Event Report

FHWA Office of Bridges and Structures

Subject: Closure of I-276 Delaware River Bridge (PA Turnpike and NJ Turnpike)				
Date of Event: January 20, 2017			**Location:** Bucks County, PA	
Discipline:	Structural Design	Structural Inspection	Geotechnical	Hydraulic
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Audience:	For Internal Use		Public	
Relevant Policy or Guidance: None				

Summary:

The Delaware River Bridge, which carries I-276 at the Pennsylvania and New Jersey border, is owned jointly by the Pennsylvania Turnpike Commission (PTC) and the New Jersey Turnpike Authority (NJTA). On January 20th, 2017, a construction inspector for an active painting job noticed a full-depth fracture in the top (tension) chord on one of the Pennsylvania deck truss approach spans, where the truss is continuous over the pier. The bridge and roads running under it were closed to all traffic. The fracture had initiated at the site of a fabrication defect, two holes drilled through a flange that had been partially filled with weld material. The response to date has been to begin constructing shoring towers, for stability and for jacking the truss for the repair, to test the material, to begin hands-on inspections looking for additional defects, to begin 3D modelling for analysis, and to design a splice-plate repair.

Current Report [February 6, 2017]

Bridge Information:

Figure 1 Main river span

The Delaware River Bridge carries I-276 across the Pennsylvania/New Jersey border, the Delaware River, PA State Route 13, several local roads, and Amtrak. The bridge opened in 1956 and is owned jointly by the New Jersey Turnpike Authority (NJTA) and Pennsylvania Turnpike Commission (PTC), with each state owning up to the state line. 2014 average daily traffic was 42,000 vehicles.

The bridge has 31 spans, with a total structure length of 6,751' and a main river span of 682'. The overall deck width is 80'. As shown in Figure 1, the main river span is a distinctive arch-shaped through truss, with suspended deck and vertical clearance of 135'.

The first ten spans on the PA side and the first four on the NJ side are standard girder/floorbeam construction. The next seven spans on either approach are steel deck truss, with 4-span and 3-span continuous units. The remaining three spans are the main river span and two hybrid thru truss/deck truss spans leading up to the arch. The center two spans of the 4-span continuous unit are shown in Figure 2 on the following page, which was the location of the critical finding. (Interesting aside– PennDOT pointed out that the bridge is also featured on the cover of the 2014 LRFD spec.)

The bridge is under a painting contract, with the PA side completed in summer 2016, and the NJ side currently underway. Based on the 2014 fracture critical inspection, the bridge was in overall fair condition with Item 58-Deck condition rating 7, Item 59-Superstructure condition rating 5, and Item 60-Substructure condition rating 6.

NJTA is responsible for inspection of the entire bridge, and each Turnpike is responsible for specific maintenance and construction costs in accordance with an existing ownership agreement. There is also an agreement for the current project that has NJTA responsible for design work, which is why the NJTA is utilizing their consultants. Some more recent rehabilitation efforts noted in the inspection report include re-decking, latex modified overlay, median barrier replacement, and routine substructure and superstructure rehab over the life of the bridge. On the PA approach, truss and gusset strengthening was performed in the early 2000s, including replacing riveted connections with A325 bolts.

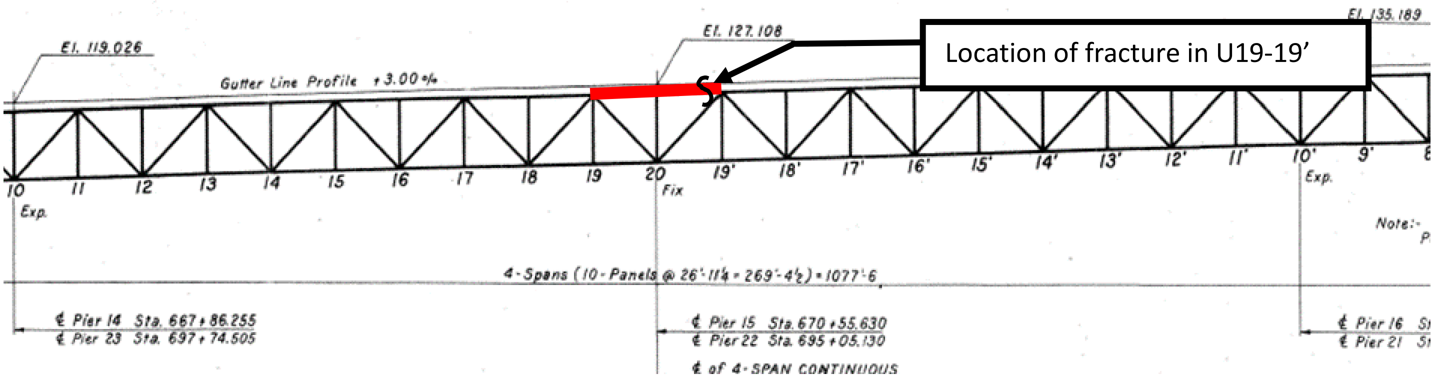

Figure 2 Deck Truss elevation view, Spans 15 and 16 (as-builts). East to the right, north into the page.

<u>Incident:</u>

On Friday, January 20th at approximately noon, the resident engineer on the active painting job noticed that the upper chord on one of the PA deck truss spans was fractured. Second hand, we have heard that, when asked, the painting contractor reported hearing a huge noise and feeling the structure and surrounding ground shake back prior to Christmas. This timeframe also corresponds with the last cold weather spell. Ultimately, it will probably never be determined when the fracture occurred precisely, but the bridge was under live load for some time with the fractured top chord.

The fracture occurred on the north truss face, on the 16th span, just prior to panel point 19'. The member is designated U19-19' and is a rolled 14WF314 that is weak axis oriented. Figure 3 shows the fracture in the field.

The Divisions were informed of the critical finding in the afternoon on Friday, January 20th, and began coordinating with their respective Turnpikes. Joey Hartmann and the Divisions made a site visit and were briefed by the Turnpike consultants on Tuesday, January 24th.

Figure 3 Fracture of U19-19', North Truss, Looking South and East at Panel Point 19', prior to any material sampling. (HNTB)

<u>Immediate Actions:</u>

The bridge and the local roads running under the damaged 4-span unit on the PA approach were immediately closed. The consultant's immediate concern was the continued stability of the bridge. With weather forecasts for Monday, January 23rd calling for high winds (30-40 mph sustained with 60 mph gusts), the painting contractor was directed to remove the paint tarps, suspended platforms, and any stored materials from this and adjacent spans, to reduce potential wind loading. In order to provide some member continuity and establish a comfort level with safe worker access, the consultant designed and fitted temporary web and flange splice plates over the fractured member. The splicing operation was completed early Sunday morning. The splice is shown in Figure 4.

Figure 4 Splice plates on U19-19', looking west from panel point 19'. (HNTB)

At the site of the fracture, the consultant measured approximately 1" of lateral misalignment, 0.75" of vertical misalignment and 2.5'' of longitudinal displacement. The contractor shimmed the splice plate to make a flush connection, not attempting to bend the member or otherwise restore geometry. Samples were torch cut from each fracture face for metallurgical analysis by Lehigh University.

Based on the length of time from when the failure is presumed to have occurred to when it was discovered, and the bridge's continued performance under full live load during that period, it is unlikely that the bridge was in immediate jeopardy of loss of stability. It is clear that the fractured member was carrying no load, and the structure had redistributed the load to stable load paths. It is unlikely that the load paths would suddenly change such that the fractured member would again be relied upon for any load carrying capacity, without restoring the original truss geometry. However, the immediate splice plate repair was necessary to provide the owners and consultants with a level of comfort with the safety of workers accessing the bridge.

In parallel, the NJTA directed their consultants to begin designing shoring towers for the damaged truss spans, to begin 3D modelling of the truss, to perform LiDAR scans of the piers to determine if pier settlement had contributed, to perform materials testing to determine the cause of the fracture, to perform a full hands-on inspection to verify whether similar conditions exist elsewhere on the bridge, and to begin preparing designs for repairs and a complete superstructure replacement of the 4-span unit as a contingency.

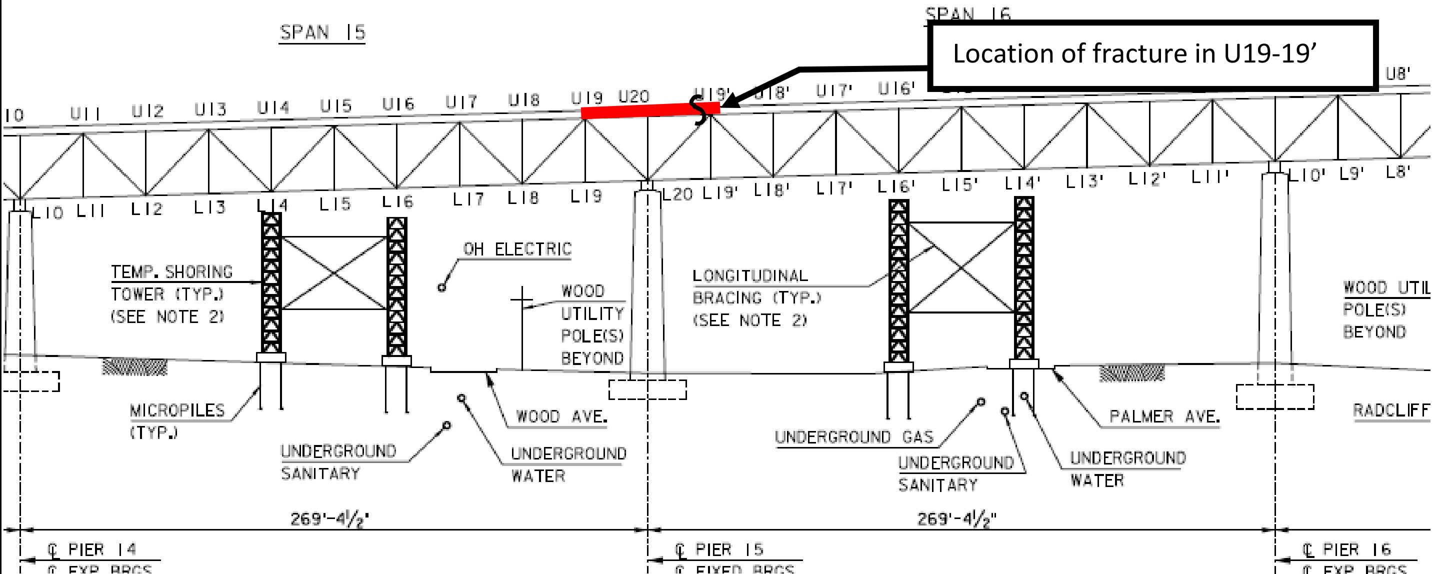

Figure 5 Shoring Tower locations, elevation looking north (HNTB)

Shoring Towers

The consultants also began designing shoring towers, to be placed in positive moment locations on either side of the pier, as shown in Figure 5. The lower chord of the truss is about 80' off the ground at this pier. They were designed to serve as a catch should the truss fail, but also provide a platform for potential jacking during the repair. A typical section is shown in Figure 6.

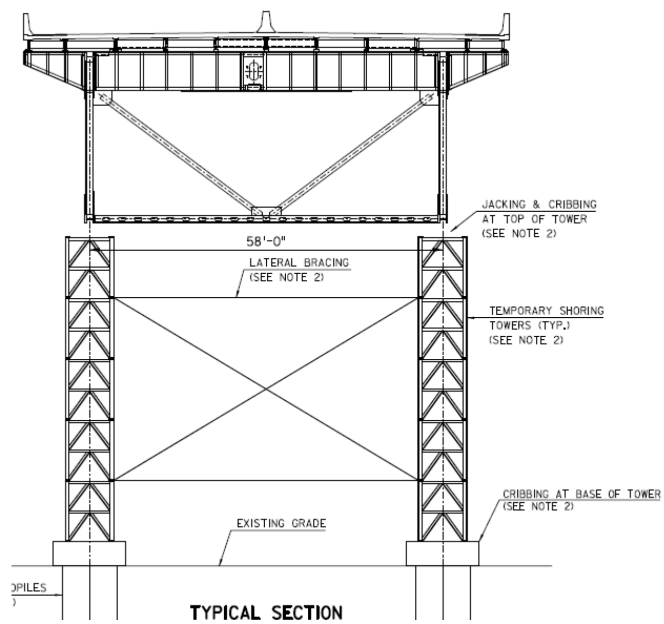

Figure 6 Shoring Tower typical section, looking east

The towers are founded on 2x3 arrays of 12.75" micropiles. Micropiles were selected for expediency, since the NJTA had a contractor with drill rigs and materials ready to go. The micropile design accommodates the bridge dead loads as well as loads induced by the jacking operation.

The towers, which are standard crane towers used to hold free standing cranes for vertical construction, are designed tosupport all imposed vertical and lateral loads and transfer to the foundation.

Tower construction required the relocation of aerial utilities. Additionally, the pile foundations had to be located around the underground utilities shown in Figure 5. Pile installation production rates were slower than initially anticipated due to the wet conditions and soft soils at the site, but foundation operations were completed by Friday, February 3rd.

Modelling:

NJTA had their on-call consultant and the project designer begin independent modelling efforts, and the PTC had one of their on-call consultants do the same for comparison. In the end, there would be 5 independent 3D linear-elastic models. The models have been consistent in this case, but the FHWA has noted that so many independent modelling efforts can sometimes cause the analysis process to get bogged down in attempts to resolve minor differences between model outputs.

The consultants' models indicated that the deck and stringers took most of the load from the damaged truss. Some load was also shed through the floor beam system to the south truss face. The top chord in the next span after the fracture, U19'-18', went from tension into compression. Field observations note this top chord has bowed or buckled between panel points U19' and U18'. There are reports of some displacement of stringers in the damaged span, but hands-on inspectors have not been able to access this area yet to provide any further observations to confirm or refine the model.

Materials Testing:

Initial visual assessment by Lehigh University indicated that the north flange of the failed member has two additional shop-reamed holes that are not present in the south flange, just past the edge of the adjacent cover plate. The holes line up with the cover plate rivet lines, so the current thought is that these holes are a fabrication error. It appears the holes had weld material placed in them and ground smooth at the surface. The weld material inside the hole did not really penetrate the surrounding steel, but is likely that the steel properties around the holes were affected by the heat of the weld material. The fracture initiated at the site of these holes and was brittle in nature, with no indication of fatigue. These holes have been called plug welds, but we note a more accurate descriptor would be weld-filled holes.


Figure 7 Sample taken from North Flange fracture surface. (HNTB)

An initial concern was that, if the holes were meant to aid in erection, they may be present in many other members. Since they were ground smooth, the defect would not have been easily detected through the paint during a visual inspection, and make it difficult now for the hands-on inspectors to find additional locations where this may have been done. Field inspection of the bridge and ultransonic testing (UT) of selected members is being performed to try to verify whether the condition exists at similar locations. Since the holes do not show up on the shop drawings or as-builts, the expectation is that these are an isolated error.


Figure 8 Close-up of north flange fracture face. (HNTB)

Plans called for the use of A-94 silicon steel; however, shop drawings indicate that a U.S. Steel proprietary alloy called Man-Ten was substituted for A-94 during construction. At this time we cannot find an ASTM reference for Man-Ten. Spec sheets from US Steel for Man-Ten indicate it is not suitable for spot welding, with weld locations having markedly lower toughness.

Charpy v-notch tests were run at 40 degrees and 0 degrees F, with 40 degree results averaging 21 ft-lbs and 0 degree results averaging 7 ft-lbs. These results are low compared to modern steel; however, there was no specification for Charpy toughness at the time the bridge was constructed. Yield strength was tested as 43 ksi and ultimate tensile strength was tested as 81 ksi. These are within spec for greater than 1.5" thick Man-Ten steel plates, as indicated by US Steel spec sheets.

Early indication from microscopy is that the material is highly variable and has high inclusion contents, which is expected for such a large and thick rolled shape from this era.

The owners plan to perform further material testing, including sampling at other locations in the bridge, to aid in their decision making process about the long-term future of this bridge. The FHWA has advised that, given the vintage and size of the member, further materials test results will probably be similarly inconsistent over the entire structure.

<u>Causes of the Fracture</u>

The FHWA believes it is unlikely that a single or satisfying answer will be determined as to the cause of the sudden fracture. It is believed that a series of factors combined to provide the conditions needed, which may have included: long term increases in dead load from deck and barrier replacement; construction loading from tarps, platforms and materials; low temperatures at the time of fracture with a relatively brittle steel; changes in structural behavior over time from strengthening or replacing riveted connections with high strength bolts; or some combination of heavy live loads. LiDAR results so far indicate that pier settlement or tilting has not occurred.

The only definitive conclusion so far is that this was a brittle fracture, which initiated in the heat affected steel around the partially weld-filled holes.

<u>Hands-On Inspection</u>

Figure 9 Tack-welded rivet shanks, member U19-19', near PP20 (HNTB)

The entire truss, on both approaches, is receiving hands-on inspection prior to the repair. Ultrasonic testing (UT) will be used in targeted locations in order to try to determine if the weld-filled holes are present in any other members. The inspectors are identifying any surface features that may need further UT investigation. Some of the similar 14WF314 shapes will have UT at locations near cover plates and panel points. They are preparing a mockup with a similar weld filled hole to help train inspectors on how this detail looks on a UT.

Some locations have been discovered on the top chord on both approaches where there are holes in the inside flange with rivet shanks filling the holes, as shown in Figure 9. The rivet heads have been removed and the top of the rivet tack welded over. It's unknown what the purpose was of these holes, if they are further fabrication errors or erection holes, or something else. These defects were not noted on the 2014 FC inspection, despite being in the FC top chord. These locations will be repaired prior to opening the bridge, by drilling out the rivet shanks, overdrilling the hole to remove the heat affected steel, then painting.

Figure 10 Elastic bowing in U19'-17', between panel points U19' and U18'. (HNTB)

No other major findings were reported on the NJ side, where the hands-on inspection of the deck truss spans has been completed.

Access on the PA approaches has been limited so far. They will need to perform the inspection from a man lift as the under bridge crane cannot safely access the deck in the damaged spans. Some hands preliminary inspection has been performed on the damaged 4-span unit, which identified some minor issues in other truss members.

In the next span beyond the fractured member, the top chord U19'-17' has an elastic buckle or bow, as shown in Figure 10. This is not unexpected since the member is relatively small (14WF87) and went into compression after the fracture. Some of the stringers have lateral displacements, and several of the stringer/floorbeam connections in these spans were broken but do not appear to have been designed structural connections.

<u>Eventual Repair Scheme:</u>

The truss has deflected vertically and laterally, and since truss load paths are highly dependent on geometry, the first step will be to lift the truss and restore the original geometry. As of this week, the consultants are preparing detailed jacking procedures. Each tower is being fitted with a 600T jack. Primary vertical jacking will come from two towers, at the north truss face at panel point 16 and 16'. Models have indicated only a few diagonal members will need to be strengthened for the jacking, and critical members have been identified for instrumentation with strain transducers.

The jacks will be engaged and provide a small amount of vertical force into the truss, at which point the strain gauge output will be compared to what the model predicts. If the model is reasonably accurate, the splice plate will be removed and the model will be proofed again to ensure the splice plate was not having any effect. From here, the jacking procedure will likely be displacement controlled, moving in 1" increments. The consultant's lead engineer will be on-site throughout jacking to make decisions as to when to stop the operation.

Post tensioning jacks will be used to longitudinally jack local to the damaged member, to attempt to draw the member sections back together. The primary jacking force in the operation will come from the post tensioning. Using primarily vertical jacking at the tower locations ran a higher risk of overstressing other members in the truss, according to the modelling.

Once the truss geometry has been restored as much as possible, new splice plates will be fitted and installed onto the member. The jacks will slowly be lowered. If the original geometry cannot be fully restored, some capacity protection (strengthening) may be necessary on other members in the adjacent truss spans as determined by the models.

After strengthening is complete, the bridge will be load tested to determine how the repaired truss is performing in comparison to the truss on the NJ side.

Total replacement of the damaged member was discussed as another possible repair scheme. However, the member is continuous across the pier, and 3 total panel points, so replacement would not be straightforward and would likely require significant stabilization. As a contingency, the Turnpikes are also having their consultant concurrently design a superstructure replacement for the 4-span and 3-span truss units, should fabrication defects found elsewhere prove to be unrepairable or if jacking does not work.

A follow-up report will be issued when the repair is complete.

Previous Reports: None.

FHWA Response: At this time, the FHWA is providing support to the investigations and permanent repairs through the Division Offices and Office of Bridges and Structures, at the request of the Turnpikes. The Divisions have also been coordinating on potential ER eligibility.

ER is eligible for use on toll facilities, but the facility becomes subject to 23 U.S.C. 129 requirements. The PTC does have a Section 129 agreement with FHWA and is eligible for federal funds; the NJTA does not. At this time, however, it does not appear to have been an ER eligible event, since there was no external cause.

Attachments: None.

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