TEST TO FAILURE OF A 54 FT. DETERIORATED PRETENSIONED PRECAST CONCRETE DECK BEAM

by

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### Test to Failure of a 54Ft. Deteriorated Pretensioned Precast Concrete Deck Beam

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**Abstract:**
A 54-ft. long, 27 inch deep, deteriorated pretensioned deck beam was taken from an existing bridge and tested to failure under equal loads applied 7-ft either side of its midspan. A systematic visual condition assessment was made of the beam before testing. After testing, the concrete cover over the strands was removed and their condition assessed. The beam was sawn in half at the failure location and the section geometry recorded. Concrete and strand samples were recovered and their mechanical properties measured. The stiffness of the beam before cracking equaled that calculated using the measured properties of the cross-section and neglecting any damage due to local deterioration. The moment for first cracking equaled that calculated using the weakest section within the constant moment region. However, that cracking was confined to the most deteriorated location. A marked stiffness change did not occur until general cracking at a moment that equaled the cracking moment calculated neglecting any strand corrosion. The immediate post-cracking stiffness also agreed with that calculated neglecting any strand corrosion. The load for failure was 4% less than that calculated using the measured properties of the weakest section and the number of non-corroded strands. The beam failed by crushing of the concrete in the compression face near midspan. The deflection for failure was only 40% of the expected deflection for the number of strands found to be effective at the load for failure.
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SUMMARY

Results are reported of a test to failure of a 54-ft. long, 27-inch deep, deteriorated pretensioned precast concrete deck beam. That beam, which was the first interior beam from a bridge over Putt Creek in Fulton County, IL and an adjacent exterior beam, had been removed from a bridge where they had been in service for over 30 years. During that time the longitudinal joint between the girders had opened sufficiently to allow salt laden water to seep down through the joint and evaporate from the soffits of the beams. That evaporation had resulted in spalling of the concrete covering the prestressing strands in the corner of the beam adjacent to the joint and the failure of three of those strands through corrosion.

The objective of the test was to provide information on how structural evaluations may best be made of beams with such corrosion damage and to investigate whether the deterioration had been proceeding from the inside out as well as from the outside in.

The more deteriorated first interior beam, and the less deteriorated exterior beam, were placed so that their top flanges faced one another and their bottom flanges faced outwards. Hydraulic rams and load cells were inserted between the beams at locations 7 ft either side of the centerline and the two beams strapped together at end supports that were 47 ft 8-1/2 inches apart. There was a roller support at one end and a pin support at the other. The weight of the interior beam was taken on roller supports placed at mid-span and the two ends. The exterior beam was supported on steel blocks at the same locations. Before testing commenced careful records were made of the damage visible on the surface, and to the strands, of the interior beam.

The beams were subject to a series of loading cycles of increasing displacements until failure occurred for the more deteriorated interior beam due to crushing of the concrete near its mid-span. Measurements were made of the applied loads and the deflections of the beams at mid-span, the load points and at the supports. Careful observations were made of the loads at which cracks developed and how those cracks developed. After testing was completed, the concrete cover was removed from the strands wherever longitudinal cracking had occurred and the degrees of corrosion of the strands and the stirrups, and any spalling damage to the concrete were recorded. An examination was also made of the degree of corrosion damage to the strands in the exterior beam at a location where longitudinal cracking occurred during the testing.

The beam that had failed was sawn in two close to the section that had crushed near mid-span. A microscopic examination was made of the condition of the strands within the section, the location of those strands, the dimensions of the concrete cross section, and the extent of cracking and spalling within that section. No concrete cracks internal to the section were observed and no corrosion was observed except around strands where the concrete over them had developed longitudinal cracks. However, within two weeks after their exposure severe corrosion had developed around the two strands closest to the corner that had spalled off.
The measured stiffness before cracking equaled that calculated using the measured average dimensions of the cross-section, neglecting any local damage. The measured load for first cracking equaled that calculated using the measured properties of the weakest section within the constant moment region of the beam. However, general cracking did not develop until the measured load reached that calculated using the average properties of the beam within the constant moment region and neglecting local corrosion damage to the strands. The post-cracking stiffness also agreed best with that calculated neglecting local corrosion damage to the strands. The load for failure was about 4% less than that calculated using the measured properties of the weakest section within the constant moment region of the beam.

Damage to the strands due to corrosion did not result in a brittle failure of the beam. However, the measured deflection for failure was only 40% of the expected deflection for the number of strands effective at failure as indicated by the load for failure.

It is concluded that, unless sophisticated instrumentation is used that can identify the effects of local as opposed to general cracking, the in-place load testing of deteriorated pretensioned deck beams will not be able to identify the number of strands that have failed in the beam due to corrosion. Further, the number of strands effective at failure may be less than the number effective at cracking. Therefore it is unlikely that the reduction in safety due to strand corrosion can be identified by the in-place load testing of pretensioned deck beam bridges.

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TEST TO FAILURE OF A 54FT. LONG DETERIORATED PRETENSIONED PRECAST CONCRETE DECK BEAM

A Report to the Illinois Department of Transportation
by
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1. Introduction

Some 10 percent of the inventory of pretensioned precast concrete box beam bridges, constructed in the 1960s and 1970s and located on Illinois State Highways, is showing significant deterioration. That deterioration is evidenced by rust stains on the side and bottom of the girders, by loss of concrete cover over the prestressing strands, and by breakage of prestressing strands. When breakage of a prestressing strand is observed by a Bridge Inspector, the bridge is automatically down graded from a Rating of 10 to a Rating of 3. No permit loads are permitted on the bridge until the Bureau of Bridges and Structures of the Illinois Department of Transportation (IDOT) has completed an analysis of the significance of the strand rupture for safety and continued use of the structure.

Currently there is little information available on which to base assessments of the relation between the evidence of deterioration on the surface of the girders and the likely degree of deterioration within the girder. However, based on field investigations there is reason to believe that when breakage of a prestressing strand is observed, there is also likely to be breakage to several adjacent strands for which the concrete cover is intact and severe corrosion of any enclosing stirrup steel.

IDOT pretensioned deck beams are manufactured in several different sizes (1). Beams are either 48 inches or 36 inches wide and of varying depths. The shallower beams contain circular voids and the shear reinforcement consists of welded wire fabric. The deeper beams contain rectangular voids and the shear reinforcement is deformed bars. The deeper beams are also all 36 inches wide so that beams have weights and sizes that can be readily transported by truck and lifted into place with a mobile crane. For a rectangular central void the flange and web thicknesses are less than for girders with circular voids. Based on field observations of deterioration there was also concern that leakage through the top flange, and atmospheric saturation inside the box, could be leading to deterioration proceeding from the inside out, for beams with rectangular voids, in addition to deterioration from the outside in.

2. Object and Scope

To provide data on which to base structural evaluations of pretensioned beams with broken strands, and to investigate whether deterioration was proceeding from the inside out, as well as from the outside in, a test to failure was conducted on two deteriorated 27 inch deep
pretensioned deck beams containing rectangular central voids. In order to determine the
exact location of the strands in the section, and to identify the degree of corrosion of those
strands relative to their position in the cross section, the failed beam was sawn in half after
testing was completed.

In the following report, the information available at the start of the test on the geometric and
structural characteristics of the beams and of the bridge of which those beams were part, is
reported in Section 3. The evidence of deterioration on the surface of the beams prior to the
start of testing is reported in Section 4. The set-up used to test the girder, the instrumentation
and the loading history are described in Section 5. Test results are described in Section 6.
During the test, cracking developed in many locations along the line of the strands in areas of
deterioration. That cracking allowed the cover over the strands in those locations to be
removed and observations to be made about the condition of the strands in the longitudinal
direction. Subsequently the beam was cut in two and the location and condition of all the
strands adjacent to the failure section were recorded. Those observations are described in
Section 7 along with the results of mechanical property tests on samples of concrete,
prestressing strand, and reinforcing steel taken from the beam after failure. In Section 8 the
test results reported in Section 6 are compared with the calculated response utilizing the
observations and mechanical properties reported in Section 7. The conclusions drawn from
this study are reported in Section 9. The Appendix provides details of the calculations of
strength and stiffness used in the comparisons of Section 8 and reports the results of chloride
tests on concrete samples taken from near the failure section.

3. Geometric and Structural Characteristics of Test Beams Based on IDOT
Records and Field Observations

The test beams were removed from Bridge 029-0039 over Putt Creek on US 24 in Fulton
County, IL. The girders had a length of 54 ft 0½ inches and ends skewed at 43°52'. At the
location of the bridge the longitudinal profile of the road was essentially north-south and flat.

One of the beams had been an exterior beam on the west side of the bridge. The other beam
had been the first interior beam adjacent to that exterior beam. In this report the exterior
beam is variously described as the "strong beam" and the "less deteriorated beam". The first
interior beam is variously described as the "weak beam" and the "more deteriorated beam".

Details for the beams as archived in IDOT files are shown in Fig. 1. The beams were
nominally 36 inches wide and 27 inches deep, with a 27-inch by 17-inch rectangular central
void. Each beam contained three such voids with each end void being 12 ft 1½ inches long
and the central void being 13'6" long. The voids were separated by 3 ft 7½ inch long solid
concrete diaphragms through which passed the transverse tie assembly. Due to the high
skew of the bridge, there had to be room for two tie assemblies in each diaphragm and at the
expansion end of the girder. A concrete curb and parapet, that had been attached to the
exterior beam in the field, had been removed for testing purposes.

The plans of Fig. 1 show that the beams were to be prestressed with 7/16 inch diameter
(0.109 in²), 248 ksi stress-relieved strand, although an alternate strand pattern using extra
high strength strand (270 ksi) was permitted. However, given that the actual strand used was not specified, any evaluation must be performed using the lower strength strand. Details of the transverse layout for the strands within the section were not available. However, it was specified that of the 21 strands in the section, 14 were to have their centers 1\(\frac{1}{4}\) inches above the soffit of the beam, 5 were to have those centers at 3\(\frac{3}{4}\) inches above the soffit and 2 were to have them at 13\(\frac{1}{8}\) inches above the soffit. The strand ends, visible at the beam ends, were consistent with that pattern. The specified initial stress for the strand was 173,600 psi (0.7\(f_{pu}\)). Since realistic losses are about 33,600 psi, the effective prestress was taken as 140,000 psi for evaluation purposes. The compressive strength specified for the concrete was 4,000 psi at release of the prestress and 5,000 psi at 28 days. The specified working stress for the deformed bar reinforcement was 20,000 psi. For strength evaluation purposes that reinforcement was taken as Grade 40. The plans for the bridge were dated June 10, 1968. The anticipated flexural and shear strengths of the beam, based on those specified properties and the loading setup used, and calculated in accordance with AASHTO LFRD principles, are derived in Appendix A.1.

The bridge site was visited prior to the removal of the beams. The beams looked in good condition except for a longitudinal crack in the roadway between the exterior beam on the west face of the bridge and the first interior beam. That crack was allowing salts to seep down the joint between the two beams and evaporate from the soffits of the beams adjacent to the crack. In the field there were stalactites 3 to 4 inches long growing along the central two-thirds of the length of the joint.

The beams as delivered for testing had an upward camber of approximately 2 inches. The plans showed an anticipated downward deflection of the edge beam, due to the dead load of the curb, parapet and wearing surface, of about \(\frac{1}{4}\) inch. The combined elastic and long term deflections due to that dead load effect undoubtedly contributed to the opening of the longitudinal crack between the exterior and first interior beam.

4. **Surface Appearance of Beams Prior to Test.**

The test beams were delivered to the test site in January 1999 and tested during November 1999. At the time of their delivery the beams were completely covered in ice and the prolonged freezing that they experienced may have contributed to their subsequent appearance. The two beams in the storage yard are shown in Fig. 2. The exterior beam is on the left and the interior beam on the right. The sawing of the longitudinal joint between the beams, in order that they could be removed, and the removal of the bridge's wearing surface, had caused loss of concrete cover along the upper edges of the joints on both sides of the beam. In the nine months between the delivery of the beams to the storage yard and the time that the photograph of Fig. 2 was taken, there had been considerable rusting of the exposed web reinforcement along the upper edges of the beam. In addition, the surfaces of the beams, that had been facing the leaking joint, had begun to slough off. As apparent from Fig. 2, that action had resulted in a deposition of aggregate pieces and mortar on the ground alongside the beams. That deposition was much greater for the interior, than for the exterior, beam. The deposition appeared to occur due to cracking that progressed through the aggregate and the mortar, in slivers that were about one inch square and 1/10th to 1/20th inch thick. The
deterioration was only on the surface of the beam that had faced the leaking joint and its removal exposed sound underlying concrete. Possible reasons for the deterioration are currently being examined through material tests. However, since much of the deterioration occurred during the summer months, rather than during freezing temperatures, it is believed that flaking was due to expansive salt crystallization within the pores of the aggregate and the concrete. Carbonate aggregates, such as the limestone aggregates used in these beams, are especially vulnerable to such attack (2).

For test purposes, the beams were re-positioned and rotated relative to the locations shown in Fig. 2. The beams were aligned north-south with the exterior (strong) beam on the east and the interior (weak) beam on the west. Further the two beams were rotated so that the bottom of the bottom flange of the beam (the soffit) faced outwards, with the corner from which deterioration had occurred facing upward for the interior beam, and downwards for the exterior beam. With the beams in that position it was easy to observe the condition of their bottom flange.

Shown in Figs. 3 through 20 are photographs of the soffit of the interior (weak) beam prior to test. For each figure, a black and white print is shown first, followed by a color print. The resolution is better for the black and white prints than the color prints. However, sometimes it is easier to distinguish features from the color prints.

A tape was attached at slightly above mid-height of that beam for its full length and distances at one-foot intervals, starting from the north end of the beam, are marked on the black and white prints. In Figure 4 the two drainage holes, present near the corners of the void are apparent between the 5 and 6-foot marks. The larger holes between the 6 and 7-foot marks, (see also Fig. 5), were made during the removal of the beams and were not present in the original construction. Indicated on Fig. 4 by broken lines is the location of the internal void. That void extends to the right from approximately the 5 ft 1 inch mark. There are two broken lines paralleling the edge of the girder. The line closest to the edge indicates the inner face of the nominal 4½-inch thick web wall. The line furthest from the edge indicates the toe of the 3-inch fillet between that wall and the flange. The dark area near the top edge of the beam between the 4 and 5-foot marks is a rust stain associated with corrosion of the prestressing strands. That corrosion has resulted in an approximately 5-inch deep by 6-inch wide piece of concrete cover spalling off. The drilling of the lifting hole between the 6 and 7-foot. marks also removed the cover over one of the stirrups at the 6 ft. 3-inch mark, showing that stirrup to be corroding.

From Figs. 5 and 6 it can be seen that there was an area of concrete cover spalling from about 9 ft. 3 inches to 12 feet. The upper edge of that spall, at about 6 inches from the upper edge of the beam, has extensive discoloration due to corrosion stains.

From Fig. 8, it can be seen that between 15 ft. and 18 ft. there is again an area of extensive discoloration due to corrosion stains. However there is no loss of concrete cover, except along the upper edge of the beam, until about the 17-foot mark. Then, there is a longitudinal spall that extends to about 7 inches from the top of the beam although no strands are exposed.
The end void terminates at about the 16 ft. 3 inch mark and a review of Figs. 4 through 8, over which that void extends, shows no pattern of corrosion products, or longitudinal cracking, that seems to be directly associated with the void. Rather the corrosion products, longitudinal cracking and concrete spalling are associated with the upper edge where the joint had been leaking.

In the subsequent loading test, one load point was at about the 20-foot mark (Fig. 9) and the other load point was at about the 34-foot mark (Fig. 14). The central void extended approximately from load point to load point. Within that length longitudinal cracking of the bottom flange is clearly visible along the length of the strands near the upper edge from 20 to 22 ft. and from 25 to 24 ft. There is also extensive discoloration of the concrete close to the upper edge due to corrosion products. Two broken strands are visible at about 27 ft. 6 in. and again at 30 to 32 ft. However, again corrosion products, longitudinal cracking, concrete spalling and strand breakage are all associated with the upper edge and there is no visible pattern of corrosion products associated with the central void.

In the final sector of the beam (Figs. 14 through 20), the pattern of corrosion products, cracking, spalling and strand breakage are the same as for the two prior sectors. There is a marked area of strand breakage between 40 ft. and 46 ft. 6 inches.

Shown in Fig. 21 is the upper face of the rotated beam. The beam face of Figs. 3 through 20 is the vertical face on the right. The area of the beam shown in Fig. 21(a) is that between 9 and 12 ft. in Figs. 5-6. It can be seen that the extensive spall on the soffit of the beam is matched by a wide crack on the side of the beam. That crack would allow the salts, flowing down the joint, direct access to the prestressing strands in the corner of the beam. The area of the beam shown in Fig. 21(b) is that for the central void section from 20 to 24 ft. The hole at the bottom left, with the bolt in it, is the location of the diaphragm at the north end of that void. It can be seen that, for about a 12-inch length, the corner of the beam has broken off allowing direct access of salts to the prestressing tendons.

There was no visible damage to the soffit of the exterior beam. An area of the soffit of that beam, that was subsequently close to the south load point, is shown in Fig. 22. In that case, the corner that matched the damaged corner of Figs. 3 through 20 is downward. Two corrosion points are visible close to the downward edge and two drainage holes are visible close to the left edge of the photograph. However there is no visible longitudinal cracking or concrete spalling. Yet there is ample evidence of salts leaking out of the joint and migrating across the soffit of the beam.

An investigation was made of the effectiveness of the ½ inch diameter drainage holes located in the corner of each void. Results are summarized in Fig. 23. For representation purposes the vertical scale is made double the horizontal scale. The broken lines show the relative positions and size of the interior voids. A hole that was open sufficiently that drainage could readily occur is indicated by a solid circle. A hole that was completely closed and through which no drainage could have occurred is indicated by a diagonal cross. A hole that was almost completely closed, but through which drainage may have been able to occur with difficulty, is indicated by a single sloping line. There was at least one drainage hole open in
each of the voids of the exterior beam. All the drainage holes for the central void of the interior beam were closed and the end voids each had only one partially open drain. All other drains were plugged. The interior beam was more deteriorated than the exterior beam and the most deteriorated edge of the interior beam was also the edge adjacent to which all drains were plugged. An examination was made of details of the prestressing wires where they had been exposed and corrosion fractures had occurred. Typical failures are shown in Figs. 24 and 25. In Fig. 24 three strands are visible. The central strand is completely fractured and a short length of tendon is completely missing. The end of one fractured wire is jagged, while the end of another is spear like. For the lower strands most of the wires have fractured, and jagged and spear type fractures are again visible. The upper tendon is essentially intact, but close examination shows that two wires are fractured. One labeled "X" has a jagged end and another labeled "Y" has a spear type end. Clearly both, jagged and spear type fracture can occur as a result of corrosion. The spear type fracture is similar to failure along an internal delamination in the wire.

5. **Test Set-Up, Instrumentation and Test Procedures**

Shown schematically in Fig. 26 is the test set-up and instrumentation. Figures 27 and 28 are photographs of the test set-up, and Figs. 29 and 30 show details of the loading jacks and reaction beams. The beams were subjected to concentrated loads applied 7 ft. either side of their centerlines. Distances between the centerline and the north reaction points were 23 ft.-71/2 inches and between the centerline and the south reaction points were 24 ft. 1 inch. It had been intended that those two distances be the same and equal to 24 ft. However, an error developed during the assembly of the test apparatus so that test dimensions were as shown on Fig. 26.

Figure 26 is a plan representation of the loading system. The weight of the test beams was taken on three wooden supports. Those supports were 12 inches wide, 12 inch high and 9 ft. long. The more deteriorated beam was set on roller supports that sat on the wooden supports. At the ends of the beam the roller supports consisted of a carriage with a table length equal to the 27-inch depth of the beam and with 9 inch wide by 6-inch diameter steel rollers located 5 inches from each end of the table. At the center of the beam, the roller support was again a carriage with a table length equal to the depth of the beam. However, the table had only a single central support which was a roller of the same dimensions as the rollers used at the beam ends. Thus the beam could translate at the ends and both rotate and translate at the center. The less deteriorated beam was set on steel I-beam supports that had a depth equal to the depth of the carriage/roller supports. It was anticipated that friction between concrete girder, steel I-beam and wood support would restrain the deformations of the less deteriorated beam while the more deteriorated beam would be free to deflect laterally relative to the less deteriorated beam.

The beams were set up so that their interior faces were approximately 44 inches apart. That distance was needed to allow the insertion of two sets of two reaction beams, a loading jack and load cell between the test beams. Those details are readily apparent from Figs. 29 and 30. The reaction beams were 8 inches deep, 8 inches wide and 20 inches long. They were attached to 1 inch thick by 10 inch wide and 24 inch long steel plates. The plates were set on
hydrastone and attached by adhesion anchors to the vertical face of the test beam. The 8 by 20-inch wide bearing area of the reaction beam simulated the contact area for a standard truck wheel at service loads. The jacks were set up 7 ft. either side of the centerline of the beams so that the spacing between them equaled the minimum spacing between axles specified for the design truck. That minimum spacing was the controlling spacing for the axles of the design truck for the given test beam span.

Bolted to the reaction beams were 60-ton centerhole jacks. Those jacks had an effective area of approximately 13.75 square inches and maximum ram extensions of approximately 10 inches. Positioned on top of that ram (Fig. 29), were two bearing plates separated by a 1½ inch diameter steel ball. Beneath the plate remote from the ram was a load cell which in turn was attached to the reaction beam on the adjacent beam. The steel ball allowed for a reasonable amount of rotation of the axis of the jack relative to the axis of the load cell. At a ram extension of about 10 inches, however, the limits of the rotation were reached.

The reactions at the beam ends are indicated schematically in Fig. 26 and are apparent in the photographs of Figs. 27 and 28. Steel reaction frames were bolted to the face of the less deteriorated beam. A roller bearing consisting of two 1-inch diameter 24-inch long rollers was positioned on the more deteriorated beam at one end and a 2-inch diameter 24-inch long rocker bearing was positioned at the other end. The rocker and roller bearings were then brought into contact by a tensile yoke positioned around the ends of the two beams. The yoke consisted of two 1⅛-inch diameter prestress bars one of which passed over the two beams and the other of which passed below the beams. On the end of each bar was an anchor nut that reacted against two interconnected 4-ft. 6-inch high channels that bore against the soffit of each beam. Before loading commenced each bar of each yoke was prestressed to 35,000 lbs.

Linear variable differential transducers (LVDTs) were used to measure displacements at the center of each beam and at the load points of each beam. In addition dial gages were used to measure the displacements at each end of each beam, and at the top and bottom edges of the more deteriorated beam at the position of the north load point. The latter measurements would indicate if significant torsion occurred during loading. No such action occurred.

With the exception of the dial gages used to examine torsion effects, the ends of the LVDTs and dial gages bore against targets cemented to the centerline of the soffit of each beam. All LVDTs and dial gages were supported on steel stands bolted to the asphalt. Those stands can be seen in Figs. 27 and 28.

The output from the LVDTs and load cells was recorded on a computer system (Fig. 28) that was also used to process the data. Readings were taken every 5 seconds during the test.

To test the beams high-pressure oil lines were used to connect the jacks to an electric pump. The pump pressure was increased in increments of about 400 psi (5.5 kips per jack) or 500 psi to a pre-determined maximum and then released back to zero or 400 psi. The pump pressure was then brought up again in 400 psi or 500 psi increments to a new pre-determined maximum before being again released. After each 400 or 500 psi increment, the pump
pressure was held steady, all dial gages read and any cracks on the surfaces of the beams were marked.

Testing took place over a period of three days. The loading history, expressed in terms of the pressure indicated by the gage on the pump and used to control the testing sequence, is shown diagrammatically in Fig. 31. The first load cycle to 400 psi was to assess if the gages, load cells and test apparatus were working properly. That load cycle was applied on the first day of testing. The second load cycle was to a condition where cracks were clearly visible in the more deteriorated beam. The peak pressure reached in that cycle was 2800 psi. The third load cycle was to a peak pressure of 3800 psi and represented the condition where the strands were expected to be stressed to slightly greater than their yield stress. The fourth load cycle was to a peak pressure of 4080 psi at which pressure the jacks reached an extension of 9 and 7/8th inches and the pressure had to be released so that the jacks could be restroked. That fourth cycle was the end of the second day of testing. In the third day of testing the pump pressure was increased to 3000 psi, reduced to 500 psi and then again increased to 4000 psi. As the adjustments necessary to be able to restroke the jacks were being made, the concrete at the bottom of the more deteriorated beam, on its compression face, began to crushing. The pressure was reduced to zero and testing terminated.

6. **Test Results.**

The test results, in the form of a series of load-deflection plots, are shown in Figs. 32 through 36. For Figs. 32, 34 and 36, the applied load is the average value for the load recorded by the two load cells. For Figs. 33 and 35 the applied load is the value for the load cell at the location where the deflection is being recorded. For Figs. 32, 33, 34, and 35 the deflection is the unadjusted value recorded at the location shown. For Fig. 36, the deflection is the average displacement of the two ends of the beam. For increasing loads that displacement was always in the west direction. Thus, to obtain the actual displacement at a given point, relative to the ends of the beam, the displacement for a given load as shown in Fig. 36, should be added to deflection values for the less deteriorated (strong) beam and subtracted from deflection values for the more deteriorated (weak) beam. For example, from Fig. 32, in cycle 2, at a load of 35 kips, the unadjusted displacement for 33 kips is 1.15 inches. From Fig. 36, the average displacement at the ends is 0.10 inches. Therefore the displacement at mid-span relative to the ends was 1.05 inches. For the strong beam, from Fig. 34, the unadjusted mid-span deflection for 33 kips is 0.79 inches. Therefore, the displacement at mid-span relative to the ends at 33 kips was 0.89 inches. For any given load the displacement for the strong beam should be less than that for the weak beam because frictional effects would have restrained the strong beam deflections.

The more deteriorated beam had clearly cracked by 2400 psi pressure (35.0 kips) but the location of those cracks was not readily apparent on the surface of the beam. However, those cracks had opened perceptibly by 2800 psi pressure (40.0 kips) and were clearly apparent on the soffit of the beam. The crack spacing was 15 inches, the cracks extended to mid-height of the beam, and they were spread throughout the constant moment region between load points. For the less deteriorated beam cracking occurred with a sharp noise at slightly before 3200 psi pressure (44.2 kips). By 3200 psi pressure the cracks had opened perceptibly and extended to about 6 inches from the top of the beam.
Based on the shapes of the load-deflection curves and visual observations it is concluded that cracking occurred at 33.3 kips and at 42.1 kips for the more deteriorated, and less deteriorated, beams, respectively. The corresponding moments are 6770 and 8559 kip-inches, respectively. The foregoing cracking loads are indicated, as appropriate, on Figs. 32 through 35.

With increases in load beyond those for cracking, additional cracks occurred between the original cracks so that by close to the ultimate load, the crack spacing for each beam had decreased to an average of 5 inches and cracks extended to within 5 inches of the top of the beam. The crack spacing is clearly visible in Figs. 47 and 48. Further it can be seen from those figures that for the length of the internal diaphragm, there was reduced cracking at the north end (14-17 ft.) but not at the south end (14 –17 ft). The degree of strand corrosion damage over the length of the diaphragm was much greater at the south end than at the north end.

The maximum load achieved in the test was 53.3 kips at a pressure of 4080 psi at the top of the fourth cycle. The corresponding moment was 10,836 kip-inches. That load did not cause failure. However, the loading rams reached their limiting strokes and the beams had to be unloaded. In the subsequent loading cycles, the maximum load reached at the limiting stroke of the jacks was 49.2 kips and at that load crushing developed over the bottom half of the compression flange of the more deteriorated beam close to mid-span. That crushing is clearly evident in Fig. 37 which shows photographs of the compression surface of the more deteriorated beam after the final unloading.

If the maximum applied load has exceeded that for cracking of the concrete but has not exceeded that for yielding of the prestressing tendons, then the decompression moment can be determined from the form of the load-deflection curve for reloading following a prior unloading. The decompression moment is the moment at which the previously formed cracks re-open. From Fig. 32 it can be seen that the decompression load for both the third and fourth loading cycles, for the more deteriorated beam, is 24.4 kips which corresponds to a moment of 4,961 kip-inches. The difference between the maximum tensile stresses at the cracking and decompression moments should be consistent with the modulus of rupture for the concrete. The prestress force can be calculated from the maximum tensile stress associated with the decompression moment.

The properties of a 27-inch deep pretensioned deck beam, as listed in IDOT's Prestress Concrete Bridge Design Manual (1) are as follows:

- Cross-sectional area = \( A_c = 522.8 \text{ in}^2 \)
- Moment of inertia = \( I_g = 48,237 \text{ in}^4 \)

Thus, based on the nominal properties of the section, and ignoring any bi-directional effects caused by self-weight, the modulus of rupture for the concrete of the more deteriorated beam was 506 psi and the effective prestress force was 290 kips. The derivation of those values is shown in Appendix A.2.
Close to the failure load cracks began to occur along the line of the prestressing strands. Those cracks were relatively fine, but their presence allowed easy removal of the cover over most of the length of the upper three strands following failure. The type of crack being discussed is a tensile splitting crack caused by bond stresses developed between the concrete and the prestressing strand. They can be seen from Figs. 38 and 39 which show views of the soffit of the less deteriorated beam, close to mid-span, after testing was complete. In Fig. 38 a short longitudinal crack close to the bottom of the less deteriorated beam is indicated by the letters X-X. A close-up view of that same crack is shown in Fig. 39. The crack is about 7 inches from the edge and is clearly visible for 18 inches.

Close to the failure load a flexure-shear type inclined crack formed in the shear span of the more deteriorated beam at about one and one-half times the depth of the beam from the south load point. As was also the case for the flexural cracks, that inclined crack did not open significantly before crushing of the concrete occurred on the compression face of the beam.

7. **Investigation of As Tested Beam Properties**

Following failure of the more deteriorated beam, its as-built properties were investigated by:

1. Mechanical property tests on concrete and prestressing strand samples taken from the beam;

2. Removing the concrete cover wherever possible over the length of the deteriorated prestressing strands in the upper corner of the beam. The longitudinal cracks that had developed along those strands close to failure were used to facilitate cover removal;

3. Cutting the beam in half and measuring: (a) the exact location of each prestressing strand and longitudinal reinforcing bar in the bottom two-thirds of the beam; and (b) the thickness of the webs and flanges of the beam; and

4. Taking 1.0 inch diameter concrete cores from the edges of the beam and using those cores to determine chloride content profiles for the concrete.

7.1 **Mechanical Properties of Concrete and Steel**

The strength of the concrete in the compression flange of the beam was determined by taking two 4-inch diameter cores vertically through the depth of the flange at positions located on the centerline of the beam and 10 ft. from the supports. The concrete at that location should not have been stressed inelastically during the test. The height of the samples was limited to the depth of the flange which was 4.25 and 4.33 inches, respectively, at the two locations. The unconfined compressive strengths of those two samples were 8,460 and 8,400 psi, respectively. When those values are corrected to values for a 6-inch diameter cylinder 12 inches long, the corresponding average strength is 6,810 psi. The weight of the concrete was 149 lbs./cu ft. and only minute voids were visible on the surface of the cores.
The stress-strain properties of the prestressing steel were determined by sawing a 43-inch long by 8 inch deep and 8 inch wide block of concrete from the soffit of the beam at the edge remote from the leaking joint. That 43-inch length was taken from about the mid-length of the shear span of the beam. The strands at that location should not have been stressed inelastically during the test. The edge of the beam, with the length removed for recovery of the strands, is shown in Fig. 40. At the location where that concrete block was removed it was verified that the shear reinforcement was #3 deformed bars at an average spacing of 15 inches.

The concrete block was taken to the laboratory and the tendons removed by using knife edges to split the concrete along the line of the tendons. There were no corrosion products visible either on the tendons or on the concrete to which they had been bonded. The surface of the tendons appeared clean and unblemished. Four tendon samples, each 43-inches long were recovered.

The tendon samples were tested to failure in a 100 kip capacity MTS servo-controlled hydraulic testing machine. An extensometer was used to measure elongations over an eight-inch gage length and a data logger was used to record the applied load and elongations at frequent intervals up to failure. The resulting load-strain relationships for all four samples are shown in Figs. 41 through 44. Specimen 1 failed at the mid-point between grips at a load of 30.25 kips and an ultimate strain of 7.1%. Specimen 2 failed at the face of the lower grip at a load of 30.44 kips and an ultimate strain of 7.25%. Specimen 3 failed at a point mid-way between grips where two of the wires of the strand had been damaged during construction of the beam. The wires appeared to have been hammered and loading was stopped when those two wires failed. The place where the hammering had occurred was also the location of one of the stirrups where it bent around that strand. The other five wires of the strand remained intact. The failure load was 29.9 kips at an ultimate strain of 4.9%. Specimen 3a failed at the face of the top grip at 30 kips and an ultimate strain of 5.5%. For discussion and calculation purposes it is realistic to neglect the result for specimen 3 and utilize the average result for the other three specimens. Individual and average properties for the other three specimens are listed in Table 1. Prior to testing each specimen was weighed and its length accurately measured. Weights varied from 0.3753 lbs./ft to 0.3758 lbs./ft for an average of 0.3756 lbs./ft. Those weights show that the strand was nominally a Grade 250 - 7/16-inch diameter material for which the required ultimate tensile strength is 27 kips. At the locations where wires failed, the failure surfaces were all of the traditional cup and cone type. None of the surfaces were of the jagged or spear type seen for the wires that failed due to corrosion.

The properties of both the concrete and the prestressing steel exceed the specified minimum strengths. Only the modulus of elasticity of the steel, evaluated as 27,800,000 psi, was less than the anticipated value of 28,500,000 psi. Because difficulties were experienced in the tests in accurately measuring strains in the elastic range, the specified value of the steel modulus, rather than the measured value, was used for analysis.

### 7.2 Appearance of Beams After Failure

The appearance of the soffit of the less deteriorated beam after testing was completed is shown in Figs. 45 and 46. Figure 45 shows the south half of the beam and Fig. 46 shows the
north half. The numbers in black along the top edge of the beam, and in increments of 3 (i.e. 3, 6, 9, etc.), are the distance in feet on the centerline of the beam from its south end. At between 21 and 24 ft., three strands at the bottom of the beam have been exposed. The condition of the beam after failure and before that exposure occurred, is shown in Fig. 22. From Figs. 22, 45 and 46, it is apparent that there was no marked corrosion damage visible on the soffit of the beam prior to testing, and the test itself caused no significant damage.

Shown in Figs. 47 and 48 is the appearance of the soffit of the more deteriorated beam after testing was completed and any loose concrete covering the strands was removed. The north half of the beam is shown in Fig. 47 and the south half of the beam in Fig. 48. Distances on the tape are measurements from the north and south ends of the beam for Figs. 47 and 48, respectively.

The information of Figs. 47 and 48 is reproduced in a slightly different form in Fig. 49 which is a diagram of the visible corrosion damage to the prestressing strand, the stirrup steel, and the concrete cover. In Fig. 49 the horizontal scale is 2 inches equals 5 ft. Distances are shown from the north and south supports respectively. The vertical scale is 0.2 inch equals 1.0 inch. Different symbols are used to indicate strands with all wires severed due to corrosion, more than half the individual wires severed, and less than half the individual wires severed. It can be seen that, in the region between load points, there are two locations at which three strands were found to have completely ruptured. One location is about 3 ft to the north of the centerline of the beam. The other location is about 4 ft. to the south of the centerline. At failure crushing occurred on the compression face of the beam (Fig. 37). That crushing was located at about the depth of the beam towards its centerline from those same locations. It should also be noted from Fig. 39 that the crushing was also at about the end of the length where the two #5 bars placed in the top of the beam had been spliced.

It was obvious, that effectively at least three strands had failed prior to testing, as a result of corrosion. Before testing had begun, it was also clear that three strands had failed at about 4 ft. from the support (Figs. 24 and 25). However, because of intact concrete cover it was not obvious that three strands had also already failed in the constant moment region.

7.3 Cross-Sectional Properties of Beam at Failure Location.

After the test to failure had been completed, a crane was brought again to the test site and the more deteriorated beam rotated, Fig. 50, so that its soffit was uppermost and that it was supported at both ends and at sections 2 ft. either side of mid-span. The beam was then sawn into two halves at a location 2 ft 6 inches north of mid-span and therefore close to one of the sections at which crushing had occurred on the compression face.

The appearance of the severed section of the beam, the day after cutting is shown in Fig. 51. The photograph on the left shows the north half of the beam and that on the right the south half. It was possible to saw the section completely through for the tension flange and both webs to the depth of the compression flange. The compression flange was then broken by lifting the two halves of the beam. The concrete broke and the longitudinal steel in the compression flange was severed by sawing.
In Fig. 51 the ends of all the strands and longitudinal steel for the north half, and for most of the south half, have been darkened so that they are more obvious. The strand pattern is symmetrical about the longitudinal centerline of the beam with the exception of one strand, in the right half, close to the end of the missing concrete corner, that is higher than its counterpart in the left half. Also at 11 inches from the soffit of the beam there was a #4 bar placed longitudinally in each web. Those bars are not specified in Fig. 1 and apparently were used to hold the stirrup reinforcement cage in place.

The measured values for flange and web thickness, and for the location of the strands, are shown to half scale in Fig. 52. Since the strand layout was essentially symmetrical with respect to the centerline of the beam, only results for the right half of the beam, (the half containing the corrosion-damaged corner), are shown in Fig. 52. Figure 52 shows the break line along which concrete cover was lost and the three strands that had corroded away in the right half are shown as hatched. Further, the location for the one strand that was positioned differently in the left and right halves of the beam is shown as a continuous circle for the location in the right half and as a broken circle for its location in the left half.

The locations for the two strand layers, specified on Fig. 1 of 1¾ inches and 3¾ inches from the soffit of the beam, are shown on Fig. 52. The cover to the centerline of the strand for the strand line closest to the soffit was about 3/8 inch less than that specified. In one case, the cover to the centerline of the strand is only 1¼ inches. The reduced concrete cover for this outer layer of strand, compared to that specified, makes the beam more vulnerable to corrosion than the design beam. However, it should also be noted that a cover of 1¼ inches is within the tolerance allowed by AASHTO for beams of 27-inch depth. The cover to the second layer of strand averaged very close to that specified.

The thickness of the bottom flange was 5-5/8 inches and that of the top flange was 4-3/8 inch. The latter value confirms the top flange depths found when those flanges in the shear span were cored to take concrete samples for compression testing. It can be concluded that the treated corrugated cardboard, that formed the void, floated upward during the manufacture of the beam. That relocation of the void has negligible effect on the moment of inertia of the section.

The thickness of both webs of the beam was 4½ inches, as compared to the specified thickness of 4½ inches. While that change could cause about a 5% decrease in the shear capacity for inclined cracking, it causes less than a 1% change in the moment of inertia and only about a 1.5% change in the area of the section.

Due to spalling of the concrete a considerable loss of area had occurred on the compression face of the more deteriorated beam before testing. That deterioration is readily apparent in Figs. 2 and 37. At the section where failure occurred in the test, the equivalent of a wedge of concrete 2 inches deep at the edge of the beam, and vanishing at 10 inches into the section, had been lost on the leaking joint side of the beam. At the other corner, the equivalent of a wedge of concrete 1 inch deep at the edge and again vanishing at 10 inches into the section, had been lost.
Examination of the interfaces between the prestressing strands and the concrete with a 10 x power microscope at the sawn section, immediately after cutting was complete, showed no presence of corrosion products for any of the interfaces that were inside the intact concrete area. A similar examination of the concrete surfaces showed no cracking. That examination looked for vertical splitting cracks between each tendon and the closest concrete surface, cracking through the depth of the flange along the centerline of the void, and cracking horizontally between adjacent strands. While one or more of those three types of cracks had been seen on the surfaces of the two beams that had failed in service, no such cracks were found for this beam.

Shown in Fig. 53 is the appearance of the sawn section two weeks after the photographs of Fig. 51 were taken. Corrosion products are clearly visible around the two strands closest to the corner where the concrete wedge had spalled off. The rapid formation of the corrosion products in that area of the section, but not elsewhere, demonstrates the high chloride content of the concrete in the vicinity of the wedge.

For the less deteriorated beam an investigation was made of the degree of corrosion of the strands in the corner of the beam analogous to the corner that had spalled off for the more deteriorated beam. The location selected was adjacent to the rust spot visible in the center of Fig. 22. The longitudinal crack that had developed in that beam, immediately above the rust spot, and shown in Fig. 39, was used as an entry point to chisel the concrete cover off the strands, as shown in Fig. 54, and expose the three corner strands.

Findings from the investigation of the less deteriorated beam are summarized in Figs. 55 and 56. The two strands closest to what had been the soffit of the beam had a clear cover of 1¼ inches, and were therefore one quarter of an inch closer to the surface of the beam than the distance specified in Fig. 1. For those two strands, the surface of the strand closest to the soffit was corroding while that facing away from the soffit was clean and uncorroded. Corrosion extended about halfway around the perimeter of the strand. Corrosion had already caused fracture of two of the wires in the uppermost tendon and fracture of one of the wires in the middle tendon. The lowermost tendon had a clear cover to the soffit of two inches and a clear cover to the side face, which had been that beneath the leaking joint of 2-3/8 inch. That tendon is just visible below the middle tendon in Fig. 55. The surface of that lowermost tendon was bright and showed no presence of corrosion products.

7.4 Chloride Content Measurements

One-inch diameter concrete cores were taken from the soffit and vertical faces of the beam at a cross section eight inches north of the location where the beam was cut in half. Those cores were used to obtain chloride content profiles for the concrete section. Most cores were 3.0 inches long and chloride contents were measured at distances of 0.25, 0.75, 1.25, 1.75, 2.25 and 2.75 inches along the core. In some cases cores of lesser lengths were used because reinforcement was encountered during the coring operation. Test results and profiles are reported in Appendix A.7.
On the soffit, cores were taken on a transverse section at distances located 4.0 inches from each vertical face of the beam. On the vertical faces of the beam, the cores were taken at distances of 2.63, 5.25, and 8.0 inches from the soffit. Those distances were selected in order that the cores did not encounter any of the longitudinal reinforcement in the beam. It can be seen from Fig. 52, that on the damaged side of the beam, the core at 2.63-inch depth was within the zone where the strands in an immediately adjacent region had been lost through corrosion. From the results in Appendix A.7 it can be seen that the soffit cores were of limited length. Unfortunately those cores encountered the web reinforcement of the beam.

After each core was taken it was carefully labeled with its location and then stored in a sealed plastic bag until removed for testing. Samples were analyzed for chloride content essentially by ASTM C1152 “Acid-Soluble Chloride in Mortar and Concrete.” It is generally accepted that chloride content values exceeding about 0.03 to 0.04 percent by weight of concrete, depending on the cement content, promote corrosion of embedded unprotected steel in concrete.

In general results are consistent with the corrosion damage observed for the beam. On what had been the east face of the beam in the field, there were greater average chloride contents through the thickness of the web and over its depth than on what had been the west face of the beam. That greater salt content was probably the reason for the greater sloughing of the concrete surface of that east face, as shown in Fig. 2, than the west face. However, except within the bottom four inches of the beam, the chloride contents at the depth to the stirrup and prestressing steel from the vertical face, were not sufficient to promote corrosion.

On what had been the west face of the beam in the field, average chloride contents for the corner between that face and the soffit, and for some distance along the soffit, were well above the values needed to promote corrosion. Further the depth from the soffit to the stirrup steel was much less than the depth from the side faces to the same steel. Therefore marked corrosion of that steel and the prestressing strands was to be expected in the west bottom corner of the beam.

8. **Comparison of Measured and Computed Strength and Stiffness Properties.**

In this section, comparisons are made between measured and computed properties for six quantities: the cracking load, the stiffness prior to cracking, the stiffness after cracking, the load for flexural failure, the load for flexure-shear cracking, and the load for a shear failure.

To make the flexural strength and stiffness comparisons, the four different cross-sectional conditions shown in Fig. 57 were used. The section shown in Fig. 57(a) is that for a 27-inch deep pretensioned deck beam as listed in IDOT's Prestressed Concrete Manual (1). The center gravity of the beam, as shown in Fig. 57(a), differs slightly from that for the same section, as shown in Fig. 1(b). The Fig. 57(a) value is the correct value. The section shown in Fig. 57(b) is that for the as-built deteriorated beam as determined from measurements of section sizes following the test to failure. The thickness of the webs was 4¼ inches rather than 4½ inches specified. The central void had also floated upwards so that the upper flange thickness was 4-3/8 inches, rather than the 5 inches specified. Those changes result in
changes in the center of gravity of the beam, its area, and its moment of inertia from those for the section of Fig. 57(a).

The sections shown in Figs. 57(c) and (d) preserve the properties of the section of Fig. 57(b). They add, however, other features that are the loss of three strands, Fig. 57(c), and that loss plus the loss of the concrete area on both the tension and compression faces, Fig. 57(d), that replicate the conditions observed at the failure section after test. From Figs. 3 through 21 it is clear that for much of the length of the beam, the section shown in Fig. 57(b) is representative of the existing conditions. For the maximum moment region, (20 through 34 ft., Figs. 9 through 13), visual observations would suggest that the section of Fig. 57(b) is still applicable except in locations such as those at 28, 30 and 32 ft. where some broken strands are visible. The theoretical distance for development of the strength of a 7/16th-inch diameter Grade 250 strand with an effective prestress of 140 ksi is 48 inches. Shorter development lengths are to be expected for rusted strand. Such development lengths are additional to the 20 inches necessary to transfer that value of the 140 ksi effective prestress to the concrete. Thus, the sections of Figs. 57(c) and (d) could be expected to perhaps apply for sections from 24 ft. through about 34 ft along the length of the beam.

Pertinent properties for each section shown in Fig. 57 are listed below that section. Row 1 lists the resultant concrete area, \(A_c\), Row 2 the moment of inertia, \(I_b\), Row 3 the effective depth, \(d_p\), and Row 4 the resultant eccentricity, \(e\), of the prestress force from the center of gravity for that section. Values for \(d_p\) and \(e\) for Figs. 57(c) and (d) are calculated based on the three strands in one corner of the beam, as shown in Fig. 52, having been severed.

In Fig. 57 Row 5 lists the prestress force, \(F_{se}\), associated with each section. That prestress force was calculated assuming an effective prestress in the strands at time of test of 140,000 psi. The 140,000 psi value is consistent with the prestress force associated with the decompression moment, Section 6. It is also equal to the stress calculated by subtracting from the specified initial stress in the tendons of 173,600 psi, losses of 33,600 psi calculated using the procedure of the PCI Design Handbook (3). That calculation is reproduced in Appendix A.3. For that loss calculation all strands were assumed effective, as would be the case in the early life of the beam. The volume to surface ratio was calculated assuming the box to be effectively unventilated and the strands were taken as stress-relieved Grade 250 tendons.

Rows 6 and 7 of Fig. 57 list geometric properties needed to calculate the cracking moment, \(M_{cr}\), listed in Row 8. The cracking moment was calculated from the formula:

\[
M_{cr} = \left( I_p / y_b \right) (f_r + F_{se} / A_c + F_{se} e y_b / I_g)
\]

In that formula the modulus of rupture, \(f_r\), was taken as \(7.5 \sqrt{f'_c}\) and the concrete strength \(f'_c\) was taken as the value of 6,810 psi established from the core tests. The corresponding load for cracking, \(P_{cr}\), is listed in Row 9.

Rows 10 and 11 list the nominal flexural moment strength, \(M_n\), and the corresponding load for failure, \(P_n\). The stress in the prestressing steel at failure, \(f_{ps}\), was calculated based on strain compatibility. Use of the approximate equation (18-3) of ACI 318-99 (4) gave almost
the same value for $f_{pc}$, as the strain compatibility calculations. For all sections shown in Fig. 57, the neutral axis depth at failure was in the flange. For the section of Fig. 57(d), the neutral axis was at the bottom of the top flange. For all the other sections the neutral axis was above the bottom of the top flange. A set of typical strength calculations, (those for the Deteriorated As Built Section of Fig. 57(c),) are included as Appendix A.4.

Shown in Fig. 58 is a plot of the envelope to the load mid-span deflection curves of the weak beam of Fig. 32 corrected for the displacement of the ends of that beam per Fig. 36. The resulting relationship is an unbroken line consisting of a series of straight-line segments.

Also shown on Fig. 58 are the predicted loads for cracking and failure for several of the different sections of Fig. 57. The stiffness changed measurably at the cracking load predicted for the section of Fig. 57(d) but did not change markedly until the cracking load for the section of Fig. 57(b). That result is consistent with what might reasonably be expected. The reduced section of Fig. 57(d) represents the worst condition within the constant moment region of the beam. Other sections within that same region would have characteristics varying between those of Fig. 57(b) and those of Fig. 57(d). Thus, while first cracking would be associated with the characteristics of the section of Fig. 57(d), general cracking would be most likely associated with the characteristics of the section Fig. 57(b). Even if strands are fractured due to corrosion at a given location in the constant moment region, the distance required to again develop the effective prestress in the strand is relatively short. According to ACI 318-99 that distance for a clean 7/16 inch diameter strand stressed to 140,000 psi is 20 inches. Slightly corroded strands can be expected to bond in an even shorter distance. Further, that 20-inch development length is consistent with the observed initial crack spacing of 15 inches.

The measured ultimate capacity was in closest agreement with the capacity predicted for the section of Fig. 57(d) and significantly less than the predicted ultimate capacities for the sections of Figs. 57(b) and (c). Again, that result is reasonable because the strength will be controlled by the properties of the weakest section in the constant moment region. However, the measured capacity is still about 4% less than that predicted for the properties of the section of Fig. 57(d). Further, to obtain the moment capacity listed for the section of Fig. 57(d) the steel stress would have needed to reach 261,000 psi. At that stress the strain in the steel would be about 2% and the cracks on the tension side would have to have opened significantly. That condition had not occurred.

For the condition of crushing of the concrete at a strain of 0.003 on the compression face of the section of Fig. 57(d), the mid-span deflection would have to be about 20 inches. For cycle 6, although the maximum deflection reached was greater than in cycle 4, Figs. 31 and 32, the ultimate load reached was less than for cycle 4. Effectively the deflection for the initiation of crushing of the compression flange was 8 inches. Thus, the warning of impending collapse for this deteriorated beam was less than that to be expected based on current reinforced concrete theory.

The reasons for the reduction in ultimate capacity and, more particularly, the deflection at that capacity are not clear. Three factors could be important. One is the deterioration in capacity as a result of the high loads imposed in cycles 3 and 4. That reduction in capacity is
usually reflected in a large increase in the permanent residual deformation for zero load. From Fig. 32 it can be seen that the permanent residual deformation after the completion of cycle 4 was only about 1.0 inch. That degree of damage would not normally be expected to be associated with a 12-inch reduction in the ultimate deflection capacity.

A second factor that could cause a decrease in ultimate deflection, is a decrease in neutral axis depth, and a resultant increase in strains near mid-span of the beam. That action could result from the development of a form of tied arch action in the constant moment region of the beam as it approached failure. From Figs. 9 and 27 it can be seen that the jack loads were applied close to, or on, the large solid diaphragms that separated the internal voids. Those diaphragms, with a length of 3 ft. 7-5/8 inches, Fig. 1, would result in a lower position for the neutral axis within them at loads above cracking, but less than ultimate, than would be the case for the central portion of the beam. Further, as loads increase beyond those for yielding, it becomes increasingly difficult to develop, within the constant moment region, the required stress for a strand that is severed due to corrosion at any point within that length. As a severed strand attempts to develop stress it causes a strain concentration in the strands that parallel it and are intact. Those considerations would reduce the ultimate deflection for failure.

A third factor is the limitations of the test setup. At 10 inches of ram extension the axes of the jack and the load cell were no longer in alignment. Effectively the beam was being subject to forces pushing towards the supports at the position of the load cells. That action would place an axial tension force on the constant moment region while also slightly reducing the moment acting on it. The effect of the former would dominate over the latter and increase as the deflection of the beam increases.

In combination the effects of high cycle fatigue, tied arch action and the limitations of the test setup may account for the reduced ultimate deflection and ultimate capacity. The calculated flexural strength is the same as that observed in the test if the section of Fig. 57(d) is used and the number of ineffective strands is taken as four rather than three. However, no evidence could be found of four strands being ineffective in the beam at any location and a reduction in the number of effective strands would not explain the marked reduction in the deflection for failure from that calculated to that observed.

Shown in Fig. 58 by broken lines are the predicted responses for a section for which the stiffness is calculated using the properties of Fig. 57(b). The section stiffness following cracking, \( I_{cr} \), was calculated using the procedures of Reference 3. The calculated stiffness before cracking, and that after cracking, are in good agreement with the measured stiffness for those same two conditions. The calculated deflection for general cracking at a load of 38.1 kips is 1.22 inches. That calculated deflection changes little with the use of the section of Fig. 57(c) rather than that of Fig. 57(b).

The calculated stiffness after cracking is considerably less for the section of Fig. 57(c) than that for the section of Fig. 57(b). For a 10 kip increment in applied load beyond that for cracking, use of the properties of Fig. 57(c) results in a 30% increase in the calculated deflection beyond that for a section with the properties of Fig. 57(b). The linear elastic
cracked section behavior associated with $I_r$ should remain valid until the concrete or steel begins to go inelastic. For the measured properties of the steel and concrete for the test beam, the steel is computed to begin to deviate from its linear range of behavior before the concrete. For the section of Fig. 57(b) that deviation is predicted as beginning at a deflection of about 4.0 inches. Thus, the measured results suggest that calculations of deflections between general cracking and first inelastic action can be best made using the section properties of Fig. 57(b) rather than those of Fig. 57(c). Reproduced in Appendix A.5 are calculations for: (1) the deflection associated with a load of 38 kips for an uncracked section stiffness; and (2) the change in deflection with an increase in load from 38 to 50 kips for the cracked section stiffness for a beam with the section properties of Fig. 57(b).

An inclined crack was observed in the void region of the south shear span of the more deteriorated beam at the ultimate load. In the test beam the south load point lay over the solid interior diaphragm (Fig.14). Usually, for two-point loading, the critical section for flexure-shear cracking is immediately adjacent to the load point in the shear span. In the test beam, however, the inclined crack developed in the void region of the south shear span immediately adjacent to the south face of the internal diaphragm.

The flexure-shear cracking load was calculated using the recommendations (5) that resulted in the procedures specified in Reference 4. In those recommendations the shear for inclined cracking is the sum of two terms. The first term is the shear for the development of a flexural crack at a distance of half the effective depth of the beam away from the critical section. The second term is the shear required to propagate that crack diagonally through the web of the beam.

For a solid section, the concrete area of the test beam is 963.2 sq. in., the c.g. of the beam is 13.43 in. above the soffit, and the moment of inertia is 58,286.6 in$^4$. The moment for flexural cracking is 6706 kip-in. when three strands are fractured due to corrosion, as was the case at the south load point. That moment for flexural cracking is about 3% less than the corresponding cracking moment for the box section of Fig.57(c). However, the shear required to propagate that crack diagonally through the web of the girder is much greater for a solid web than for a box section. The corresponding shears are 42.5 kips and 10.0 kips, respectively. Without the solid web, the calculated flexure-shear cracking load at the south load point is 45.8 kips. With a solid web, that shear is 77.4 kips. For a shear crack in the voided section immediately adjacent to the south face of the south load point, the calculated flexure-shear cracking load is 50.1 kips. That value and the position for which that flexure-shear cracking load are computed to be a minimum are in good agreement with the strength and location for cracking observed in the test. Calculations for the foregoing flexure-shear strength values are included in Appendix A.6.

The length of the internal diaphragms depends on the skew of the bridge. For a bridge of the same length as the test beam, but without skew, the internal diaphragm would have less than half the length of the diaphragm of the test beam since the diaphragm then needs to accommodate only one, rather than two, transverse rods. In such a beam the flexure shear cracking capacity would be more critical than in the test beam.
There was a finite probability of shear failure of the test beam since theoretically the shear capacity of the deteriorated beam was slightly less than its flexural capacity. The shear reinforcement provided in the test beam was 2-#3 Grade 40 bars at 15-inch spacing. The shear capacity provided by that reinforcement is 13.9 kips and that value is slightly greater than the required minimum shear reinforcement of 10.7 kips. Thus the calculated flexure-shear strength of the test beam for the critical location of that crack is 64.0 kips for the as built beam of Fig. 57(b) and slightly less than the flexural capacity of 57.7 kips for the section of Fig. 57(c). However, if there had been no internal solid diaphragm, or the applied loads had been arranged slightly differently, the flexure-shear capacity would have been 59.7 kips if the shear reinforcement was fully effective. If the shear reinforcement had been only 50% effective, as might reasonably be concluded from the degree of corrosion shown for the stirrups in Fig. 49, that flexure-shear capacity would have been only 52.8 kips and less than the theoretical flexural capacity of 55.34 kips.

The mode of failure of the beam did not provide direct evidence that a flexure-shear failure occurred. However, the growth of a flexure-shear crack causes a concentration of strains on the compression face of the beam and could have been the cause of the crushing that developed. The development of an incipient flexure-shear failure would explain why the deflections for concrete crushing and the crack widths at crushing were less than those anticipated for a flexural failure. The testing was stopped because of the extension limits of the jacks. If the deflections had been pushed beyond those applied in the test a flexure-shear failure may well have occurred. Because attention during the test had been concentrated on the cracking on the soffit of the beam, the extent to which the flexure-shear crack opened during the final loading was not observed.

The comparisons of this section demonstrate that even if transverse load distribution between girders is lost, the in place load testing of deteriorated girders is unlikely to be able to identify the number of corroded strand in a beam. Therefore in place testing is also likely to be unable to identify the likely reduction in ultimate capacity due to strand failure, unless sophisticated instrumentation can be used that identifies local as opposed to general cracking.

9. Conclusions

Based on the results of this study it is concluded that:

1. All evidence was that deterioration of the beam had been caused by penetration of salts from the outside to the inside of the girder as a result of the evaporation of salt laded water from the bottom of the leaking joint between the two girders. There was no evidence of deterioration from the inside towards the outside due to leakage through the top flange of the beam or freeze-thaw damage due to saturation of the cardboard that formed the central void of the beam.

2. The degree of corrosion of the strands was directly related to the clear concrete cover over the strand and the distance of the strand from the source of the evaporating salt solution.
3. The stiffness of the beam before cracking equaled that calculated using the actual dimensions of the cross-section in the constant moment region, neglecting any local damage.

4. The load for first cracking equaled that calculated using the properties of the weakest section within the constant moment region of the beam and therefore included the effects of any local damage. However, that cracking was also local and the stiffness of the beam did not change markedly until cracking occurred throughout the constant moment region.

5. The load for general cracking equaled that calculated using the average properties of the beam throughout the constant moment region and therefore did not include local effects due to concrete spalling or corrosion fracture of the prestressing strands.

6. The post-cracking stiffness also equaled that calculated using the average properties of the beam in the constant moment region and therefore neglecting local effects due to concrete spalling or corrosion fracture of the prestressing strands.

7. The load for failure was about 4% less than the flexural strength calculated using the properties of the weakest section within the constant moment region. Further the deflection at failure due to crushing of the concrete was only about 40% of that calculated using the properties of the weakest section within the constant moment region. Three possible reasons were identified for the over estimation of the ultimate capacity and mid-span deflection for failure: (1) the effects of cumulative damage caused by applying several cycles of severe loading to the beams; (2) the development of tied arch action in the constant moment region at high loads due to the presence of the stiff internal diaphragms at each end of that region; and (3) limitations of the test setup which resulted in a tensile axial force being applied to the constant moment region when large deformations developed.

8. The test setup was configured to investigate flexural behavior. The test results showed that in practice the flexure-shear strength could also have been a limiting consideration. The test setup was not configured to give the worst flexure-shear strength loading condition.Corrosion damage to the stirrups and tendons may have caused the crushing of the concrete observed at failure to be associated with an incipient flexure-shear failure. The development of such a failure would account for the deflections and crack widths at failure being less than those expected for a flexural failure.

9. Stirrups were observed to have fractured due to corrosion, or to have had significant reductions in areas due to corrosion, in many locations adjacent to where the strands had fractured. Japanese research has shown that in pretensioned beams subjected to salt spray, corrosion initiates on the stirrup because it is located closest to the beam’s surface. That corrosion then quickly transfer itself to the prestressing strand in electrical contact with the corroding stirrup because the high stress in the strand makes that strand more vulnerable to corrosion than the stirrup. The small wire diameters that are used for the seven wires of the prestressing strand loose section faster with corrosion than the stirrup steel, leading to rapid failure of the strands once the adjacent electrically connected stirrup is attacked. That behavior is clearly observable in the corrosion documented in Figs. 55 and 56.
10. Unless sophisticated instrumentation is used that can identify the effects of local as opposed to general cracking, the in-place testing of the beams in a deteriorating bridge is unlikely to be able to identify the number of strands in that beam that have fractured due to corrosion. Therefore the in-place load testing of pretensioned deck beam bridges is unlikely to be able to identify the reduction in safety caused by corrosion fracture of some of the prestressing tendons in one or more beams of that bridge.

References


4. Building Code Requirements for Structural Concrete (318-99) and Commentary (318R-99), American Concrete Institute, Farmington Hills, MI, 1999.

5. ACI Committee 318, “Commentary on Building Code Requirements for Reinforced Concrete (ACI 318-63),” SP-10, American Concrete Institute, Farmington Hills, MI, 1965, pp.78-84.
**TABLE 1 - Results of Strand Tests**

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Yield Force*</th>
<th>Ultimate Strength</th>
<th>Ultimate Strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>No.</td>
<td>-kips</td>
<td>-kips</td>
<td>(ksi)**</td>
</tr>
<tr>
<td>1</td>
<td>25.5</td>
<td>30.25</td>
<td>280</td>
</tr>
<tr>
<td>2</td>
<td>24.5</td>
<td>30.44</td>
<td>282</td>
</tr>
<tr>
<td>3a</td>
<td>24.5</td>
<td>30.00</td>
<td>278</td>
</tr>
<tr>
<td>Avg.</td>
<td>24.8</td>
<td>30.23</td>
<td>280</td>
</tr>
<tr>
<td>Specified Properties</td>
<td>24.3</td>
<td>27.0</td>
<td>250</td>
</tr>
</tbody>
</table>

*Value at 1% strain

**Based on nominal area of 0.108 in²

***Value specified in ASTM A416

Modulus of elasticity = 27,800,000 psi
Figure 1a   IDOT Archived Plans for Test Beams
STATE OF ILLINOIS
PUBLIC WORKS & BUILDINGS
BEN MOE HIGHWAYS

![Diagram of beam assembly with reinforcement details]

**GENERAL NOTES**

- Prestressing steel shall be non-galvanized high strength, stress-relieved 7-wire strand.
- The nominal diameter shall be 0.56 in, and the nominal cross-sectional area shall be 0.03 sq in.
- Lifting hooks shall be 90° diameter, 6x19 class wire rope with fiber core and shall have a minimum ultimate tensile strength of 18,000 lbs.
- The 14 rods in the transverse tie assembly shall be tightened to a snug fit and the threads set. Pockets that receive transverse tie bar on outside beam shall be filled with grout after transverse tie assembly is in place.

**BAR LIST**

<table>
<thead>
<tr>
<th>Bar</th>
<th>No.</th>
<th>Size</th>
<th>Length</th>
<th>Shape</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>83</td>
<td>8</td>
<td>83&quot;</td>
<td></td>
</tr>
<tr>
<td>B</td>
<td>3</td>
<td>6</td>
<td>31/2&quot;</td>
<td></td>
</tr>
<tr>
<td>C</td>
<td>3</td>
<td>6</td>
<td>31/2&quot;</td>
<td></td>
</tr>
<tr>
<td>D</td>
<td>3</td>
<td>6</td>
<td>31/2&quot;</td>
<td></td>
</tr>
<tr>
<td>E</td>
<td>8</td>
<td>8</td>
<td>100&quot;</td>
<td></td>
</tr>
<tr>
<td>F</td>
<td>8</td>
<td>8</td>
<td>100&quot;</td>
<td></td>
</tr>
</tbody>
</table>

*For one beam only*

**BEAM DETAILS SPANS 1/3**

S.B. Pt 168 Sec 115B-1
FULTON COUNTY
STATION 486+16.40

*For exterior beam only*
Figure 2. Condition of Test Beams in Storage Yard Shortly Before Test.
Figure 3
Soffit of Interior Beam (0-4 ft) Before Test
Figure 3. Soffit of Interior Beam (0-4 ft.) Before Test.

Figure 4. Soffit of Interior Beam (3-7 ft.) Before Test.
Figure 6  Soffit of Interior Beam (9-13 ft.) Before Test
Figure 5. Soffit of Interior Beam (6-10 ft.) Before Test.

Figure 6. Soffit of Interior Beam (9-13 ft.) Before Test.
Figure 7. Soffit of Interior Beam (11-15 ft.) Before Test.

Figure 8. Soffit of Interior Beam (14-18 ft.) Before Test.
Figure 9. Soffit of Interior Beam (17-21 ft.) Before Test.

Figure 10. Soffit of Interior Beam (20-24 ft.) Before Test.
Figure 11. Soffit of Interior Beam (23-27 ft.) Before Test.

Figure 12. Soffit of Interior Beam (27-31 ft.) Before Test.
Figure 13. Soffit of Interior Beam (29-33 ft.) Before Test.

Figure 14. Soffit of Interior Beam (32-36 ft.) Before Test.
Figure 15. Soffit of Interior Beam (35-39 ft.) Before Test.
Figure 16. Soffit of Interior Beam (37-42 ft.) Before Test.

Figure 17. Soffit of Interior Beam (40-45 ft.) Before Test.
Figure 18. Soffit of Interior Beam (44-48 ft.) Before Test.

Figure 19. Soffit of Interior Beam (47-51 ft.) Before Test.
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Δ     Deflection Measurement Location
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Load - Strain Relationship for Strand Sample #2
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Load - Strain Relationship for Strand Sample #3
Figure 45. Appearance of Tensile Flange of Less Deteriorated Beam After Testing.
From 27 to 39 ft. from South End.

From 39 to 52 ft. from South End.

Figure 46. Appearance of Tensile Flange of Less Deteriorated Beam After Testing.
Figure 47. Appearance of Tensile Flange of More Deteriorated Beam After Testing and Removal of Any Loose Concrete Over Strands.
Figure 48. Appearance of Tensile Flange of More Deteriorated Beam After Testing and Removal of Any Loose Concrete Over Strands.
Figure 49  Strand and Wire Fractures and Areas Where Loose Concrete Was Removed After Testing for Tensile Flange of More Deteriorated Beam

- **Strand Condition**
  - * Completely Severed
  - O Broken Wires < 50%
  - ▲ Broken Wires > 50%

- **Stirrup Condition**
  - * Fractured
  - ▲ Area Reduction > 50%
Figure 50. Use of Crane to Rotate More Deteriorated Beam.
Figure 51. Appearance of Ends of Sawn Section for More Deteriorated Beam.
Figure 52  Half Scale Diagram of Strand Locations and Concrete Thickness for Tensile Flange of More Deteriorated Beam

Average Depth to First Strand Layer = 1\% inches

Break Line

# 4 Bar

5\%8"

3\%4"

1\%4"

3\%2"

4\%4"
Figure 53. Ends of Sawn Section Taken Two Weeks after Photographs of Figure 51.
Figure 55  Broken Wires in Corroding Strands of Less Deteriorated Beam
Figure 56  Location of Strands in Leaking Joint Corner of Less Deteriorated Beam
### SECTION PROPERTIES

<table>
<thead>
<tr>
<th>Description</th>
<th>(a) As Specified</th>
<th>(b) As Built</th>
<th>(c) Deteriorated As Built</th>
<th>(d) Deteriorated As Built At Failure Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>Area, $A_c$, in$^2$</td>
<td>522.75</td>
<td>514.25</td>
<td>514.25</td>
<td>483.5</td>
</tr>
<tr>
<td>Moment of Inertia, $I_g$, in$^4$</td>
<td>48,237</td>
<td>47,689</td>
<td>47,689</td>
<td>43,703</td>
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<tr>
<td>Effective Depth, $d_e$, in.</td>
<td>23.77</td>
<td>24.04</td>
<td>23.83</td>
<td>23.83</td>
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<tr>
<td>Eccentricity, $e$, in</td>
<td>10.12</td>
<td>9.85</td>
<td>9.64</td>
<td>9.60</td>
</tr>
<tr>
<td>Prestress Force, $F_{sc}$, kips</td>
<td>317.5</td>
<td>317.5</td>
<td>272.2</td>
<td>272.2</td>
</tr>
<tr>
<td>$(ec_g / I_g) (10^3)$, in$^{-2}$</td>
<td>2.801</td>
<td>2.645</td>
<td>2.589</td>
<td>2.805</td>
</tr>
<tr>
<td>$I_g / c_g$, in$^3$</td>
<td>3613</td>
<td>3723</td>
<td>3723</td>
<td>3422</td>
</tr>
<tr>
<td>Cracking Moment, $M_{cr}$, kip-in.</td>
<td>7642</td>
<td>7729</td>
<td>6898</td>
<td>6660</td>
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<tr>
<td>Cracking Load, $P_{cr}$, kips</td>
<td>37.64</td>
<td>38.07</td>
<td>33.98</td>
<td>32.80</td>
</tr>
<tr>
<td>Ultimate Moment, $M_u$, kip-in.</td>
<td>12,663</td>
<td>12,807</td>
<td>11,715</td>
<td>11,236</td>
</tr>
<tr>
<td>Ultimate Load, $P_u$, kips</td>
<td>62.37</td>
<td>63.08</td>
<td>57.70</td>
<td>55.34</td>
</tr>
</tbody>
</table>
APPENDIX A.1 Nominal Strength of Beams Based on Specified Properties

Specified Properties – Fig. 1

Overall depth = h = 27 in.
Reinforcement: Grade 250 −7/16-inch diameter strands stressed to 18,900 lbs each.
14 strands at 1.75 inches up, 5 at 3.25 inches up, and 2 at 13.5 inches up.

Location of strand centroid, \( d_p \): 
\[
\text{h-}d_p = [14(1.75) + 5(3.25) + 2(13.5)]/21 = 3.23 \text{ in.}
\]

\[
d_p = 27.00 - 3.23 = 23.77 \text{ in.}
\]

Steel stress at failure, \( f_{ps} \), per Eq. (18-3) of ACI 318-99 ignoring compression steel
\[
f_{ps} = f_{pu} \{1 - 0.5(A_{ps}/bd_p)f_{pu}/f'c\}
\]
\[
= 250\{1 - 0.5 (21\times0.108/35.25\times23.77)250/5\} = 233.13 \text{ ksi}
\]

Stress block depth at failure, a: \( a = A_{ps} f_{pu}/b \ 0.85 \ f'c = 21\times0.108\times233.13/35.25\times0.85\times5 \)
\[
= 3.53 \text{ in.}
\]

Nominal flexural moment capacity, \( M_n = A_{ps} f_{ps} (d_p - a/2) = 2.268\times233.13\times(23.77-1.77) \)
\[
= 11,630 \text{ kip-in.}
\]

Nominal shear capacity, \( V_n \), corresponding to \( M_n \): \( V_n = 11,630/17.08\times12 = 56.74 \text{ kips.} \)

APPENDIX A.2 Derivation of Modulus of Rupture and Effective Prestress from Measured Load-Deflection Results

Measured load-deflection result is that of Fig. 32.

For more deteriorated beam, cracking moment, \( M_{cr} = 6,770 \text{ kip-in} \)
Decompression moment, \( M_{dc} = 4,961 \text{ kip-in} \)

For as-designed properties, cross-sectional area, \( A_g = 522.8 \ \text{in}^2 \)

Moment of inertia, \( I_g = 48,237 \ \text{in}^4 \)

For first cracking of more deteriorated beam 
Modulus of rupture, \( f_r = (M_{cr} - M_{dc})y/I_g = (6,770 - 4,961)13.5/48,237 = 0.506 \text{ ksi} \)

From Eq.(9-9) of ACI 318-99, \( f'_c = (f_r/7.5)^2 = 4552 \text{ psi.} \)

For general cracking of more deteriorated beam, \( M_{cr} = 38.0\times203.03/7,715 = 7,715 \text{ kip-in.} \)

\( f_c = 0.771 \text{ ksi} \) and \( f'_c = 10,568 \text{ psi.} \)

N.B. For loading geometry utilized maximum moment was under the load point at 17ft-1in. from the north reaction. For equal loads at each load point, the moment at the north load point in kip-in. was the load in kips times 203.03 in. The moment under the south load point is the load in kips times 201.42 in.

Effective Prestress: \( F_{sc} = (M_{dc} y/I_g)/(1/A_g + e/(I_g/y)) \)
\[
= (4961\times13.5/48,237)/(1/522.8 + (13.5 - 3.23)/(48,237/13.5))
\]
\[
= 290.5 \text{ kips}
\]
APPENDIX A.3  Prestress Losses

Calculations per PCI Design Handbook, 5th Edition, Sec. 4.7.3.

T.L. = ES + CR + SH + RE

Where T.L. = total loss; and
ES = elastic shortening; CR = creep; SH = shrinkage; and RE = relaxation

Initial allowable stress in tendons = 18,900 / 0.108 = 175,000 psi.
That value is the allowable stress after elastic shortening, i.e., ES = 0.

\[ SH = (8.2 \times 10^{-6}) E_s (1 - 0.06V/S)(100 - RH) \]
\[ V/S = \text{volume/surface} = (36 \times 27)/(36 \times 2 + 27 \times 2) = 7.71 - \text{(Box is assumed unvented)} \]
\[ RH = \text{relative humidity} = 70\%, \ E_s = \text{modulus of elasticity of steel} = 28.5 \times 10^6 \text{ psi} \]
\[ SH = 3,768 \text{ psi} \]

\[ CR = K_{cr}(E_s/E_c)(f_{c_{ir}} - f_{c_{ds}}) \]
\[ E_c = 57,000 \sqrt{f_c} \]
\[ E_c = 57,000 \sqrt{6,810} = 4,700,000 \text{ psi}; \ K_{cr} = 2; \]
\[ f_{c_{ir}} = \text{net compressive stress in concrete at centroid of prestressing steel} \]
\[ = P_i/A_g + P_i e^2/I_g - M_g e/I_g \]
\[ \text{where } P_i = \text{initial prestress force} = 21 \times 18.9 = 396.9 \text{ kips} \]
\[ M_g = \text{bending moment due to dead weight of member} = (522.8/144) \times 0.150 \times (523)^2/(12/8) \]
\[ = 2234.4 \text{ kip-in.} \]
\[ f_{c_{ir}} = 396.9/522.8 + 396.9 \times 10.27^2 / 48,237 - 2234.4 \times 10.27/48,237 = 1.151 \text{ ksi.} \]
\[ f_{c_{ds}} = \text{stress in concrete at c.g.s. due to all superimposed dead loads applied after member} \]
\[ \text{has been prestressed = effect of asphalt wearing surface = 55 psi.} \]
\[ CR = 2 \times (28.5/4.7)(1151 - 55) = 13,890 \text{ psi.} \]

\[ RE = [K_{re} - J (SH + CR)]C; \]
For 250 grade stress-relieved strand, \( K_{re} = 18,500, \ J = 0.14; \ C = 1 \)
\[ RE = 18,500 - 0.14(3,768 + 13,890) = 16,028 \text{ psi.} \]


Take \( f_{se} = \) effective prestress in prestressing steel = 175,000 - 33,686 = 141,314 psi
Use \( f_{se} = 140,000 \text{ psi.} \)
APPENDIX A.4 Strength of Deteriorated As-Built Section (Fig. 57c) Based on Measured Material Properties

Section properties: \( A_g = 514.25 \text{ in}^2 \) and \( I_g = 47,689 \text{ in}^4 \) because: 1. The webs at 4.25 in. wide were smaller than the specified width of 4.5 in., and 2. The cgc at 12.81 in. from the bottom was lower than specified value of 13.35 in. due to the central void having floated up within the section.

As shown on Fig. 52, three strands in first layer were ruptured at failure section due to corrosion. The remaining strands in first layer were at a depth of 1.5 inches, those in the second layer were at 3.0 in., and those in the third layer were at 13.0 in.

\[
\text{Hence } h-d_p = 27 - \{(11x1.5) + (5x3) + (2x13)\}/18 = 23.83 \text{ in.}
\]

\[
\text{From Eq. (18-3) of ACI 318-99, ignoring effect of compression steel and using measured } f_{pu} \text{ (Table 1) of 30.23/0.108 = 280 ksi,}
\]

\[
f_p = 280\{1 - 0.57 [(0.108x18x280/(35.25x23.83x6.81) + 0.4x40/23.83x35.25x6.81)]\},
\]

where measured \( f'_{c} = 6,810 \text{ psi, } \gamma_p = 0.4, \) and \( \beta_1 = 0.70. \) \( \gamma_p/\beta_1 = 0.57. \)

\[
f_p = 262 \text{ ksi.}
\]

Also for 2-No.4 bars at depth of 16.0 in., \( f_y = 40 \text{ ksi} \)

Rectangular stress block depth = \((0.108x18x262 + 2x0.2x40)/ (35.25x0.85x6.81)\)

\[
= 2.58 \text{ in. } \text{< top flange depth of 4.75 in.}
\]

Nominal moment capacity, \( M_n = 0.108x18x262x(23.83-2.58/2) + 0.4x40x(16-2.58/2)\)

\[
= 11,480 + 235 = 11,715 \text{ kip-in.}
\]

Ultimate load, \( P_u = 11,715/203.03 = 57.70 \text{ kips.} \)

APPENDIX A.5 Uncracked and Cracked Section Deflections.

**Uncracked Section:** Curvature = \( \phi = M/E_g I_g \)

At general cracking load of 38.0 kips, \( M_{gcr} = 38.0x203.03 = 7715 \text{ kip-in.} \)

\( E_g = 57,000\sqrt{6,810} = 4,704 \text{ ksi, } I_g = 47,689 \text{ in}^4, \phi_{gcr} = 7715/4704x47689 = 3.44 \times 10^{-5} \text{ in}^{-1} \)

Assume slope of elastic curve is zero at midspan.

\( \Delta_{gcr} = \text{moment of area of } \phi \text{ diagram between end and midspan taken about end} \)

\[= \text{moment of area between load and midspan + moment of area between load and end} \]

\[= [84x(205 + 42) + 205x0.5x205x0.667] \times 3.44 \times 10^{-5} \]

\[= 1.196 \text{ in.} \]

Measured deflection (Fig. 58) = 1.2 in.

**Cracked Section Deflections**

Calculations per PCI Handbook, 5th Edition, Sec. 4.8.3.

Cracked section stiffness, \( I_{cr} = nA_p d_p^2 (1 - 1.6\sqrt{n}) p_p; \quad p_p = 0.108x18/35.25x23.83 = 0.0023 \)

\( n = 28.5/4.7 = 6.06; \quad I_{cr} = 6.06x0.108x18x23.83^2 (1 - 1.6\sqrt{6.06x0.0023}) = 5,426 \text{ in}^4 \)

For load change from 38.0 to 50 kips, change in moment = \( 12x203.03 = 2,436 \text{ kip-in.} \)

\( \phi_{change} = 2,436/4704x5,426 = 9.55x10^{-5} \text{ in}^{-1}, \quad \Delta_{change} = 1.196x9.55/3.44 = 3.32 \text{ in} \)

\( \Delta \text{ at 50 kips} = 1.20 + 3.32 = 4.52 \text{ in.} \)
APPENDIX A.6 – Shear Strength Based on Measured Material Properties

The nominal shear strength is evaluated per ACI 318-99 procedures. The nominal shear strength, \( V_n = V_c + V_s \), where for this beam the shear at inclined cracking is controlled by flexure shear so that \( V_c = V_{ci} \), and \( V_s \) is the contribution of the shear reinforcement.

\( V_{ci} \) is evaluated using the original empirical expression from which Eq. 11-10 of ACI 318-99 was derived. That expression is reported in University of Illinois Engineering Experiment Station Bulletin 493, 1967, “Investigation of Prestressed Reinforced Concrete for Highway Bridges, Part IV: Strength in Shear of Beams with Web Reinforcement.” by S. Olesen, M.A. Sozen and C.P. Siess.

For a beam subjected to concentrated loads
\[
V_{ci} = 0.6 b d_p V_{c} f_y + M_{cr} \left( \frac{M}{V} - d_p/2 \right).
\]
For the test beam \( M/V \) is the ratio of the moment to the shear at the south load point and equals 201.42 in. \( V_{ci} \) equals the shear to cause flexural cracking at a distance \( d_p \) outside the south load point, plus the shear increment for that crack to incline and penetrate through the web to the upper flange.

From Fig.57c, \( M_{cr} = 6,898 \) kip-in.

\[
egin{align*}
V_{ci} &= 0.6 \times 4.25 \times 2 \times 23.83 \times \sqrt{6810} + 6,898/(201.42-23.83/2) = 10.02 + 36.40 = 46.42 \text{ kips} \\
V_s &= A_s f_c d_p/\ell = 0.22 \times 40 \times 23.83/15 = 13.98 \text{ kips} \\
V_c &= 46.42 + 13.98 = 60.40 \text{ kips} \\
P_v &= 60.40/1.0096 = 59.82 \text{ kips}.
\end{align*}
\]

In test beam the stirrup on one side of the beam had ruptured due to corrosion. Hence, \( V_s = 6.99 \) kips and \( V_c = 53.41 \) kips. \( P_v = 52.9 \) kips.

However, at the location \( d_p/2 \) outside the south load point, and at that load point, there was a solid diaphragm. The shear at flexure-shear cracking for a solid section is computed to be 77.4 kips. The increase in capacity from 53.41 to 77.4 kips is due almost entirely to a computed increase in the shear to cause the inclined crack to penetrate through a solid, rather than a voided, section. The shear capacity associated with the situation where the flexural crack starts in a voided section and must subsequently penetrate through a solid section has not been systematically studied in the laboratory. Therefore the realism of using an expression like Eq. 11-10 to predict the flexure-shear cracking strength is unknown. \( P_v \) can be expected to be greater than 52.9 kips but may not be much greater than that value.
APPENDIX A.7 Chloride Content Measurement Results

Results of the chloride content measurements are given in Table A.7.1 and are plotted graphically in Fig. A.7.1. Chloride values range from 0.0290 to 0.4261 percent by concrete weight. Values above about 0.03 to 0.04 percent by concrete weight promote corrosion of embedded unprotected steel in concrete.

In general, in Fig. A.7.1, the profiles for the vertical edges of the beam show maximum chloride contents at depths of 0.75 to 1.25 inches from the surface. That result suggests that there is some leaching away of the chlorides by the water seeping down the vertical surface of the beam. That effect is much more marked for the vertical edge PNE, on which no spalling occurred, than for the edge PSE, where spalling occurred at the corner between which samples PSE1 and PBE1 were taken.

On the soffit of the beam the chloride contents at location PBE1, close to where spalling occurred, are 10 times the trigger value of 0.03 to 0.04 percent by concrete weight. Further those contents remain high through to a depth of 1.25 inches which was the depth to the surface of the No.4 stirrup bars surrounding the prestressing strands. By contrast, in the opposite corner of the beam, at the location PBW1, the chloride contents drop from unacceptable values of about 0.23 at the surface and to a depth of 0.75 inch, to acceptable values at 1.25 inch depth and greater. Thus, consistent with visual observations, corrosion was to be expected of both the stirrup steel and the prestressing strands in the PBE corner of the beam, but not in the PBW corner.

In the webs of the beam, chloride contents do not reach acceptable values until depths of about 2.75 inches from the vertical surface of the beam. Further, on the PNE side, average chloride contents for the first 2.5 inches of depth are almost constant over at all three sample locations, PNE1, PNE2 and PNE3. By contrast, on the PSE side, average chloride contents for the first 2.5 inches decrease markedly with distance from the soffit of the beam. While the values at location PSE1 are comparable to those for all PNE locations, the values at locations PSE2 and PSE3 are only 65% and 37%, respectively, of that at location PSE1. These results suggest that, while the total amount of salt being washed through the longitudinal joints on either side of the beam was about the same, on the PNE side the major evaporation was occurring on the vertical face. By contrast, on the PSE side the major evaporation was occurring at the corner of the beam. This result may explain the marked salt crystallization observed for that side of the beam and shown in Fig. 2. The same degree of salt crystallization was not observed for the PSE side of the beam.

No.3 Bars were used for stirrups in the beam. From Fig. 5.2 it can be seen that the clear cover to that stirrup on the soffit of the beam was only 1.1 inches and at that depth, the stirrup was clearly in the zone of potential corrosion on the PSE1/PBE1 corner. By contrast, the clear cover to the stirrup from the vertical faces of the beam was 2.25 inches. At that depth, it can be seen from Fig. A.7.1 that attack of the stirrup was likely only for about the bottom 4 inches of the beam. Above that depth, the chloride values at 2.25 inch depth were generally insufficient to initiate corrosion.