NONDESTRUCTIVE TESTING AND MATERIALS ANALYSIS OF THE PA / NJ TURNPIKE CONNECTOR BRIDGE

FRANCESCO M RUSSO

THOMAS P MURPHY

ROBERT J CONNOR

BIOGRAPHY

Dr. Francesco M Russo is Associate Vice President and Technical Director for Michael Baker International. With over 25 years of experience in bridge engineering, Dr. Russo is responsible for complex project support nationwide including serving as the NDT and on-site engineering support services manager for the PA Turnpike for the emergency repair of the I-276 Delaware River Bridge.

Dr. Thomas P Murphy is a Vice President and the Chief Technical Officer of Modjeski and Masters. His professional experience has included the analysis, design, and detailing of a variety of bridges including cable-stayed, suspension, arch, truss, and girder bridges. Dr. Murphy has been involved in all stages of the bridge design process; from the development of design specifications, to the completion of conceptual studies for specific crossings, preliminary and final design, and construction stage issues.

Dr. Robert J Connor is a Professor of Civil Engineering and Director of the S-BRITE Center at Purdue University. He has over 25 years of experience in evaluating steel bridges, in particular in the area of fatigue and fracture, repair and retrofit, and field testing.

SUMMARY

Following the fracture of a top chord in a four-span continuous deck truss due to the presence of an unknown plug weld near a gusset plate connection, an emergency response program consisting of stabilization, repair, jacking, and partial replacement of the top chord was carried out, restoring the bridge to service in approximately seven weeks. At the same time, many questions were raised about the suitability of the truss material, a unique steel grade called Man-Ten Steel produced by US Steel in the 1950’s, and its overall fracture toughness. Questions were also raised about the possibility of other fracture initiation sites in the bridge. This paper discusses the NDT inspection of tension and reversal members of the bridge as well as the extensive material sampling and testing program that were carried out. The role of the testing was to provide a degree of confidence to the two Turnpike’s that the fracture was at an isolated location, where a plug weld had been installed in the 1950’s, and did not represent a more global problem with the mile-long steel truss bridge.

NONDESTRUCTIVE TESTING AND MATERIALS ANALYSIS OF THE PA / NJ TURNPIKE CONNECTOR BRIDGE

Introduction

The I-276 Delaware River Bridge (DRB) is a 6,571-ft., 31-span bridge joining the Pennsylvania and New Jersey Turnpike's across the Delaware River. A complete fracture was discovered in the top chord tension member of a 4-span continuous deck truss unit on the Pennsylvania approach on January 20, 2017, resulting in an immediate closure of the bridge. This paper focuses on the nondestructive testing and materials sampling and analysis that was conducted in parallel to ascertain whether other possible fracture initiation sites were present or whether substandard materials contributed to the fracture. An aerial view and section locating the fracture are provided in Figures 1 and 2.

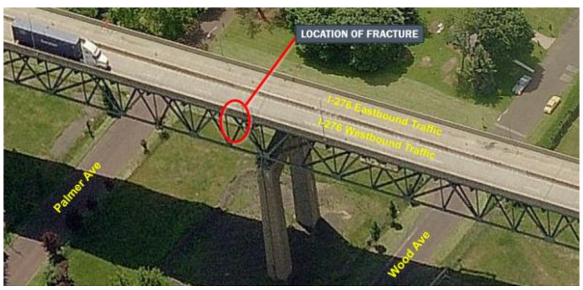


Figure 1 Aerial View of Fractured Chord Location

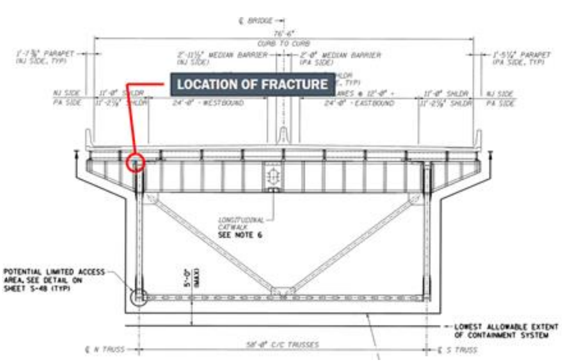


Figure 2 Cross Section of Truss Spans

Nondestructive Testing

Within the first day of the closure, photographs of the fracture site and small samples removed from the fractured surface led various parties to believe that the fracture emanated from a plug weld, or some attempt to fill a mis-drilled hole with weld metal. A photo of the fractured chord taken the first night is provided in Figure 3. Visual observation of the fracture surface provided the telltale signs of holes filled incompletely with weld metal. Fracture was hypothesized to have originated from these holes. The fracture surface is jagged and clearly originates at the two filled holes.

Figure 3 Fractured Chord

Review of the shop drawings confirmed there were to be no holes at this location. These holes are well outside the limits of the gusset plate connection, the extreme limit of which can be seen at the left side of the photo. Given that this fracture came from what was later confirmed as a hole filled with weld the concern immediately became "*if this hole exists, and was filled, is this a systemic problem or a single occurrence?*" It is not possible to answer this question other than through nondestructive testing (NDT). Thus, a decision was made that a testing program would be initiated. The initial question was what would be tested, and how quickly?

UT Testing – Project Planning and Phase I Scope of Work

The development of the UT testing scope was a collaborative effort between Dr. Robert Connor, Michael Baker International, and HNTB with input and concurrence from the PTC and NJTA. The Phase I UT Testing approach coalesced around a sampling of roughly 25% of the tension members and testing of the end 3 ft. outside the limits of the gusset plates. The 25% number was satisfied by testing two of four spans of the PA 4-span unit, on the north side only, and two of the three spans of the PA 3-span unit, on the north side only. The initial list of members is depicted in Figure 4, a working graphic developed to guide the technicians. The first two spans of the three-span unit are shown as well as two of the four spans of the four-span unit. Members highlighted with a dashed line were those chosen for testing. Members in yellow on these graphics are always in tension. Members shown as pink are in reversal. All tension members were tested except for the four-span unit member U9-U11 which has full length coverplates. This member was intentionally omitted due to the built-up redundant nature of the member. The Phase I testing was performed beginning February 3rd and was completed on February 15th.

The Phase I work included the development of UT scanning procedures as well as the fabrication of test plates at the laboratories of Purdue University for NDT technician qualification and onboarding. These tasks are described in the following text.

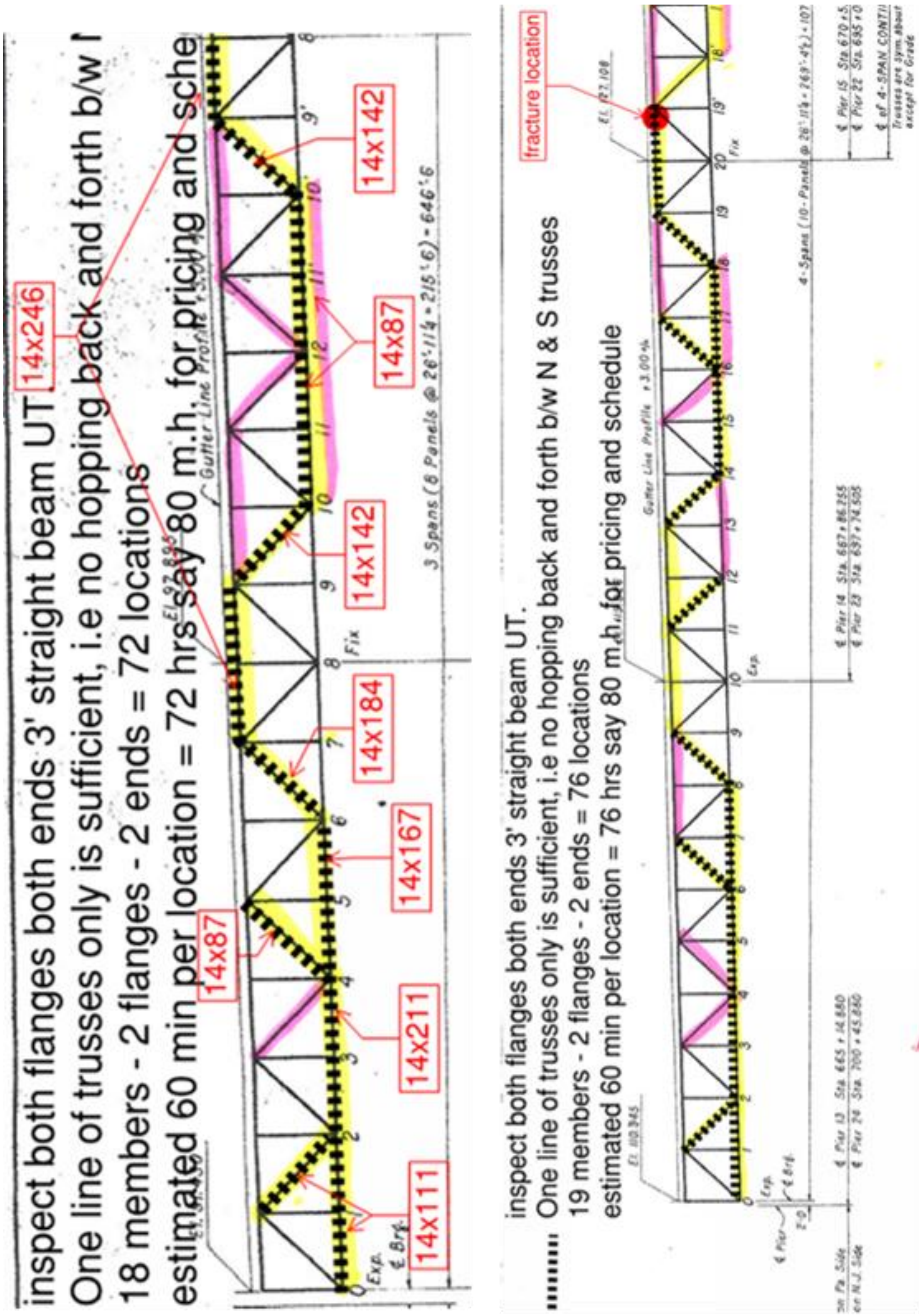
A UT testing procedure was developed by an ASNT Level III technician. It was developed with the objective of standardizing the testing of beam flanges with the intent to scan for plug welds. As articulated in the scanning procedure, the purpose was

"To establish a uniform Ultrasonic Testing approach for the discovery of major weld defects remaining in plug welded hole repairs, made during the bridge construction. Potential areas for defective weld sites being investigated, are in the flanges of W shaped rolled members."

The procedure calls for scanning each flange from the tip of the flange towards the web from the top flange edge down towards the web and from the bottom flange edge up towards the web. This is done on both flanges so four individual scans are needed at each tested location. A photo of this procedure being used in the field is provided in Figure 5. The technician is scanning across the flange thickness for a prescribed length, in towards the web, the intent being to identify any plug welds (or other indications of concern) that exist in each half, upper or lower, of a flange.

The objective of the UT test procedure was to provide a uniform UT approach for the discovery of major defects suspected to be from other weld-filled holes. The UT scanning procedure was defined as the application of a 2.25 or 5 MHz compression wave / straight beam transducer with the scan conducted along the flange edges, scanning towards the web of the W-shaped truss members. Technicians were required to meet a minimum of ASNT Level II qualifications and were subject to on-site qualification testing.

The scanning procedure included distance calibration using IIW test blocks and further validation of the procedure based on measurement to a standard 1/16 in. diameter side drilled hole in one of the Purdue test plates. This procedure for calibrating the equipment was performed on both unpainted and painted test plates; no meaningful attenuation was detected for the painted specimens. The scanning procedure detailed that technicians were to evaluate any *potential defective areas* found with straight beam using the shear wave inspection techniques. The shear wave testing was to be performed following techniques described in AWS D1.5-2008, clause 6. (part c). It was noted that AWS D1.5, Table 6.3 is the acceptance criteria for normal CJP welding subject to tensile stress and that the AWS criteria is not necessarily applicable for the evaluation of filled holes. No standardized method of testing and reporting is available for the type of work performed for this project. Strictly interpreted, AWS criteria are defined as those related to the acceptance of welds. They were used, lacking other guidance, to also apply to the recordation of findings related to the testing of base metals that would surely contain discontinuities such as those normally expected in steels from the 1950's. There was a great deal of discussion throughout the project about the terms "recordable", "rejectable", "defect", "discontinuity" and other terms commonly used to describe NDT findings. The meaning of these terms and the distinction between them can easily be misconstrued by engineers not familiar with NDT methods and the reporting and discussion of results.



Two Spans of the PA 3-span unit

Two Spans of the PA 4-span unit

Figure 4 Phase I NDT Testing Limits and Members

Figure 5 UT Scanning of Chord Flange Tips

The existence of a validated and agreed upon NDT test procedure was an important element to maintaining focus on the clarity of the intent of the testing, to seek out and record gross defects.

As part of technician qualification, each technician was required to complete a qualification test using two groups of three test plates shown in Figure 6. The testing of technicians is virtually unprecedented including for those who perform shop inspection. This qualification process was adopted as a project specific requirement to provide a high degree of confidence in the work of the various personnel who were working independently in the field. Each technician was required to test both groups of three plates. The first three plates were used to ensure their equipment was functioning and to give them some comfort in what was being asked of them. For the second three plates, each was given roughly one hour to complete a qualification test. They were asked to first scan the plates using a 90-degree compression wave (the same procedures as in the field) and note their findings. Any suspected defects must then be further located, sized and

Figure 6 Collection of Six Test Plates

characterized using a shear wave scanning procedure. The plates are described below.

Plate #1 - Side-Drilled-Hole – This plate is unpainted with a 1/16” diameter hole drilled through the thickness. This plate was used to determine detectability of a small defect in an unpainted plate. Unlike standard calibration blocks, where the location of the hole is known and whose purpose is equipment calibration, this test plate was used to determine if the scanning procedure was adequate to find small unknown defects. The hole was intentionally hidden by TIG welding the hole closed and grinding the surface to obscure its location. The UT technicians did not have advanced notice where the hole was located.

Plate #2 - Side-Drilled-Hole – This is a painted plate having a similar 1/16” drilled hole but with the hole located at a different location. The intent of this second plate was to judge if the paint system used on Bridge P-00 would impact the UT testing procedure. Plate 2, and the plug weld plate described below, were sent for painting using the same coating materials and procedures used for the field painting.

Unnumbered - Plug Weld Plate – This plate, also painted, had an intentional plug weld to test if the technicians could locate weld-filled-holes prior to asking them to do so in the field. To make this plate, the researchers at Purdue endeavored to make the “best” plug weld they could, i.e. that which would be most difficult to find. This was expected to be different than what would have happened in the 1950’s if a mistake was made and the hole plugged quickly to keep the fabrication on schedule. The test weld is of a much higher quality than visibly observed on the fractured member. The plugged hole measured 1 in. in diameter and was the full thickness of the test plate. During each pass the weld was chipped, slag removed, the plate turned upside down and cleaned with compressed air in an attempt to make a high-quality weld. The rationale was this would create a defect, that if found by the technicians, would be much harder to find than an actual filled hole in the field.

As additional testers were brought to the site, and as an additional measure of quality control, three additional plates were made and shipped to the project office. These were unpainted and tested as such. The intent was now to further test the technicians and provide additional confidence in the quality of the inspectors and procedure.

Plate #3 – This plate had no defects in it. It was intentionally made this way. It was chosen as a 2-in. thick plate to represent some of the larger flange thicknesses that would be encountered on the project. Since the technicians were using probes ranging from 0.5 – 1.0 in. diameter, this thicker plate required each technician to scan across the width of the flange to detect embedded flaws as would be required for many members in the field. Interestingly, though this plate was made with no intentional defects, some of the technicians could detect internal discontinuities anyway even in this modern high-performance-steel (HPS) base metal. This confirms that some level of discontinuity is present and acceptable in all base metals and these are unavoidable.

Plate #4 – This was also a 2-in. thick plate. This plate had various intentional and hidden defects. These included a small 1/16 in. and 3/32 in. diameter partial depth drilled hole, and two plug welds, each 1 in. diameter, each partial thickness, and drilled in from opposite plate faces.

Plate #5 – This was a 3/4 in. thick plate with a partial depth 3/32 in. diameter hole. This thin plate is representative of some of the thinner flange material and the intent was to challenge the inspectors to maintain adequate coupling to thin material while also scanning for very small defects.

UT Testing – Phase II Scope of Work

While the Phase I testing was underway a decision was made to expand the testing from 25% to 100% of the tension and reversal members on the PA side and begin the same testing on the NJ side as well. Because of the expansion of the work from 25% of end regions in PA only to 100% in both states, additional firms were called in to assist with the work. This was partly the impetus for the performance testing described previously. This testing commenced Monday February 20th. Each firm was given an orientation walk-through in the office and in the field, both on the PA and NJ sides. A revised set of plans covering all tension and reversal members was provided to all technicians to provide explicit direction to each as to the scope of work. With the greatly expanded number of technicians working in the field clear communication was even more critical.

UT Testing – Phase III Scope of Work – Full Length Members

As the testing of the member ends was completed, there was a desire to make use of the time prior to opening the bridge to also test a portion of the tension and reversal members for their full length. A testing plan was devised based on assumed production rates of two full-length scans per person, per day, to screen as many members as possible prior to the bridge opening date. The members selected met two criteria: (a) they are in tension or reversal and (b) they were identified as "failure critical" in the 2011 *Redundancy Investigation of P0.00 Deck and Arch Trusses* prepared by Weidlinger Associates. The Weidlinger report labeled some members as failure critical that were in compression. Since these are not sensitive to potential fracture these were not tested. Some tension members were not tested full length because they were not identified as critical in the Weidlinger report.

NDT Findings

There several important findings most notably that no additional plug welds were found at any location along the bridge. Several locations were identified for further investigation by the UT Technicians. The finding and disposition of each is presented below.

Figure 8 Indications at U7-U9 Marked for Further Examination

PA 3-span, north truss, U7-U9 - A finding at U7-U9 on the North truss of the PA 3-span deck truss was noted on February 4th, early in the Phase I work. Testing with a 70-degree shear wave on the same day was inconclusive. Further testing with a 60-degree shear wave was completed on February 6th and confirmed the presence of indications on the inner and outer flanges both above and below the web. These ranged from "recordable", meaning these must simply be noted, to "rejectable" if these were present in a weld. There is no UT rating system in AWS D1.5 for base metal, it relates only to acceptance of welds, so the significance of these findings in a rolled shape is difficult to characterize. On February 8th and then again on February 10th an attempt was made to etch the indications to determine if weld metal or some other surface indication could be detected. These attempts also revealed no apparent welds. Prior to etching on the 8th, a second UT Technician confirmed the location of the indications so the indications were confirmed by two separate technicians using the 90-degree compression wave and a single technician using the shear wave. The markings on the inside of the flange below the web are shown on Figure 7. All indications transferred to the outside flange face are shown on

Figure 7 Coring of Indication at U7-U9

the right. Since each of the indications was roughly in line with the rivet pattern it was suspected these might be filled holes.

After paint removal and etching there was no visual evidence to suggest a plug weld was present yet there were multiple indications. It was decided to core two of the indications. The Class A inside-top and Class B outside-bottom indications were removed using an annular cutter. The core being taken from the outside bottom is shown in Figure 8. The core was taken with a 3-in. outer diameter cutter. The intent of taking the cores was to

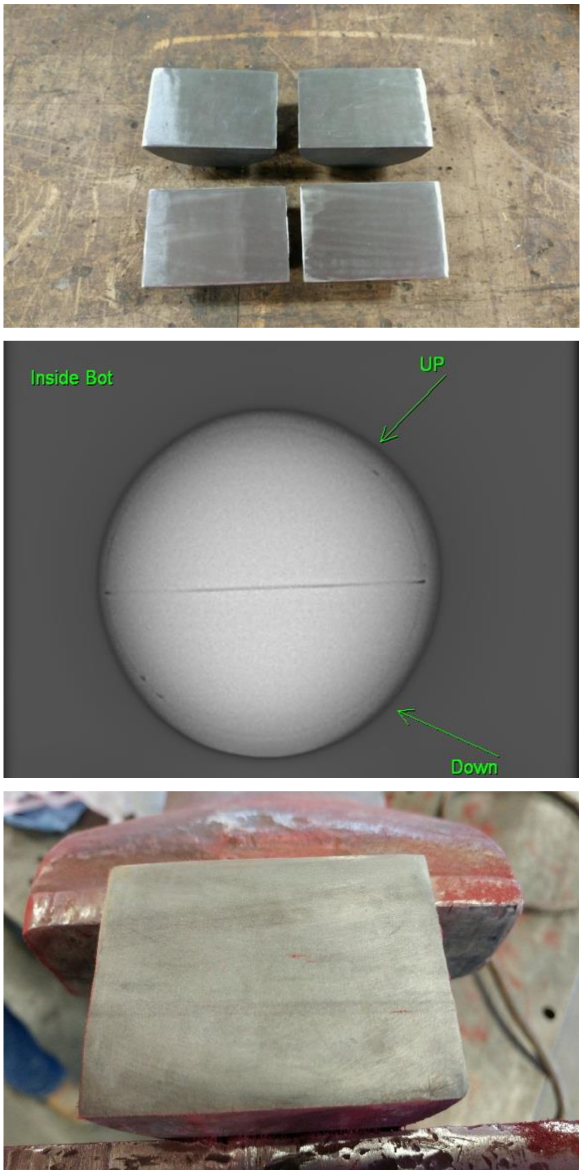


Figure 9 Section, Radiograph and MT Testing of Core U7-U9

examine them destructively by sectioning to determine what was present inside. It had the added benefit of removing the indications.

Once the cores were removed they were sent to the laboratories of Purdue University who polished and sectioned them (see Figure 9). No evidence of a plug weld was found. The material was solid base metal with no evidence of internal defects. With no evidence of what caused the indications the cores were sent to High Steel who subjected them to examination using digital radiography. Those results are shown in Figure 9 as well.

The RT testing also revealed no sign of internal defects. Both cores were tested in the through-thickness direction as they were removed from the flange as well as 90 degrees, i.e. along the axis of the member, as seen in the sectional view. In both cases the test was unable to locate any defects in either core. At this point there was no evidence as to what was found by multiple technicians. Even though these two cores were apparently clean of any defects that could be detected by RT, other indications remained in the member. It was decided in lieu of trying to further characterizes these remaining findings that a cover plate retrofit of this area would be provided.

The cores were returned to Purdue University. A decision was made to not further section them since what caused the UT reflection was of such a small size that it could not be detected using the RT testing. Testing with magnetic particle (MT) testing revealed very small internal discontinuities on the sectioned face (bottom photo of Figure 9). It is hypothesized that these, or similar, are responsible for the UT test result. These are of no consequence and illustrate the judgment that must go along with UT testing when assessing the significance of a finding.

PA 4-span, north truss, L6-L8 - A finding consistent with a plug weld (a large defect) was detected by UT testing using the 90-degree compression wave technique. These were at the same gage line as where existing rivets were so it was suspected that these might also be plugged holes. After paint removal and acid etching it was apparent that they were in fact filled holes but not with weld metal (see Figure 10). The suspicion was that these were rivets placed in a mis-drilled hole and the heads were chipped off and ground. It was decided to core these anyway even though they were obviously not plug welds to see what had been done at these locations. For the mechanically plugged holes shown, it was determined once a 3-in. core was taken and sectioned that the two holes were apparently mis-drilled holes that were tapped, filled with a bolt, with the head and stick through ground flush and then painted. These are of no concern with respect to fracture or fatigue since they are mechanical in nature.

Figure 10 Mechanically Filled Holes at L6-L8

Further examination of the L6-L8 connection also revealed additional mis-drilled holes that had additional holes drilled next to them and then were subsequently riveted. These were apparent on the inside face the I-shaped chord member but not from the outside face. A sketch of the general location of the mis-drilled holes was transferred using a marker to the outside face of the member. The two offset holes correspond to the inner line of rivets at the fill plates. Again, with no real understanding of what the fill material was at these locations, a core was taken. It was found that these holes were filled with a paste or putty-like substance. It was somewhat magnetic but not as magnetic as the steel itself. Chemical testing of the compound was inconclusive but it is not a weld and this material is of no concern with respect to fracture.

NDT Conclusions

No additional plug welds were found. The NDT technicians were successful at finding other anomalies but each of these turned out to be a mis-drilled hole filled by mechanical means. In conjunction with the materials testing discussed in subsequent pages, the finding of no additional weld-filled-holes was fundamental to providing both Turnpike agencies with the confidence needed to reopen the bridge.

Material Sampling and Testing

Most primary truss members in the approach spans were rolled W14 shapes furnished in a high-strength steel marketed at the time as Man-Ten (shortened form of Manganese-Tensile) steel by the United States Steel Corporation. Many of these members are classified as “heavy” sections, with weight exceeding 210 lbs./ft. Previous research has revealed that the fracture toughness of heavy shapes can be significantly lower than that of “normal” shapes (weight less than 210 lbs./ft.), due to the large thicknesses of the flanges and webs and the existence of a core with coarse grain structure at mid-thickness and within the “k-regions” (flange-to-web junctions).

Given the possible differences in Man-Ten steel material properties between members of different sizes, the truss members were grouped into two main categories for the selection of sampled members: normal sections with weight less than 210 lbs./ft. and flange thickness less than 1.5 in. (Group 1), and heavy sections with weight greater than 210 lbs./ft. and flange thickness greater than 1.5 in (Group 2).

Members to be sampled were selected based on the following criteria. Compression members were chosen in preference to tension members due to their lower in-service stresses and lower impact of removing a core (compression members controlled by global buckling rather than cross-section strength); however, both member types were ultimately sampled for the best distribution of thicknesses and locations throughout the bridge. The same section sizes were used as tension and compression members throughout the structure, and as such this criterion did not affect the overall distribution of sampled member sizes. Members with lower dead load were prioritized over members with higher dead load.

In total, 44 cores were taken from the bridge, with half coming from the PA trusses and half from the NJ trusses. All cores were 4 in. outside diameter, with approximately 3.75 in. inside diameter after extraction. Two coring teams extracted all 44 cores in two 12-hour workdays (February 15 and 16, 2017). All core holes were deburred and cover-

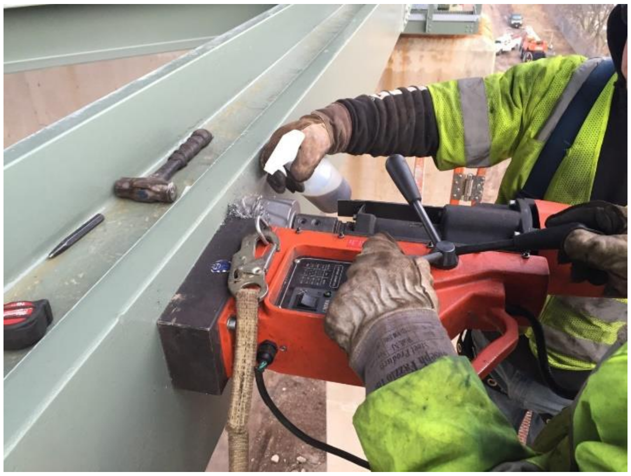


Figure 11 Removal of Material Sampling Cores

plated at the clients request though from a stress standpoint coverplating is not needed. The cover plates were designed by selecting plate sizes which replaced the strength of the section area removed because of the core hole.

Machining of Specimens

All cores were shipped to the NIST-certified Chicago Spectro Service Laboratory in Chicago, IL for machining. Each core was given a designation which indicated the type of specimens to be machined from the core. Depending on the core designation different samples were extracted from each. Cores were sampled as follows:

- a single layer of five CVN specimens along the mid-thickness or quarter-thickness of the core;
- two layers of five CVN specimens, with one layer at mid-thickness and one layer at quarter-thickness;
- a single layer of four CVN specimens and a single tensile specimen at mid-thickness;
- and disk-shaped compact tension (CT) specimens from the core.

The CVN specimens were sent to Purdue University and tested. The tension specimens were tested at Chicago Spectro, and the CT specimens were outsourced by Chicago Spectro and tested by Landow Metallurgical Consulting, LLC.

The tensile test results for the Man-Ten specimens revealed average yield and ultimate strengths of 45.1 ksi and 85.9 ksi, respectively, for specimens from the mid-thickness of plates 1.5 in. thickness or less (Group 1 plates), and average yield and ultimate strengths of 42.2 ksi and 80.0 ksi, respectively, for specimens from the mid-thickness of plates greater than 1.5 in. thickness (Group 2). For Man-Ten tests, the minimum elongation in 1 in. and reduction of area were 28.2% and 63.2%, respectively. Given the elongation and reduction of area values, there are no concerns that the Man-Ten (or similar results from carbon steels tested as well) has insufficient ductility. Additionally, it should be noted that the tensile specimens were taken from the mid-thickness of the flanges where the steel strength and ductility are typically less than the overall average strength and ductility. Therefore, the use of mid-thickness results is more conservative than the current ASTM A6 (2016) approach for new steel production, where specimens are taken from the quarter-thickness to approximate the average of the entire thickness.

Charpy V-Notch Impact Test Results

The results of the Charpy V-Notch (CVN) impact tests for Man-Ten material samples from rolled shapes are presented in as a scatter plot of impact energy vs. test temperature in Figure 12. It is important to note that the CVN impact tests are not a direct measure of fracture toughness. However, fracture toughness can be inferred from CVN test data using known correlations. Nevertheless, the term impact toughness is commonly used when discussing CVN impact energy data.

The data points are grouped by specimens taken from (1) the quarter-thickness of plates less than 1.5 in. thick, (2) the quarter-thickness of plates greater than 1.5 in. thick, (3) the mid-thickness of plates less than 1.5 in. thick, and (4) the mid-thickness of plates greater than 1.5 in. thick. The points are slightly offset from actual test temperatures (10, 40, or 70 °F) for clarity. Also plotted are the Zone 2 CVN impact toughness requirements for fracture-critical members (FCMs) and non-FCMs per the current AASHTO LRFD Specifications. There are no requirements given for ASTM A242 or A440 steel, and as such, the values for ASTM A709 Grade 36 steel are plotted for a simple comparison to modern steel toughness.

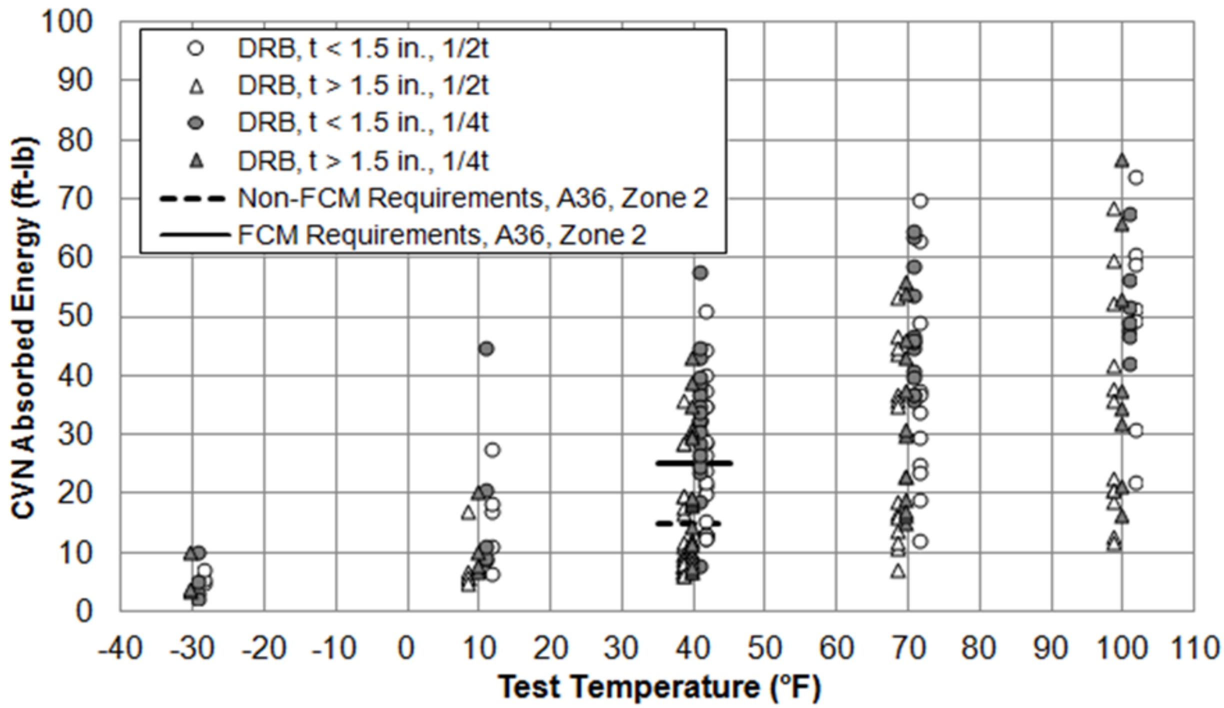


Figure 12 Delaware River Bridge CVN Test Results

It should be noted that the modern impact toughness requirements were plotted only to highlight the differences between modern and historic steels. There were no impact toughness requirements during the era of the bridge's construction. Additionally, modern CVN requirements as per the AASHTO/AWS specifications are based on the requirements for welded construction. Historically, lower CVN requirements were enforced for mechanically-fastened structures (such as the bridge in question) due to their reduced propensity for initial flaws and lack of high residual stresses introduced by welding. Separate CVN requirements for mechanically-fastened structures were removed from the AASHTO specifications in 2009, as most modern structures contain both welded and mechanically-fastened connections and having a single toughness requirement simplifies material ordering and tracking. Further, modern steels regularly exceed the required impact toughness for welded structures and hence the need to differentiate between the two requirements was effectively unnecessary. As such, the DRB steel, intended for a riveted structure, should not be expected to meet the modern-day requirements intended for welded structures.

Past studies have revealed that the fracture toughness of heavy shapes (flange thicknesses > 1.5 in.) can be significantly lower than that of normal shapes (flange thickness < 1.5 in.), due to the large thicknesses of the flanges and the existence of a core in the mid-thickness of these sections resembling a cast steel with coarse grain structure. This observation was confirmed by the test results.

For specimens from plates less than 1.5 in. thickness, there appears to be some dependence of the CVN absorbed energy on the location of the sample through the thickness, with generally higher results for quarter-thickness than mid-thickness samples. For specimens from plates greater than 1.5 in. thickness, this distinction is not present, with the absorbed energies approximately the same regardless of where the sample was taken through the thickness. This observation suggests that the extent of steel worked by rolling processes extends a fixed depth into the thickness rather than a percentage of the thickness. As such, for specimens from plates of 1.5 in. thickness or less, the quarter-thickness samples are within the worked region and the mid-thickness samples are not. For specimens from plates greater than 1.5 in. thick, the quarter-thickness and mid-thickness samples both fall within the non-worked core, where the steel exhibits less toughness.

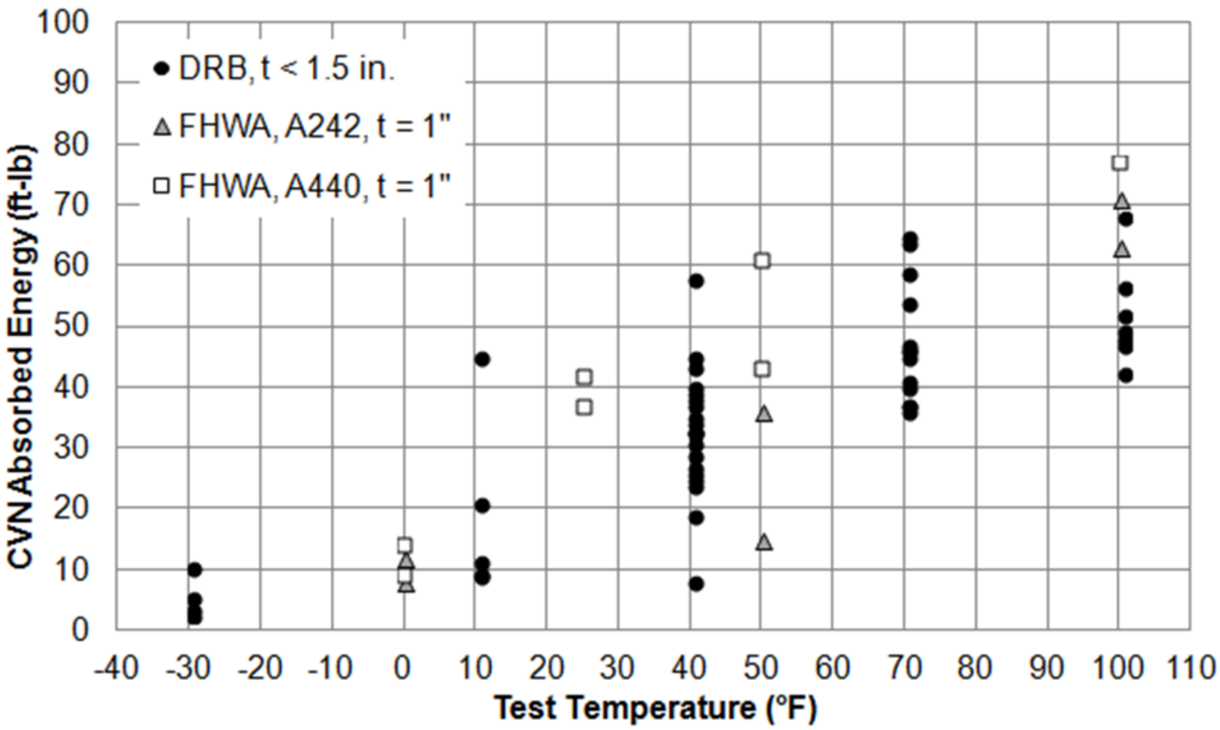


Figure 13 Comparison of Delaware River Bridge Man-Ten CVN Values to Historical Data

To further assess the findings, a comparison was made to toughness data compiled in 1974 by the FHWA in a study on the fracture toughness of bridge steels. As part of that study, CVN impact tests and direct dynamic fracture toughness tests were completed, and data was tabulated for ASTM A242 and ASTM A440 specimens from plates of 1 in. and 2 in. thickness. These sizes and material grades correspond well with the DRB test specimens, and were used for comparison.

Comparisons of CVN impact test data from the DRB and FHWA studies are presented in Figure 13. The data points from the two studies correlated well, which indicated that the DRB steel CVN toughness was typical for ASTM A242 and A440 steel of that era.

Material Testing Conclusions

The material study discussed herein was initiated to (1) determine the chemical and mechanical properties of the steel used in the Delaware River Bridge and any possible variations dependent on section size or plate thickness, and (2) to assess the adequacy of the steel for its original intended use in the bridge.

The Man-Ten steel in the bridge was identified as a modified version of ASTM A242 manganese steel, USS Man-Ten (A242) steel circa 1954, containing more carbon and manganese than the original A242 specification. The bridge steel exhibited the expected strength and ductility characteristics for this steel type with minor variations typical of the results of any material testing program. The carbon steel in the bridge was identified as either ASTM A7 or A373 steel, and exhibited the expected strength and ductility characteristics with similar minor variations.

The lowest fracture toughness was exhibited by mid-thickness specimens in heavy rolled Man-Ten sections (Group 2). This was expected given previous research that has demonstrated reduced impact toughness of (1) plates tested at mid-thickness and (2) thicker plates with cast steel-like grain structure at the core.

Qualitative comparison to modern toughness requirements indicated that the steel used in the DRB possessed lower toughness than modern steels. It should be noted that there were no CVN impact toughness requirements during the era of the bridge's fabrication. The modern requirements also represent the historical requirements for welded structures, whose higher propensity for flaws and large residual stresses makes them more susceptible to fractures than mechanically-fastened structures such as the DRB. Accordingly, it should not be expected that the materials used in the DRB would meet modern requirements.

The lower bound fracture toughness of the Man-Ten and carbon steels at the lowest anticipated service temperature (LAST) were also calculated, and full lower bound fracture toughness curves were presented that may be used in future evaluation of the structure. Comparison of the DRB results to a FHWA study on the toughness of similar steels also confirmed that the DRB steel is typical of the era.

Based on the test results obtained in this investigation, the steel of the Delaware River Bridge is consistent with similar steels of the era in terms of strength and toughness. While not exhibiting the CVN impact toughness of modern steels used in welded construction, the material properties are sufficient for use in the bridge as originally intended, i.e. as a mechanically-fastened structure with no welds.

Conclusions

In what is believed to be the largest ever field NDT / material testing program for a steel bridge in the United States, a large team of professionals from many firms was assembled to provide the field, office, and laboratory expertise to provide critical information to the PA and NJ Turnpike. In parallel with this NDT work, many other critical activities also occurred. These include erection of jacking towers, design of a permanent chord splice, field load testing of the repaired bridge, and a detailed inspection of the entire truss. In what is in hindsight, a short seven weeks, a critical infrastructure link was repaired and reopened for service. The information gained through this investigation was critical to providing the peace of mind to the owners and allowed this bridge to be opened to traffic once again.