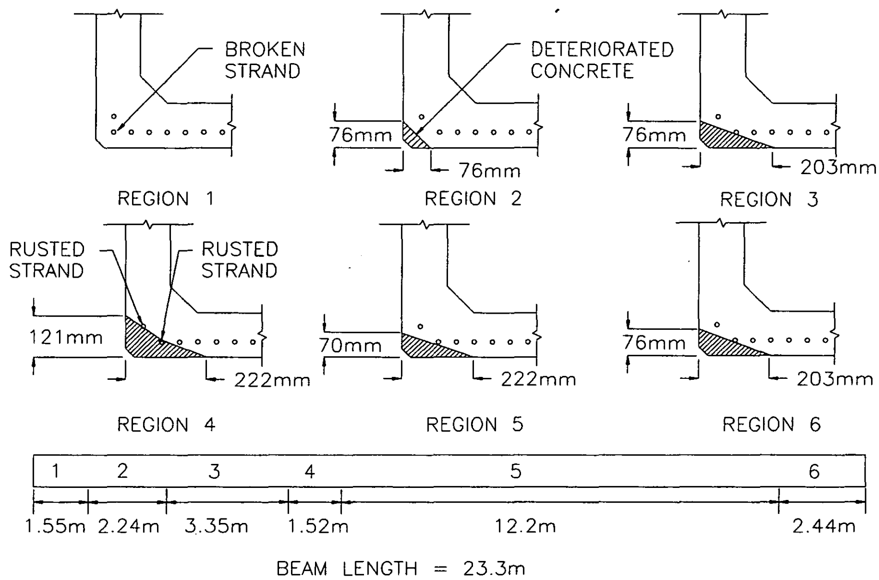

FIGURE 2 Map of damage to deteriorated beam.

of an existing foundation that could be used for securing two testing frames and because equipment for casting the undamaged beam and moving both the deteriorated and undamaged beams was readily available. The beams were loaded with two point loads placed 6.4 m (21 ft) apart, 8.5 m (28.8 ft) from either end. This load position was dictated by the position of existing tie-down plates for the testing frames. Two steel testing frames were fabricated to apply the loads (Figure 3).

Concrete end blocks were cast to provide beam end supports. These blocks were designed to simulate the actual end condition of the original bridge (Figure 4). The beam was doweled into the end blocks, but rather than grouting the dowels, the area around the dowels was packed with wet sand. This was done to allow removal of the beam from the supports after the test.

The Ohio Department of Transportation (ODOT) engineers desired to match field conditions as closely as possible during the test. Because the test beam was an edge beam, the lateral deflection in one direction would be constrained by the presence of other bridge beams. Therefore, during the test a "bumper beam" was placed beside the test beams. For the deteriorated beam, the bumper beam was placed on the side away from the damage since the damage actually occurred on the outside edge of the bridge. The presence of this bumper beam had no effect on the test because neither beam ever touched it. It prevented mapping of the cracks on one side of the beam.

FIGURE 3 Testing frame.

Loads were applied using two capacity servo-controlled hydraulic actuators 1.5 MN (350 kips). A digital controller was used to control the cylinders. The undamaged beam was tested in load control. However, by the time the deteriorated beam was tested the capability for displacement controlled testing had been developed and the test of the deteriorated beam was conducted in displacement control. The system was capable of controlling only one displacement, so a master/slave configuration was used. In this method, the displacement under one load point was used for control and the hydraulic cylinders were linked so that the system supplied the same pressure (load) to each cylinder.

A clevis was installed on the end of each cylinder to transfer the load to the beam. These clevises had bearing plates that were 480 mm (18 in.) square. This size bearing plate spread the load enough to prevent local failure of the box beam top flange. Load was transferred from the clevis plate to the beam by an elastomeric pad 480 mm (18 in.) square to ensure even application of the load.

The undamaged beam was loaded in 18-kN (4-kips) increments and at various points and then unloaded and reloaded so that changes in stiffness could be monitored. The deteriorated beam was loaded in 13-mm (½-in.) displacement increments. At various displacement levels this beam was also unloaded and reloaded. For both beams, after each application of a load or displacement increment, the test was paused and the beam was inspected for cracking. The cracks were marked on the beams.

INSTRUMENTATION

The vertical and horizontal displacements and the angle of twist were measured by wire potentiometers arranged as shown in

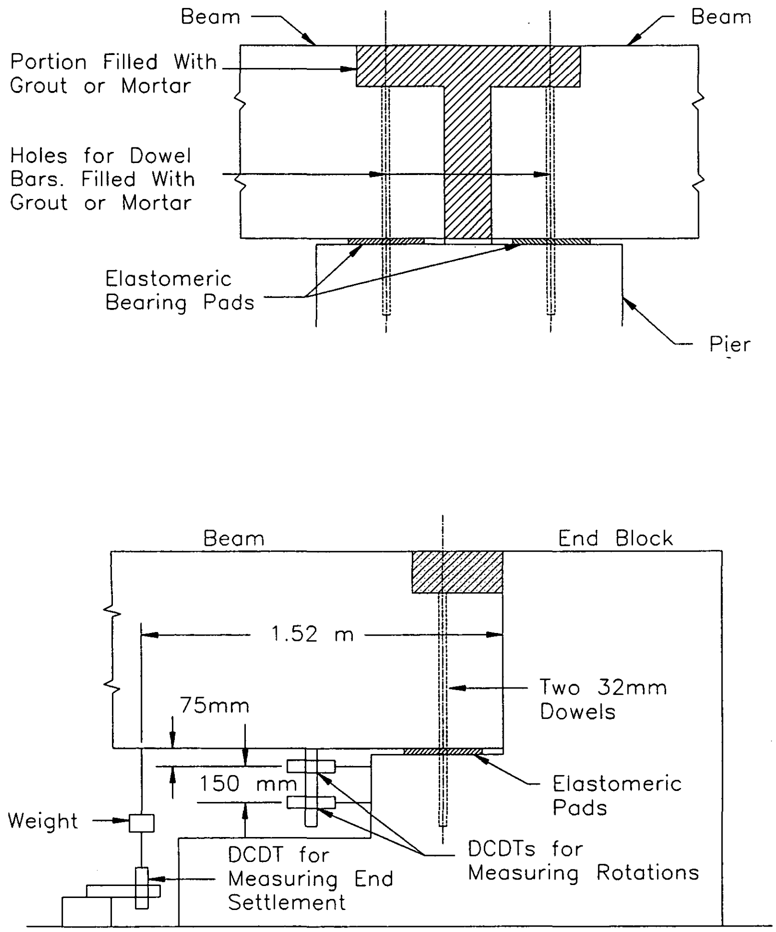

FIGURE 4 Support condition on original bridge and support block and instrumentation for test.

Figures 5 and 6. These instruments were chosen because they had a range sufficient to measure the large beam deflections. Each wire potentiometer had a range of either 254 or 380 mm (10 or 15 in.). Because of the large deflections in both the vertical and lateral directions and because the corners of the beams were moving in two directions at once, the wire potentiometer could not measure vertical and lateral deflection directly. The wire potentiometers basically measure the change in the length of the wire running from the instrument to the beam, so that at any time the distance from the wire potentiometer to the corner of beam was known. Because the distance between wire potentiometers was known, the deflection of the beam in the vertical and lateral directions could be calculated by triangulation. Theoretically, only three wire potentiometers are needed at each point to determine the deflection and rotation of the beam, but five were used to provide for averaging and redundancy.

Linear variable differential transformers were used to measure deflections (support settlement) and rotations of the beam ends as shown in Figure 4. Steel strains were measured by strain gauging the steel tendons. Where necessary, holes were cut into the concrete to expose the prestressing tendons (no holes were necessary where the tendons had been exposed by deterioration) (Figures 5 and 6). Because the tendons were made of seven individual strands, strain gauges were attached to two of these individual strands to measure the steel strain. Strain gauges were installed on the concrete surface to measure concrete surface strains, but several of these gauges failed. However, some concrete strain data were obtained for the deteriorated beam.

TEST RESULTS

The undamaged beam was tested first at an age of 21 days. A plot that shows load versus midspan deflection is indicated in Figure 7. The first cracks were observed at an applied load of 135 kN (30 kips) at each of the two loading points. The test was stopped at an applied load of 258 kN (58 kips) at each load point and a midspan deflection of 432 mm (17 in.) because the deflection capacity of the test frame had been exhausted (i.e., the beam touched the ground). Test results showed that the steel had yielded, but no strands were ruptured. No lateral deflection was noted. Typical crack patterns (Figure 8) show flexural cracks with the characteristic "forking" at the top. In all, the test of the undamaged beam yielded results that were consistent with those of published tests of box beams (*1*).

The results of the deteriorated beam test were different. As loading began, lateral deflection in the direction of the damaged side of the beam was observed. The first cracks occurred at an applied load of 107 kN (24 kips) at each point. Once the beam cracked, the vertical deflection increased rapidly and the load-versus-vertical-

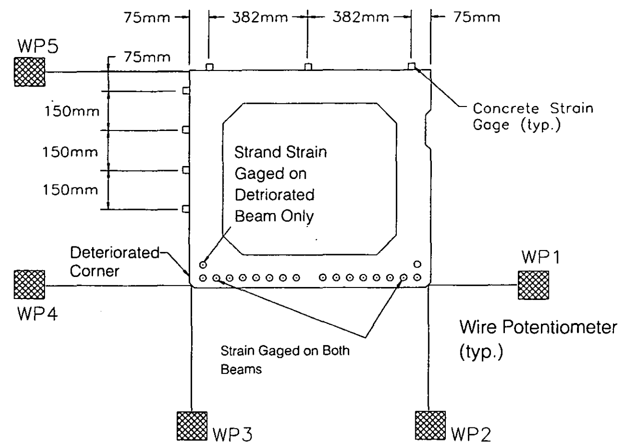

FIGURE 5 Position of instruments on cross section.

midspan deflection curve became flatter than that of the undamaged beam (Figure 7). The lateral deflection also increased significantly after cracking (Figure 9). It was noted that the initial flexural cracks propagated much higher into the web of the deteriorated beam than into the web of the undamaged beam (Figure 10).

At an applied load of 120 kN (27 kips) at each point, the corroded tendon in the bottom layer of steel (Figure 2) ruptured. The other corroded tendon (in the upper layer) remained partially bonded in the concrete but appeared to be pulling loose as the deflection increased. As seen in Figure 2, this tendon was partially exposed in some areas and nearly completely embedded in other areas. As the tendon began to pull loose it caused additional spalling and cracking in the areas where the tendon was still mostly embedded. The most severe spalling occurred near one of the loading points (Figure 10). Under an increasing load, cracks propagated from the area end back toward the support (and the load point) in a fan shape (Figure 10). This spalling and cracking associated with the pull-out and debonding of the corroded tendon seems to indicate that the tendon carried some force during loading. As a result, an assumption that this tendon is ineffective would be conservative.

As the load increased, the flexural cracks propagated high into the web and began to propagate into the top flange (Figure 10). The cracks in the deteriorated beam also are much farther apart than those in the undamaged beam, probably because most of the prestressing steel under this web is missing and the remaining steel is not sufficient to properly distribute the cracks.

At an applied load of 147 kN (33 kips) per load point, the beam failed suddenly and collapsed. At the time of failure, the beam had deflected 270 mm (10.7 in.) vertically and an additional 28 mm (1.1 in.) laterally (the beam already had a 13-mm (½-in.) lateral deflection before the test began).

ANALYSIS OF RESULTS

Figure 7 shows that, before it reaches the cracking load, the deteriorated beam is less stiff than the undamaged beam, but the difference is small. The small difference in stiffness is not unusual because precracking stiffness is largely influenced by gross cross-section properties, and the loss of gross cross-sectional area for the deteriorated beam was small compared with the total gross cross-sectional area.

The cracking load of the deteriorated beam was lower than that of the undamaged beam. Calculations show that the lower stiffness and cracking load of the deteriorated beam can be explained by accounting for the loss of cross section and the loss of prestressing

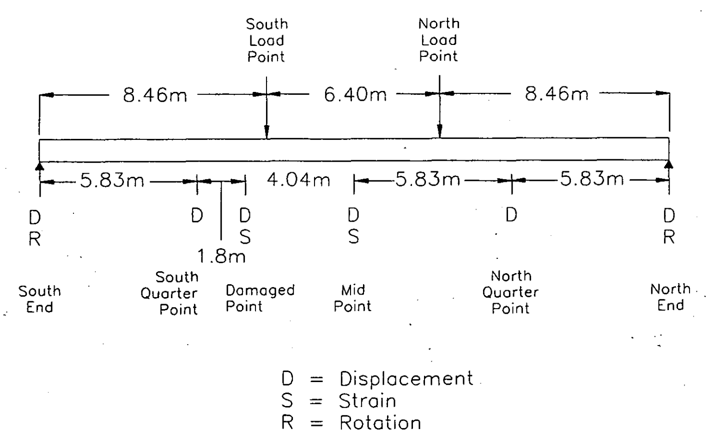

FIGURE 6 Position of instruments along length of beam.

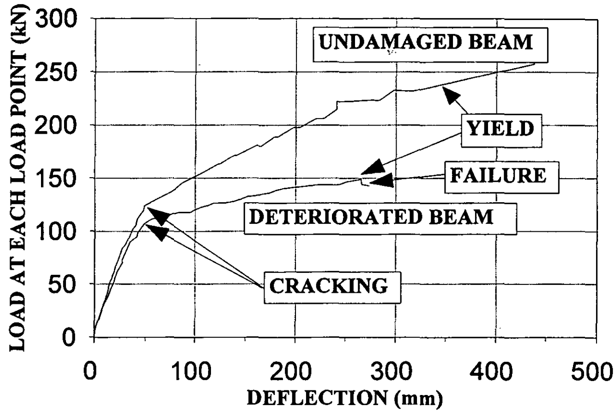

FIGURE 7 Load versus midspan deflection for deteriorated and undamaged beams.

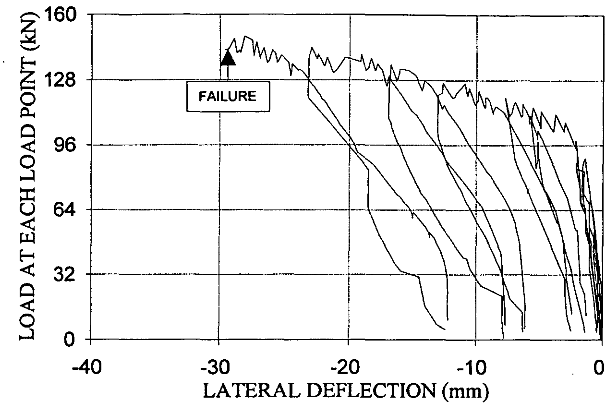

FIGURE 9 Load versus midspan lateral deflection for deteriorated beam.

force in the tendons. The loss of prestressing force was estimated using the provisions of the 1989 AASHTO Code (*3*). These losses were estimated at 10 percent for the undamaged beam and 18 percent for the deteriorated beam. It was the intent of the investigators to determine the actual loss of prestress in the deteriorated beam by strain gauging and severing some of the tendons after the test was complete, but the catastrophic failure of the beam made this impossible.

The loss of prestressing force will be affected by the lateral bending moment caused by the asymmetry of the beam. Once removed from the bridge, the beam was free to deflect laterally because of the eccentric prestressing force. The lateral deflection will increase over time as a result of creep (because the beam was 10 years old at the time of removal, shrinkage was ignored since most of the shrinkage had already occurred). The lateral deflection will cause tendons away from the damaged web to lose prestressing force because they are on the "compressive" side for lateral bending. By the same argument, tendons near the damaged web may gain prestressing force because they are on the "tensile" side. Calculations of these changes in prestressing force, assuming that 50 percent of the ultimate creep had occurred, showed that the prestressing force in a tendon may change by less than 2 percent of the original prestressing force, at most. Also, because the lateral bending is parallel to the tendon line, the change in prestressing force varies linearly along the cross section. When the average loss of prestressing force for the tendon group as a whole was calculated, it was found to be negligible.

The moment of inertia of the deteriorated beam was calculated by assuming the loss of a triangular section 220 $\times$ 120 mm (8.7 $\times$ 4.7 in.) from the lower corner of the beam. This roughly corresponds to the damage in Region 4 in Figure 2. Using calculated prestressing losses and the normally assumed values of E ($4700 f_c'^{0.5}$ MPa or $57000\ f_c'^{0.5}$ psi) and f_r ($0.63\ f_c'^{0.5}$ MPa or $7.5\ f_c'^{0.5}$ psi), the calculated cracking moment was found to be 1410 kN-m (1035 kip-ft). The actual cracking moment of the deteriorated beam was 1440 kN-m (1060 kip-ft) (applied load + beam weight). The calculated cracking moment is in reasonable agreement with the experiment. Note that measured material properties were used in the calculation. For the undamaged beam, the calculated cracking moment was 1590 kN-m (1170 kip-ft) compared with the measured value of 1625 kN-m (1195 kip-ft), again showing reasonable agreement.

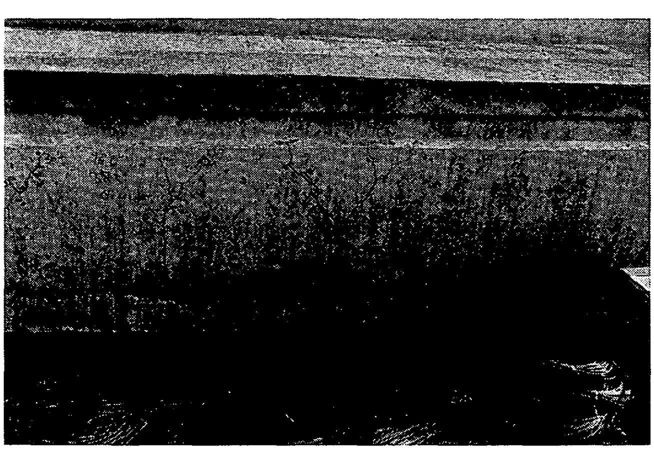

FIGURE 8 Crack pattern for undamaged beam (note: failure notation in photo is incorrect).

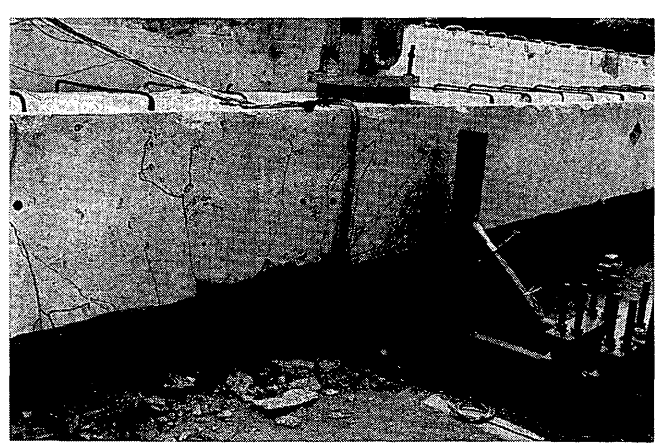

FIGURE 10 Crack pattern for deteriorated beam just before failure.

In the postcracking region, the deteriorated beam was significantly less stiff than the undamaged beam (Figure 7). This lower stiffness was caused by the loss of tendons and because once the beam cracked the deteriorated beam showed significantly longer cracks on the damaged web. Because there was no longitudinal mild reinforcing in the web and because three of the prestressing strands under the web were missing, there was little to control web cracking caused by the increased tensile stresses from the lateral moment. This can be seen by comparing the crack patterns for the two beams (Figures 8 and 10). Under a load of 258 kN (58 kips) per point, the cracks in the undamaged beam had propagated only 660 mm (26 in.) from the bottom of the beam. The deteriorated beam had crack lengths that measured 760 mm (30 in.) from the bottom of the beam under a load of 147 kN (33 kips) per load point. The cracks in the deteriorated beam were long enough to penetrate the top flange. It therefore appears that the lateral bending caused much more cracking in the damaged web (note that cracking in the undamaged web could not be observed because of the presence of the bumper beam). This cracking would have reduced the postcracking stiffness. The previously noted cracking caused by pullout of one of the corroded tendons also would have reduced the postcracking stiffness.

It is also of interest to determine whether the provisions of the 1989 AASHTO Code (*3*) will reasonably predict the ultimate moment capacity of the undamaged and deteriorated beams. The calculated capacity of the undamaged beam, using measured material properties, was found to be 2360 kN-m (1735 kip-ft). During the experiment, a total moment (applied load + beam weight) of 2720 kN-m (2005 kip-ft) was applied to the beam and the beam did not fail. This illustrates the conservative nature of the AASHTO Code.

For the deteriorated beam, the AASHTO Code is not conservative. Using the actual material properties and assuming 15 strands to be effective, the calculated moment capacity is 1950 kN-m (1435 kip-ft). The actual failure moment (applied load + beam weight) was 1805 kN-m (1310 kip-ft), about 8 percent below the calculated moment. Note that the calculation of ultimate moment was a lower bound because it assumed that both corroded tendons were ineffective; however, the experimental evidence indicates that one of the corroded tendons may have been at least partially effective. Also note that, since bending capacity is only slightly affected by the concrete compressive strength, the calculated ultimate moment hardly changes if the design concrete strength of 39 MPa (5,500 psi) is used in place of the actual compressive strength of 56 MPa (8,000 psi).

The mode of failure is of great concern. The deteriorated beam failed suddenly as opposed to the undamaged beam, which showed ductile behavior. The exact cause of the final failure is not certain because it occurred suddenly; however, there is a reasonable possibility that the failure is linked to the lateral bending. It was noted that strain in the prestressing steel of the damaged beam was measured at approximately 0.005 at the time of failure. If it is assumed that the prestressing steel was originally stressed to 0.7 f_y (as required by the specifications) and the prestressing losses [calculated by the provisions of the 1989 AASHTO Code (*3*)] were 18 percent, the strain in the steel before the test began would also have been about 0.005. Thus, the total strain (prestressing + applied load) would be approximately 1 percent, which is usually taken as yield in the prestressing steel. It is believed that once the prestressing steel yielded, it was unable to restrain the lateral bending of the beam. Because there was no mild steel in the damaged web and the damaged web was already extensively cracked because of lateral bending, there were no additional mechanisms to prevent a lateral instability. It is therefore believed that the final failure occurred because the beam became laterally unstable. Note that the failure was not a compressive failure since the maximum measured compressive strain in top flange, measured over the undamaged web, was 0.0014—well below the crushing strain.

COMPARISON WITH PREVIOUS TESTS

Comparisons with the box beam tests of Shenoy and Frantz (*1*) show that the effect of losing tendons and cross section is severe. The beams tested by Shenoy and Frantz had no missing tendons or significant loss of cross section. These beams were ductile and had strengths that exceeded the predicted values. In contrast, the beam tested in this work was not ductile and showed values of strength that were lower than predicted.

When comparing this test with that of Olson (*2*), there are several similarities. As previously noted, comparisons of the failure loads of Olson's damaged and undamaged beam showed that the damaged beam failed at a much lower live load and that the reduction in live load capacity cannot be easily explained by loss of tendons. Similar results were obtained in this study. A comparison of the load/deflection for Olson's beams reveals that the damaged beam also was much less stiff in the postcracking region. Olson gives no data about lateral deflections. Finally, Olson noted that the damaged side of the beam had cracks that propagated back toward the supports and that the bottom flange on the damaged side of the beam appeared to "peel away" from the web. These were believed to be caused by tensile stresses from the lateral moments caused by the lack of symmetry. Olson reported a compressive failure of the beam, and the effect of any tensile stress generated by the asymmetric nature of the cross section on the I-girder is not clear from Olson's report.

CONCLUSIONS

1. A prestressed box beam that had lost 3 of 18 tendons to corrosion was tested to failure. For comparative purposes, a similar, undamaged beam was also tested. The deteriorated beam exhibited a slightly lower precracking stiffness when compared with the undamaged beam. The deteriorated beam also exhibited a lower cracking load. However, the lower precracking stiffness and lower cracking load in the deteriorated beam can be explained by accounting for loss of prestressing force, loss of cross section, and loss of tendons in the deteriorated beam.

2. The deteriorated beam showed significant lateral deflection under load. This lateral deflection was caused by the fact that the deterioration caused a lack of symmetry in the tendon pattern and concrete cross section. Before testing, the beam exhibited 13 mm (0.5 in.) of lateral deflection because of the asymmetry of the cross section. At failure, the beam had deflected an additional 28 mm (1.1 in.) in the lateral direction. This lateral deflection is believed to have significantly influenced the postcracking and failure behavior.

3. In the postcracking range the deteriorated beam was much less stiff than the undamaged beam. Some of this loss of stiffness is attributable to the loss of three tendons. However, the lateral moments caused by the lack of symmetry raised the tensile stresses in the damaged web. As a result, when the deteriorated beam was compared with the undamaged beam, it was found that the cracks

in the damaged web propagated further into the web. The cracks in the deteriorated beam were also spaced further apart because there was no prestressing steel in this area to distribute the cracking. Additional cracking also occurred because a corroded tendon had pulled out. The additional cracking caused by lateral bending and tendon pull-out contributed to the reduction in postcracking stiffness.

4. The undamaged beam held a total moment (applied load + beam weight) of 2720 kN-m (2005 kip-ft) and did not fail. The deteriorated beam failed suddenly at a total moment of 1805 kN-m (1310 kip-ft). The lower failure moment of the deteriorated beam is not totally attributable to the loss of tendons and cross-section area.

5. The ultimate moment for the deteriorated beam was 8 percent lower than that predicted by the AASHTO Code, showing that the AASHTO Code was not conservative for the deteriorated beam. Of more importance is that the deteriorated beam failed suddenly.

6. The final failure of the deteriorated beam was a sudden collapse of the beam. It is believed that the lateral bending contributed to this failure. At the time of failure, strain gauges on the prestressing steel showed that the steel had just reached yield. It is believed that as the steel yielded, the beam became laterally unstable and failed. Normally, box beams in bridges are tied together by transverse posttensioning. Because lateral deflection contributed significantly to the failure of the beam, this transverse posttensioning may help prevent premature failure of deteriorated box beams.

ACKNOWLEDGMENTS

This research was funded by ODOT and FHWA. The authors thank William Edwards, Vik Dalal, David Hanhilammi, Berkeley Hill, and Maher Girges, all from ODOT, for their support in this project. The authors also thank ESSROC Corp. of Melbourne, Kentucky, and Prestressed Services of Decatur, Indiana, for their assistance with this project.

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Publication of this paper sponsored by Committee on Dynamics and Field Testing of Bridges.