NATIONAL ACADEMIES *Sciences Engineering Medicine*

TRB TRANSPORTATION RESEARCH BOARD

TRB Webinar: T-1 Steel, I-40 Bridge, and the Way Forward

November 10, 2022

12:00 – 1:30 PM

PDH Certification Information

1.5 Professional Development Hours (PDH) – see follow-up email

You must attend the entire webinar.

Questions? Contact Andie Pitchford at TRBwebinar@nas.edu

The Transportation Research Board has met the standards and requirements of the Registered Continuing Education Providers Program. Credit earned on completion of this program will be reported to RCEP. A certificate of completion will be issued to participants that have registered and attended the entire session. As such, it does not include content that may be deemed or construed to be an approval or endorsement by RCEP.

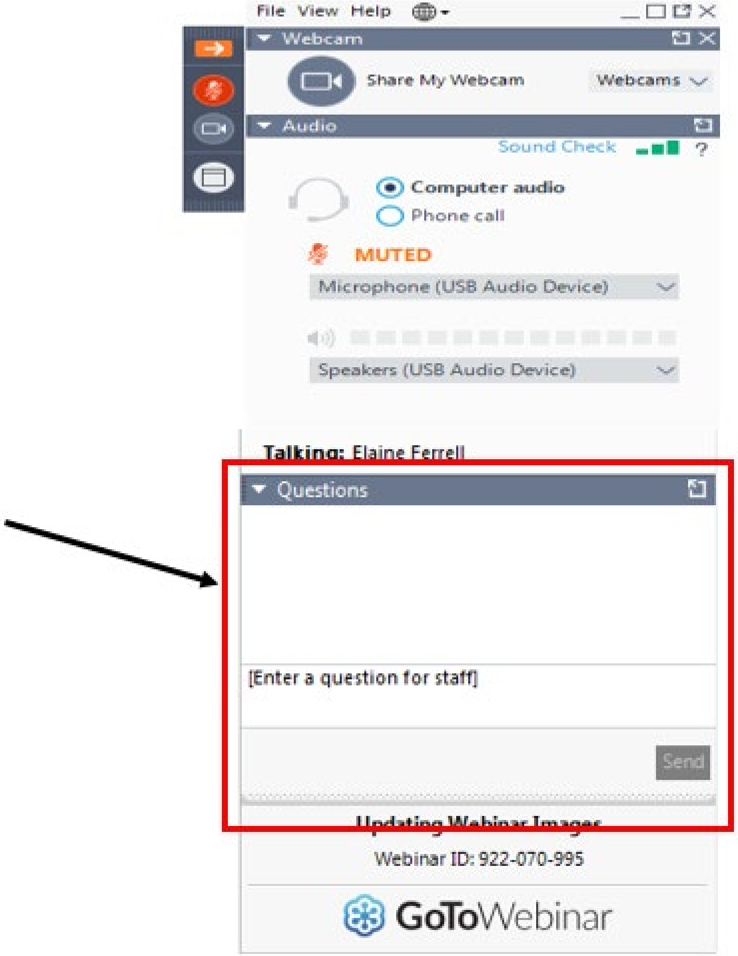
RCEP
REGISTERED CONTINUING EDUCATION PROGRAM

Learning Objectives

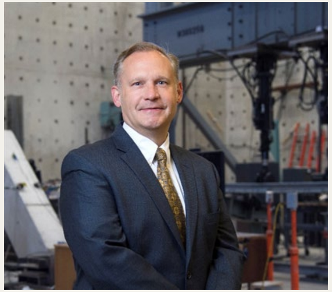
- Identify T-1 steel and the importance of the FHWA directive
- Summarize lessons learned from a response to a partial fracture on a high traffic bridge
- Explain the testing and repair of the I-40 bridge and takeaways from the event

Questions and Answers

- Please type your questions into your webinar control panel
- We will read your questions out loud, and answer as many as time allows



Today’s presenters



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T-1 Steel, I-40 and the Way Forward

Rob Connor
Jason Stith
Curtis Schroeder
Derek Soden

Seminar Agenda

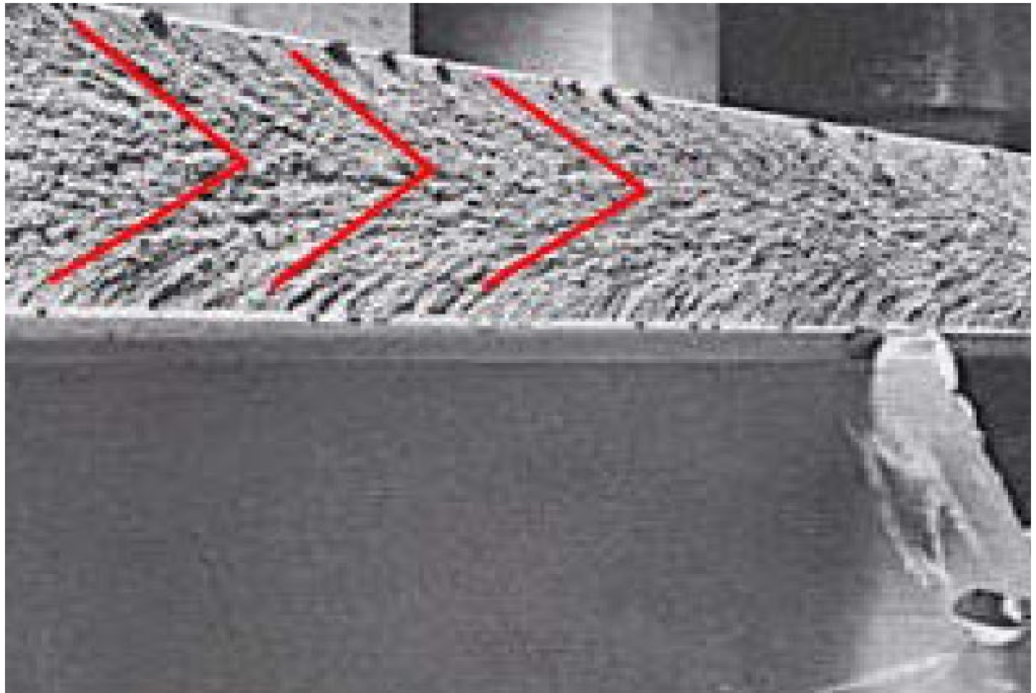
- Background (What is T1 steel, Sherman Minton, I-40)
  - Jason Stith, Michael Baker
- FHWA Directive
  - Derek Soden, FHWA
- Testing and Lessons Learned
  - Curtis Schroeder, WJE
- NDT and Testing
  - Dr. Rob Connor, Purdue
- Q & A

T-1 Steel

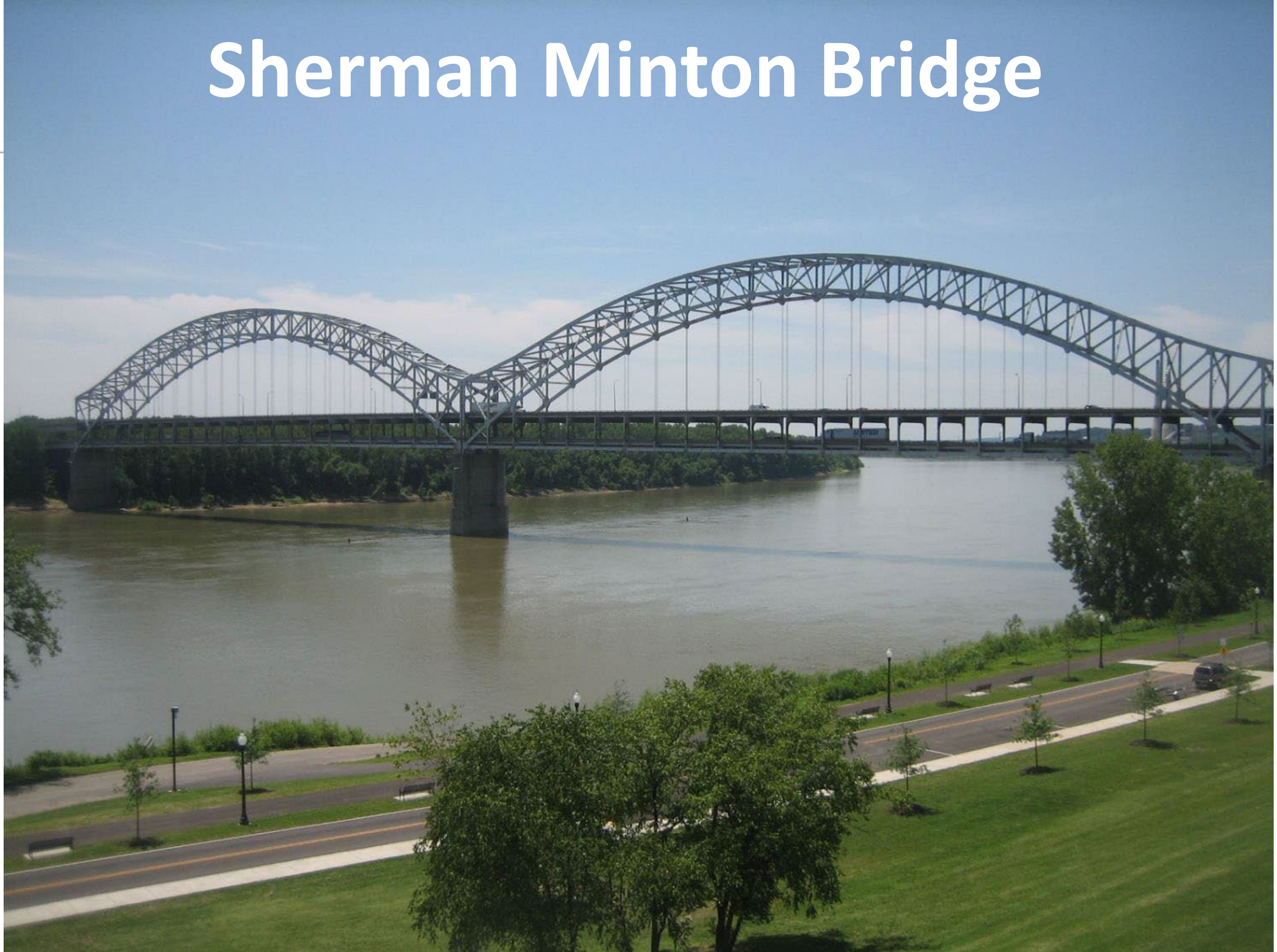
- Quenched and tempered high strength steel first used in bridge in the early 1960s
- T-1 was a proprietary brand by United States Steel Corp
- ASTM A514 and A517 were standardized
- Typically
  - 100 to 110 ksi yield strength
  - 115 to 135 ksi

T-1 Steel

- T-1 Steel has no inherent challenges or issues
- All the issues with T-1 Steel have been with welds
- There was no chemical control, pre-heating, or testing
- I-80 Bryte Bend Bridge near Sacramento, CA: Tub Girder
  - QT steel
  - June 13, 1970
  - Top flange weld of box girder fractured during deck placement
- 1978 AWS Fracture Control Plan



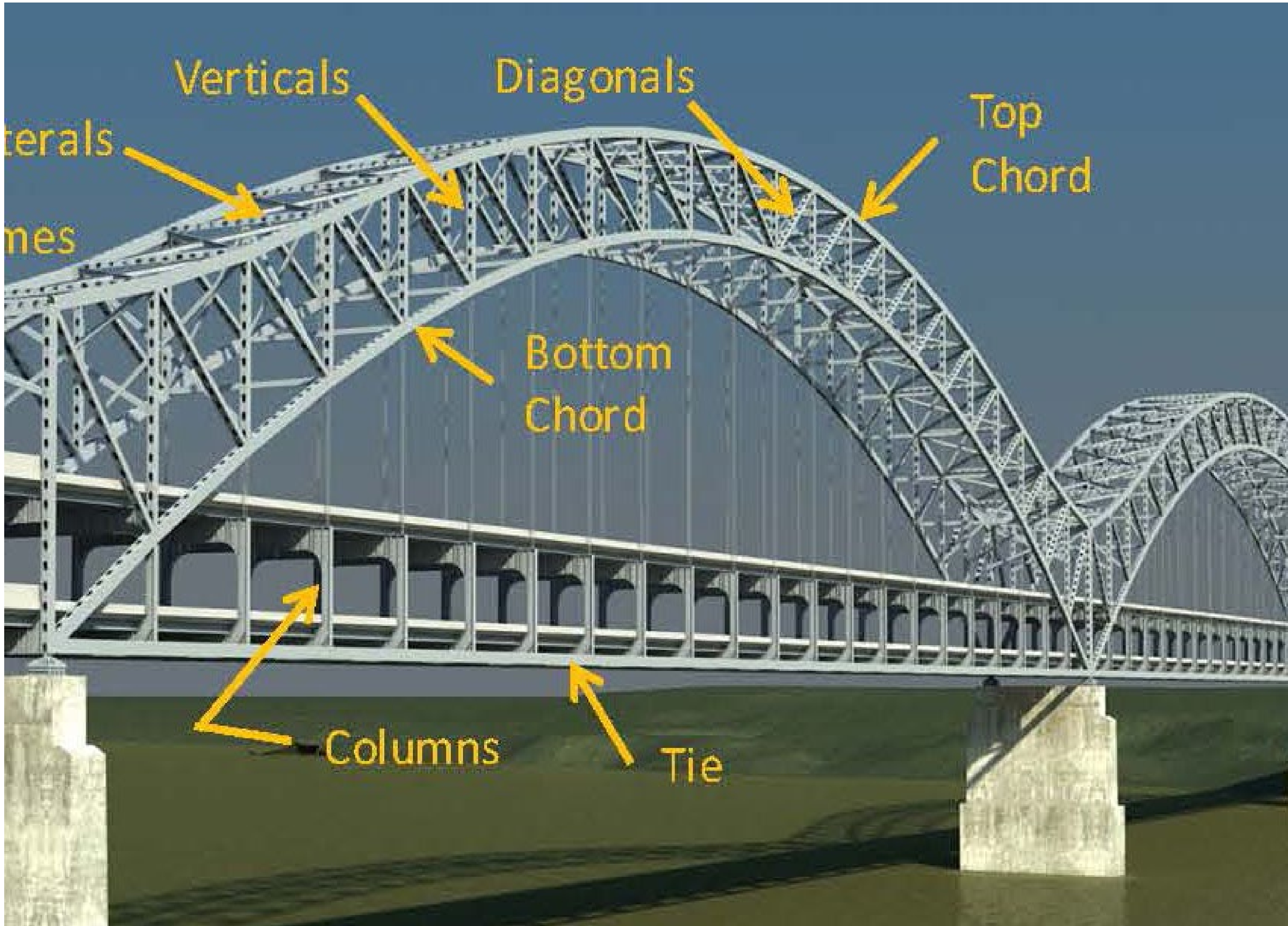
Sherman Minton Bridge



INDOT

Sherman Minton Bridge

- Structure Information
  - Tied Arch Built Early 1960's
  - 2 – 800 foot spans
  - Double-deck structure
  - Navigable waterway underneath

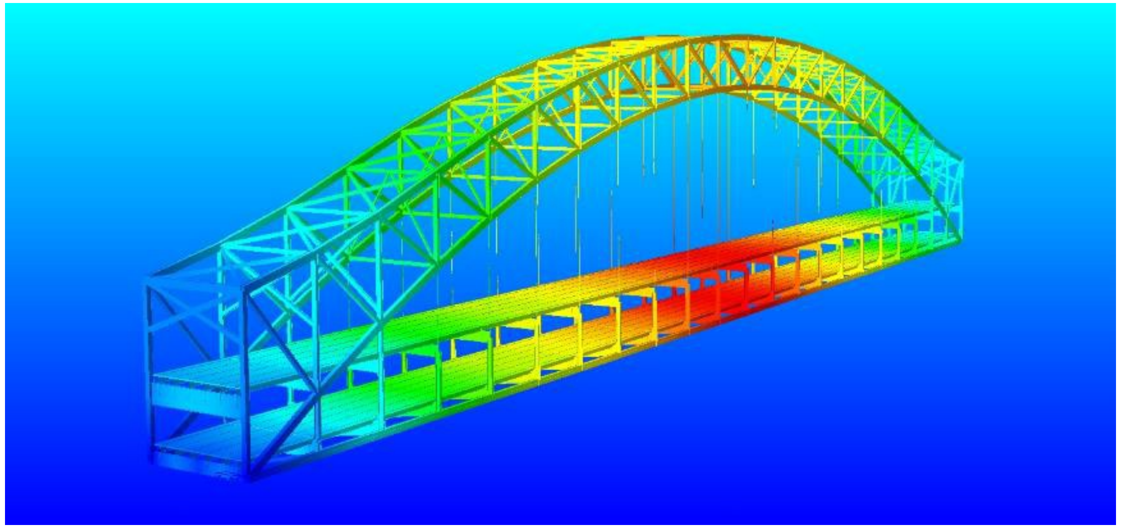


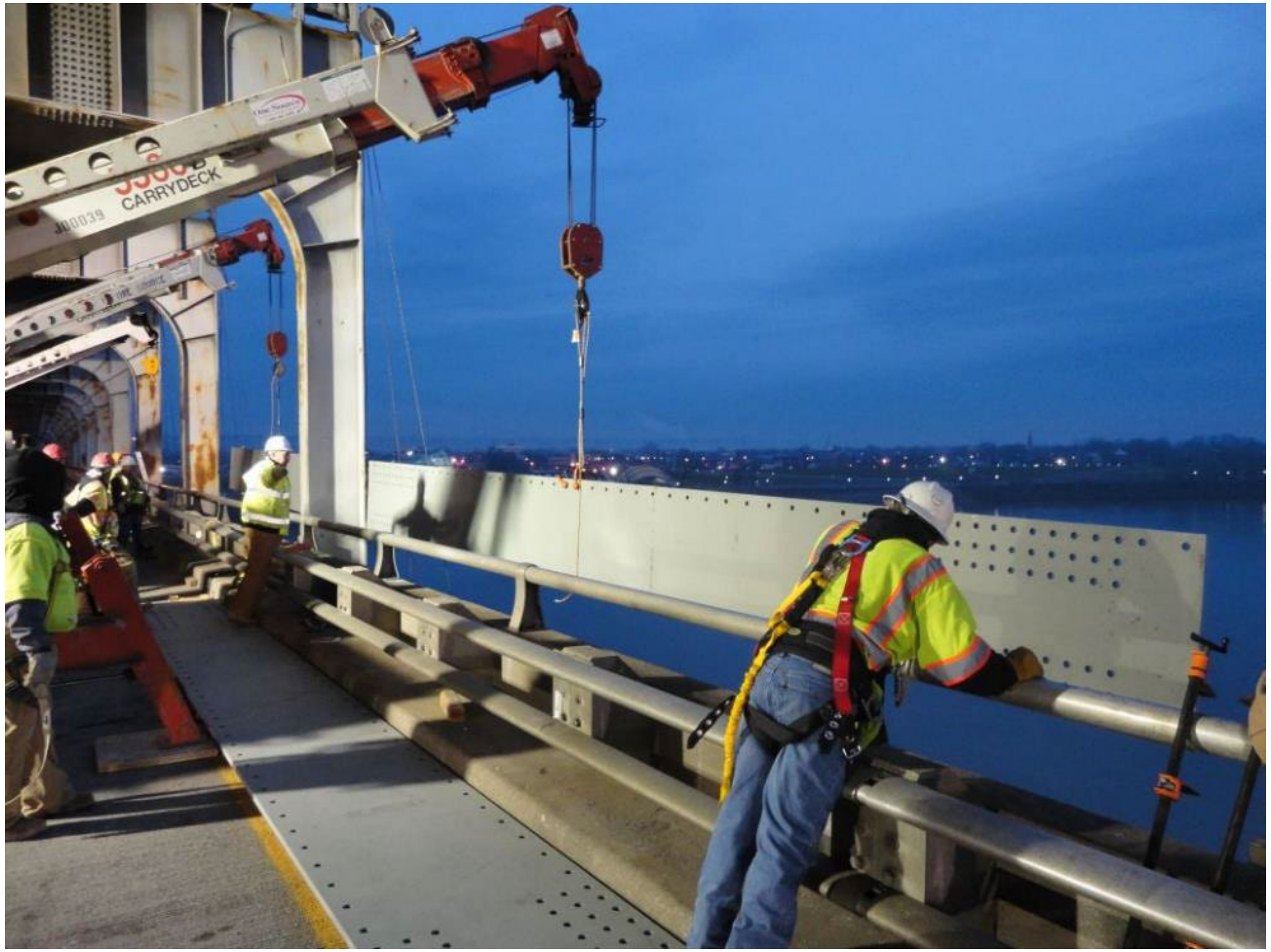
Inspection & Testing - 2011

- Hands-on visual inspection of weld metal on the tie girder
  - What can we find?
- Nondestructive testing (MT, UT, RT, X-Ray)
  - What can we find that we can't even see?
- Sampling of bridge materials
  - How strong, how tough, how big of a crack is "too big"?
- The most comprehensive visual, NDT & physical testing program ever conducted on this bridge

Major Project Quantities

- 2.4 M Pounds of Structural Steel
- Just over 1,000 Steel Plates
- 55,000 Field Drilled Holes
- 73,000 High Strength Bolts





FHWA Technical Advisory 5140.32

- September 12, 2011
  - **Recommend** NDT

3. It is also recommended that on fracture critical bridges fabricated using T-1 steel prior to the adoption of the Fracture Control Plan of the AASHTO/AWS D1.5-88 Bridge Welding Code, where cracks due to a lack of hydrogen control during welding have previously been found, that the soundness of all butt welds in those tension components be verified through visual and non-destructive testing unless this verification has been previously conducted.

U.S. Department of Transportation
Federal Highway Administration

Technical Advisory

Inspection of Fracture Critical Bridges Fabricated from AASHTO M270 Grade 100 (ASTM A514/A517) Steel

Questions and Answers
September 12, 2011
Technical Advisory 5140.32

PURPOSE

The purpose of this Technical Advisory is to provide recommendations regarding the in-service inspection of, and the treatment of critical findings identified on, fracture critical bridges fabricated from AASHTO M270 Grade 100 (ASTM A514/A517) steel, more commonly known as "T-1" steel.

BACKGROUND

1. The I-64 Sherman Minton Bridge is a fracture critical bridge which consists of two 800-foot tied arch truss main spans that carry six lanes across the Ohio River between Louisville, Kentucky and New Albany, Indiana that was constructed between 1960-1961, before the material and fabrication requirements of the AASHTO/AWS Fracture Control Plan for this type of bridge were adopted.
2. As the result of in-service inspection, several cracks were found in the butt welds or their associated heat-affected zones of the tension ties of both spans. It was subsequently determined that the cracking was very likely caused by hydrogen that was introduced into the weld as the result of improper fabrication procedures. T-1 steel is known to be very susceptible to this type of cracking.
3. Earlier this year, retrofit and repair work to address those cracks and additional inspection work to verify the soundness of the remaining butt welds in the tie began.
4. On September 8, 2011, inspectors discovered an additional critical crack in the tension tie that previously could not be seen through visual inspection because of the removal of a connection plate detail as part of the ongoing retrofit process.
5. After study and analysis of this newly found crack, it was determined that an unacceptable level of risk to the traveling public was associated with the continued operation of the bridge. As a result, on September 9, 2011 the bridge was closed.

RECOMMENDATIONS

1. This Technical Advisory strongly recommends that State Departments of Transportation and other bridge owners review the inspection records of their inventory of fracture critical bridges to ensure any components fabricated with T-1 steel have been regularly and appropriately inspected and that any critical

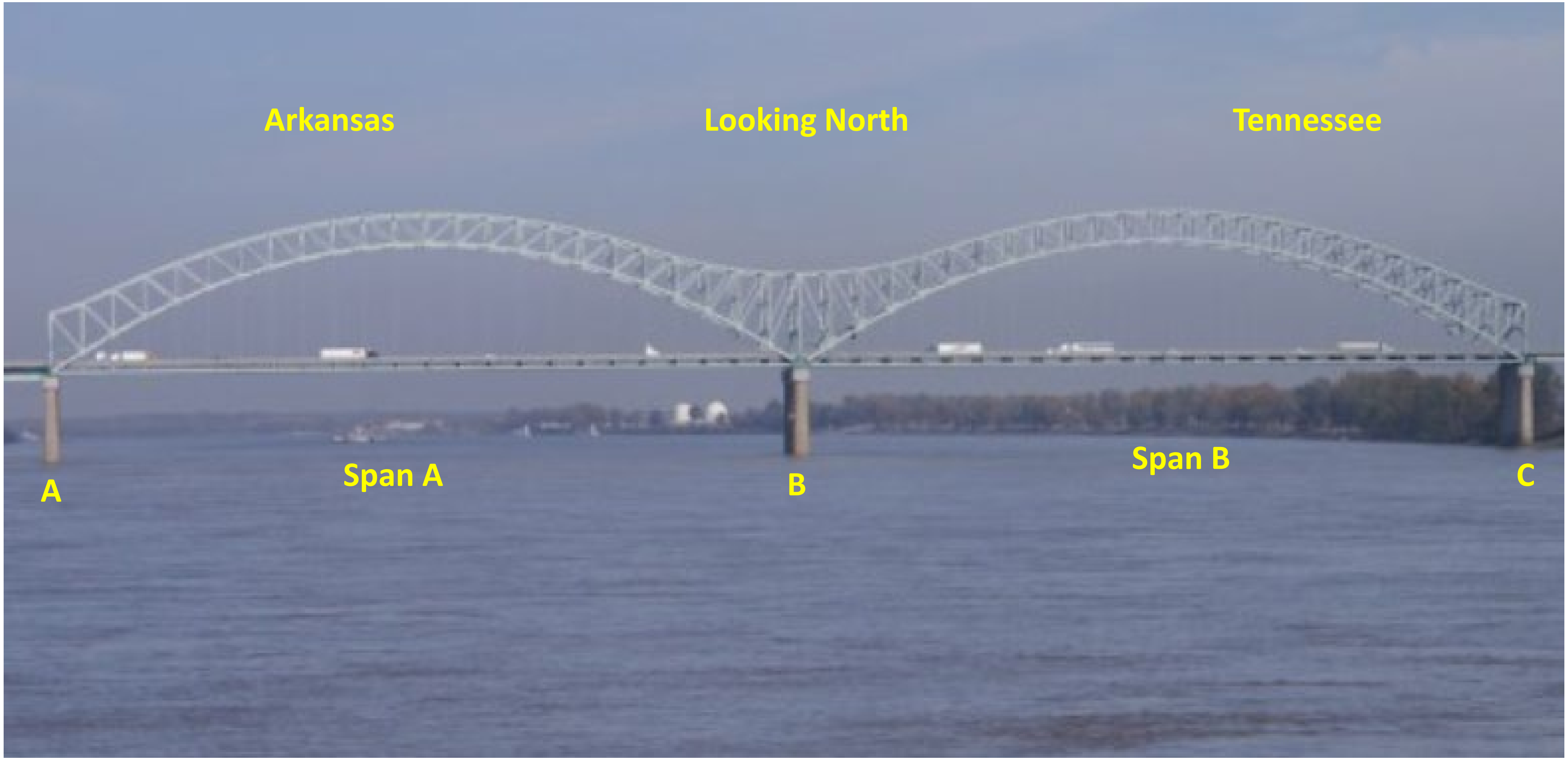
2

findings have been properly identified and addressed. As defined in the National Bridge Inspection Standards, a fracture critical member inspection involves a hands-on inspection that may include visual and other nondestructive evaluation.

2. If deficiencies are found, follow up with those structures placing priority on inspection or remediation of components primarily in tension such as arch ties, hangers or truss members that contain butt welds.

3. It is also recommended that on fracture critical bridges fabricated using T-1 steel prior to the adoption of the Fracture Control Plan of the AASHTO/AWS D1.5-88 Bridge Welding Code, where cracks due to a lack of hydrogen control during welding have previously been found, that the soundness of all butt welds in those tension components be verified through visual and non-destructive testing unless this verification has been previously conducted.

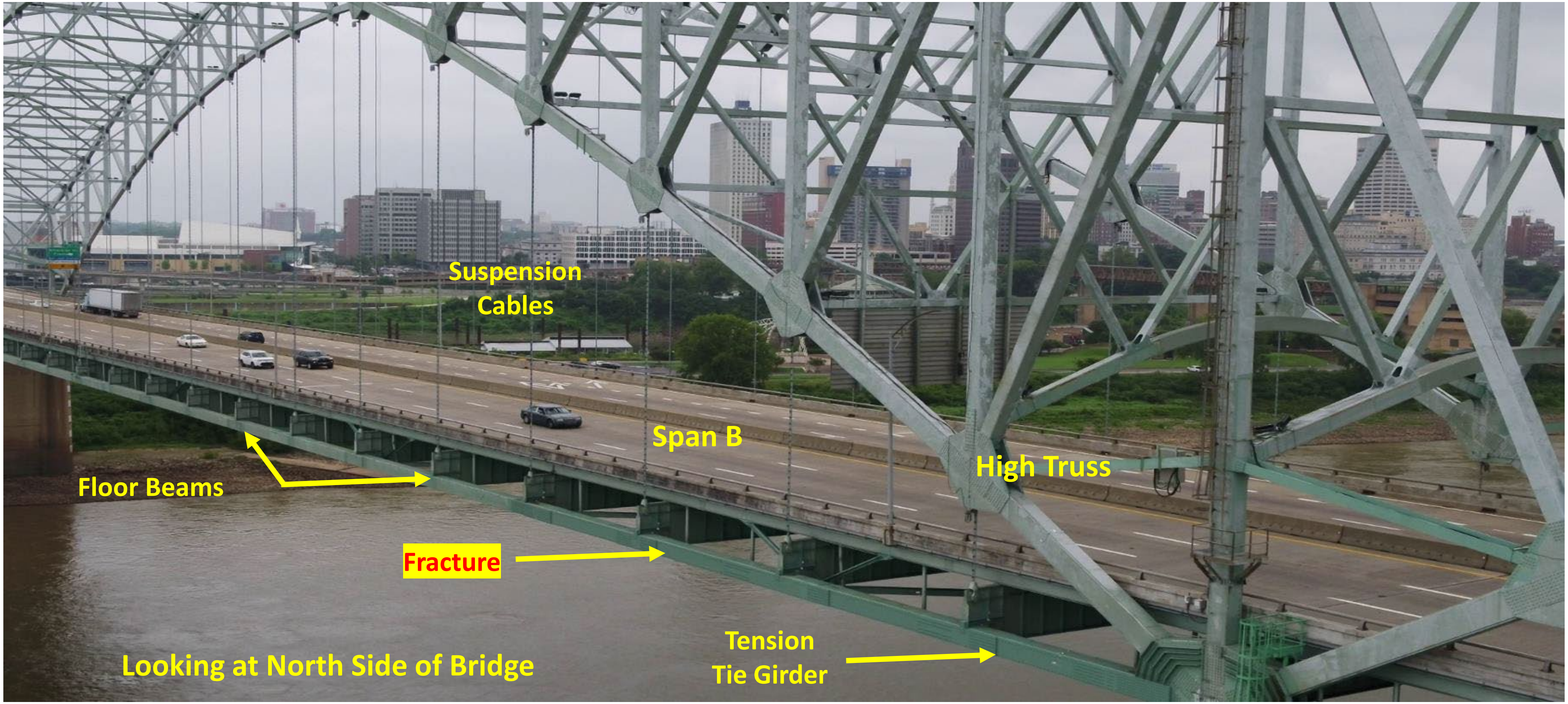
Interstate 40 Hernando de Soto Bridge



Bridge History

- Hernando de Soto
- Constructed 1967-1973
  - Opened August 2, 1973
- Two Span Continuous Tied Arch Bridge
  - 2 – 900ft spans
  - 109ft above the water
  - Designed by Hazlett and Erdall

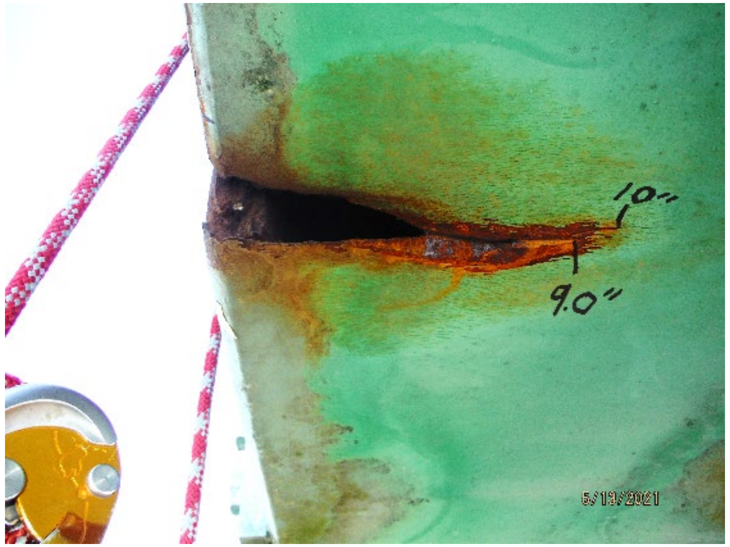
Interstate 40 Hernando de Soto Bridge



Immediate Response

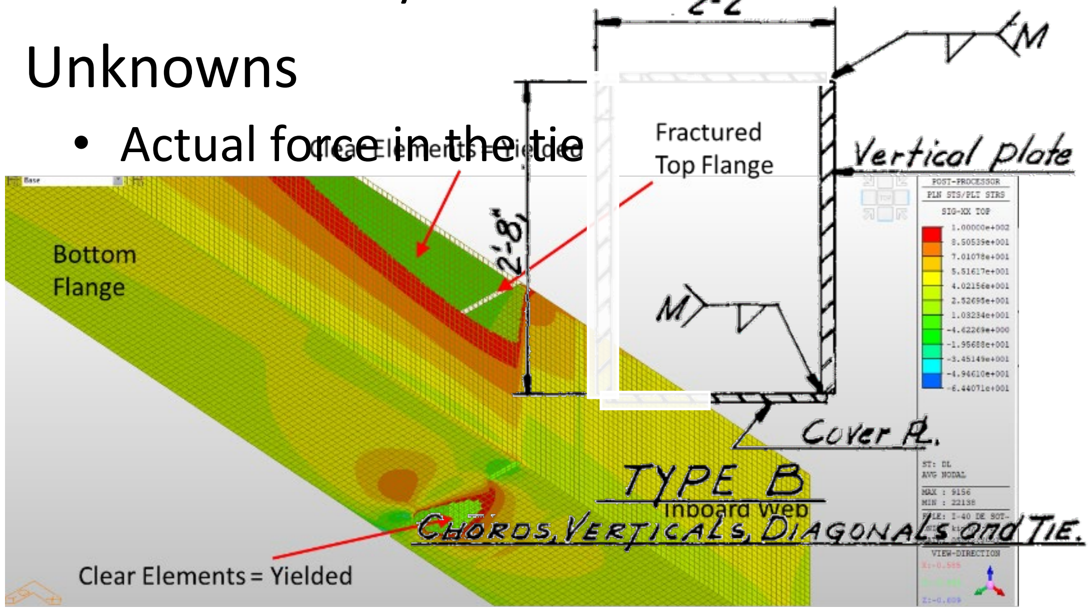
Early Actions in the days immediately following the critical find

Hernando de Soto – UAS Live Feed Inspection



How Bad is it?

- T1 steel = 100 ksi (+)
  - P/A design = 38ksi
- Fracture 113 in2 -> 51.5in2 (**45%!**)
  - P/A after Fracture = 83ksi
- Eccentric Loading
  - Refined Analysis
- Unknowns
  - Actual force in the tie



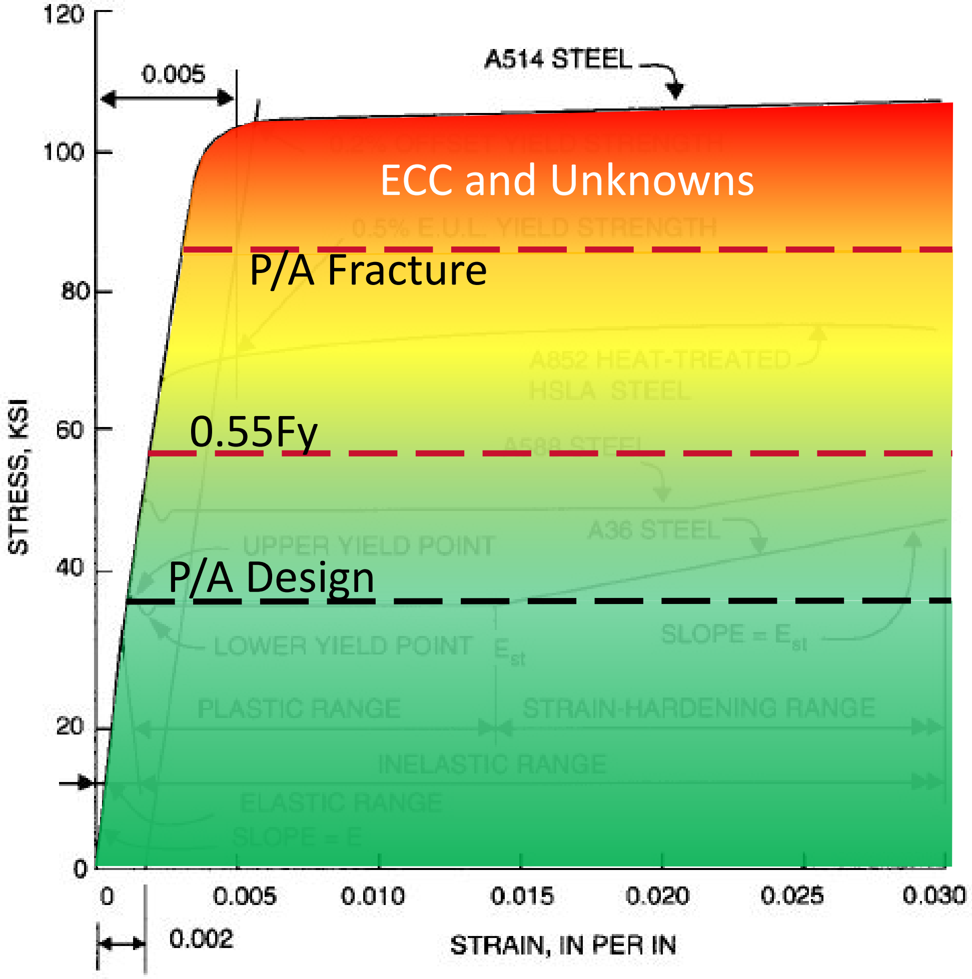


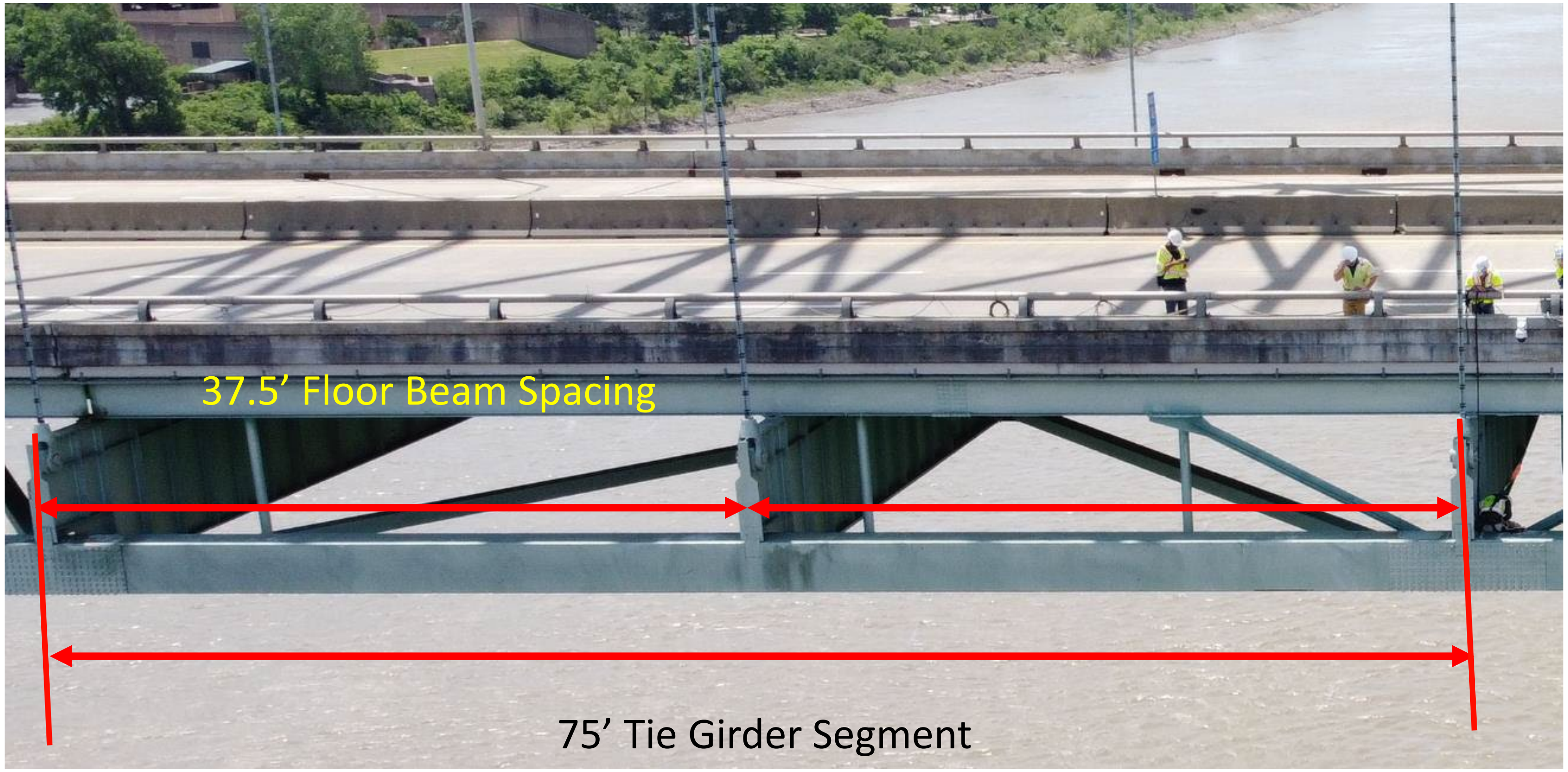
FIGURE 1.4 Partial stress-strain curves for structural steels strained through the plastic region into the strain-hardening range. (*From R. L. Brockenbrough and B. G. Johnston, USS Steel Design Manual, R. L. Brockenbrough & Associates, Inc., Pittsburgh, Pa., with permission.*)

Phase 1 Repairs

Stabilizing the Structure

Phase 1: Stabilize the Structure

- Design Challenges for Phase 1 –
  - 100 ksi plate in tie girder and select high truss members
  - Thin (1.375”) plate sections limited bolting options due to net section limitations
  - Geometry (twist) of current tie girder impacted plating design



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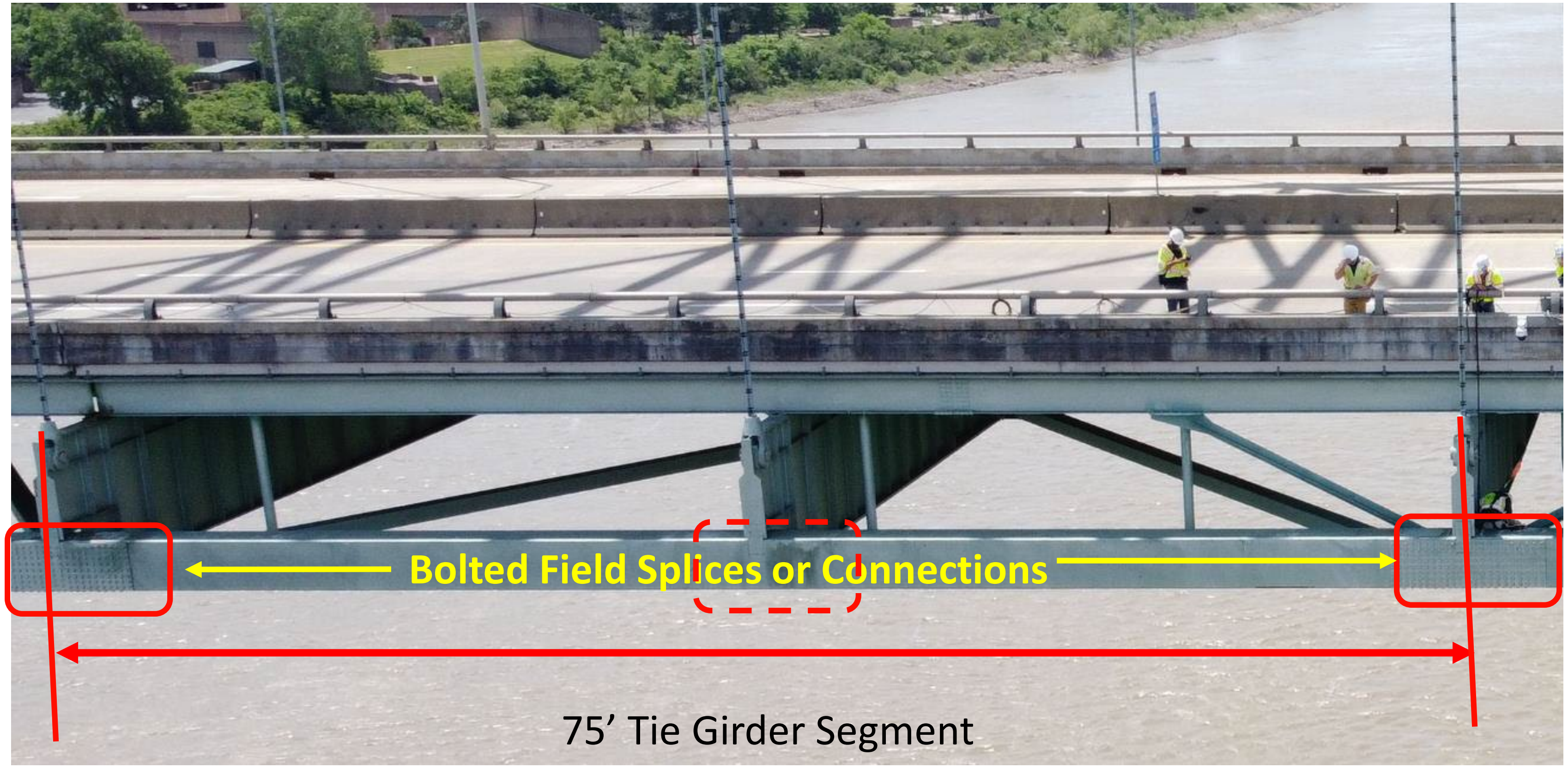

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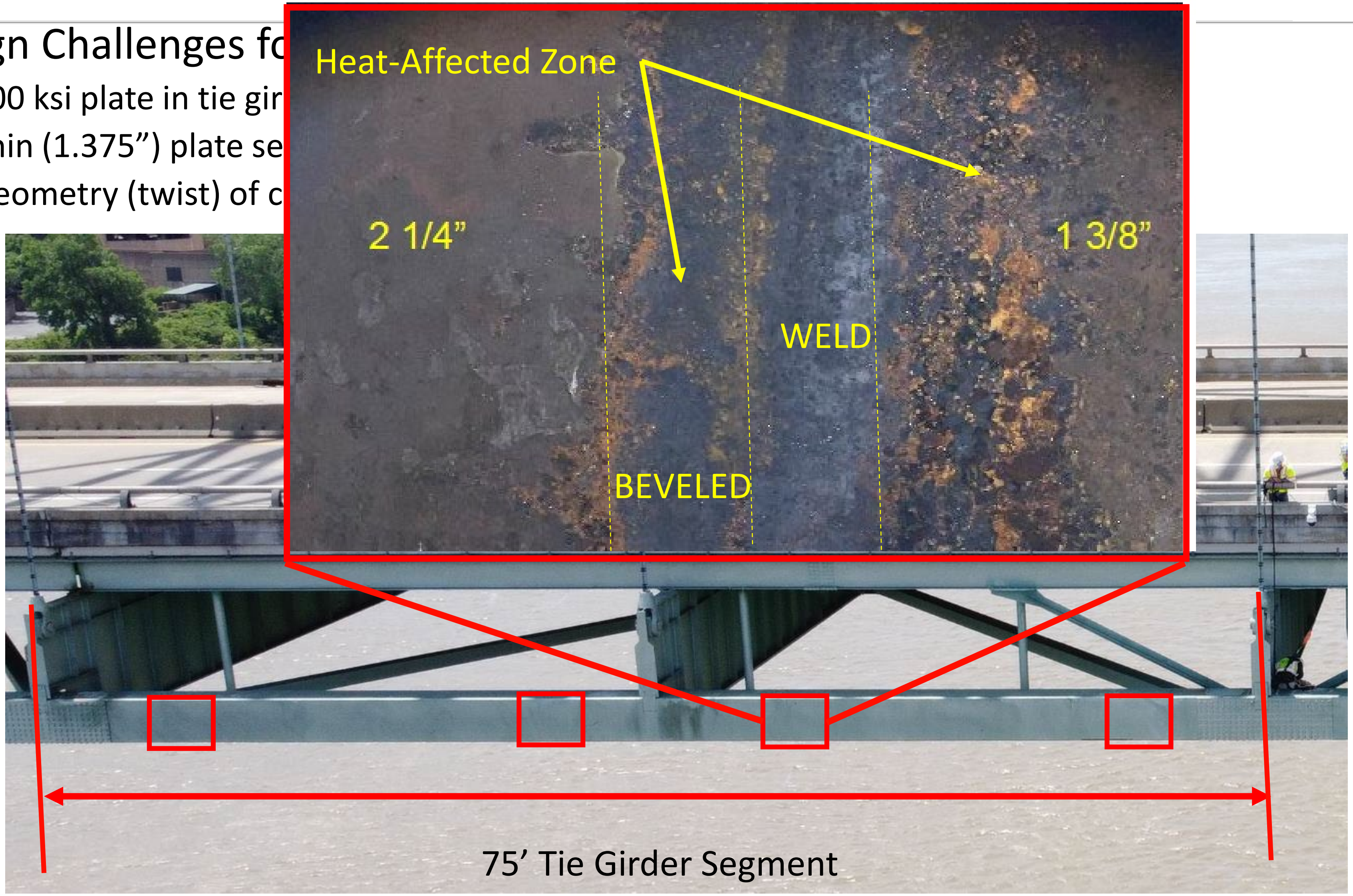

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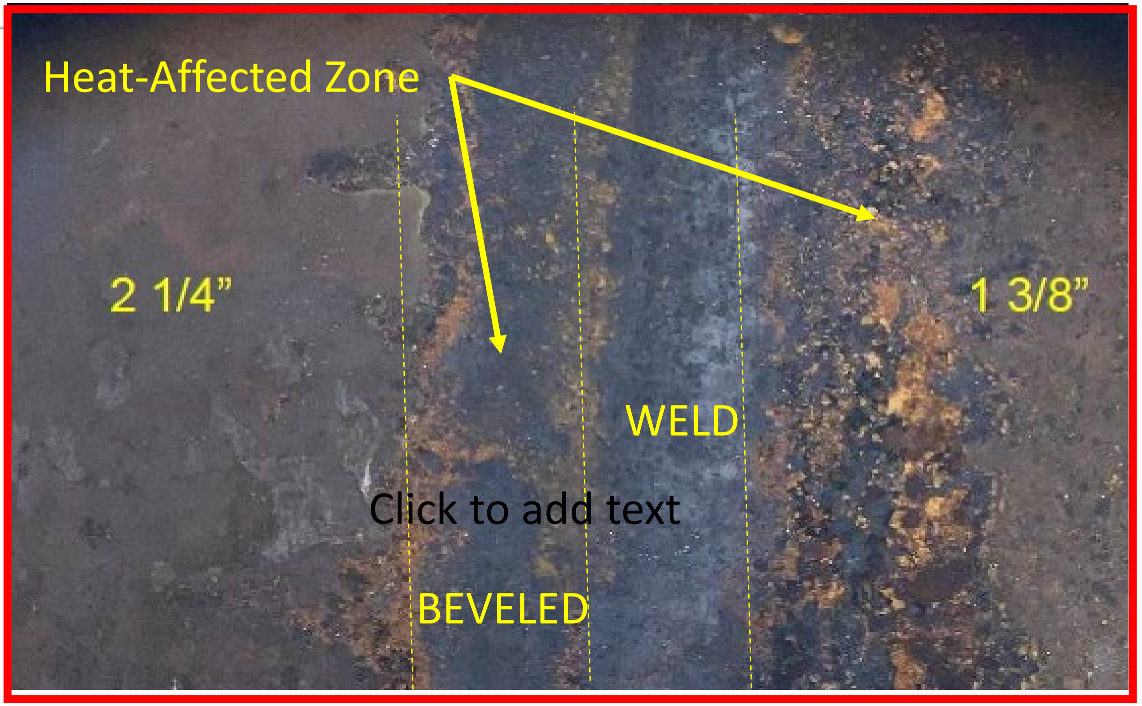
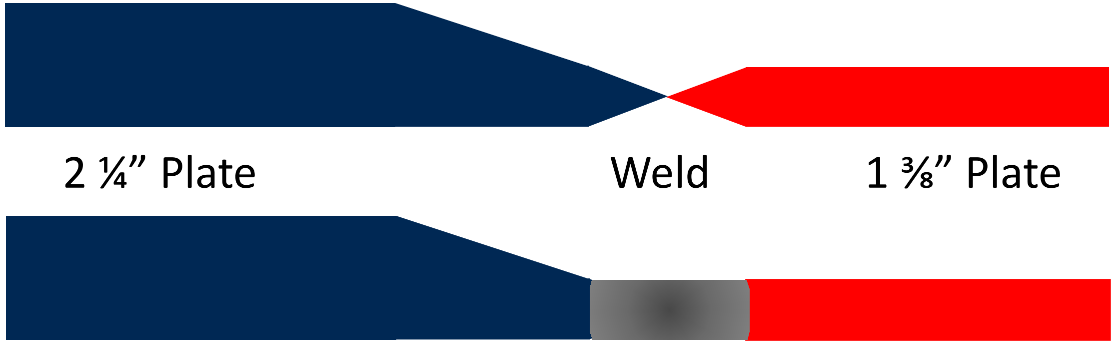


Inside Surface of Tie Girder Box Section

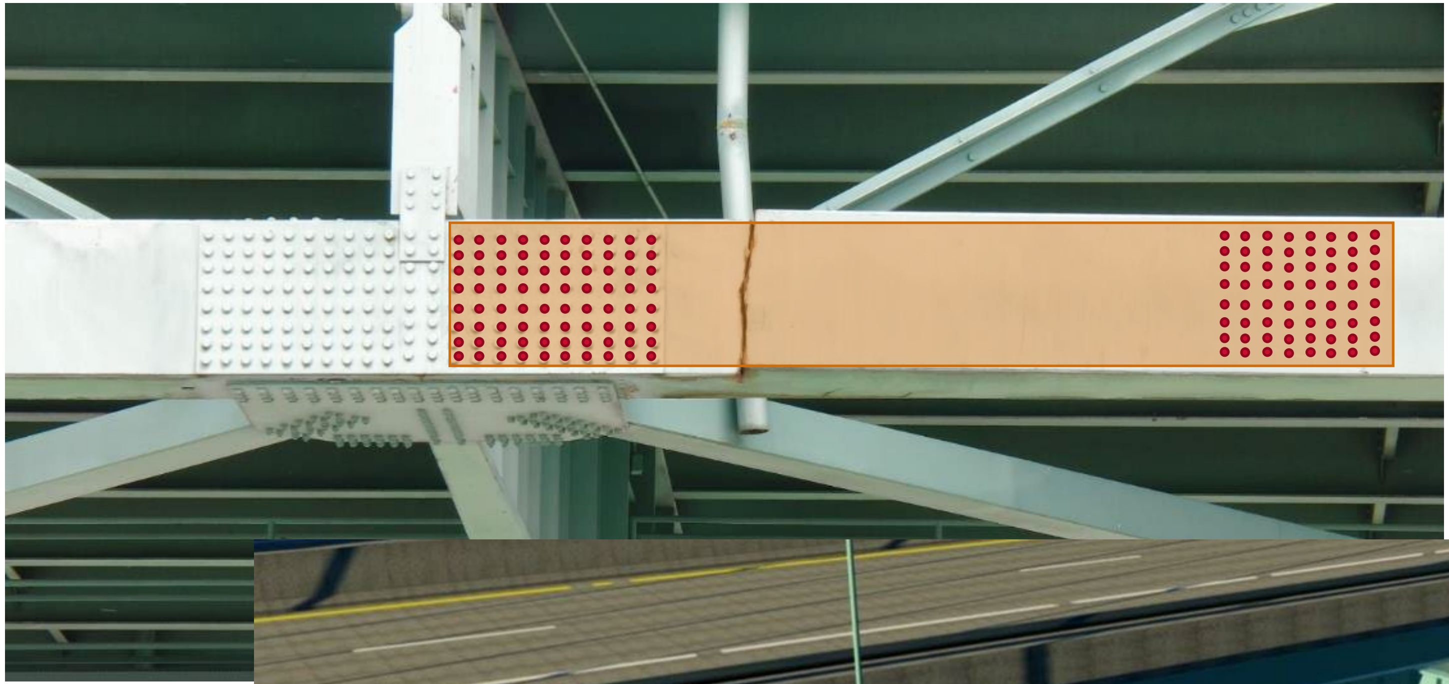


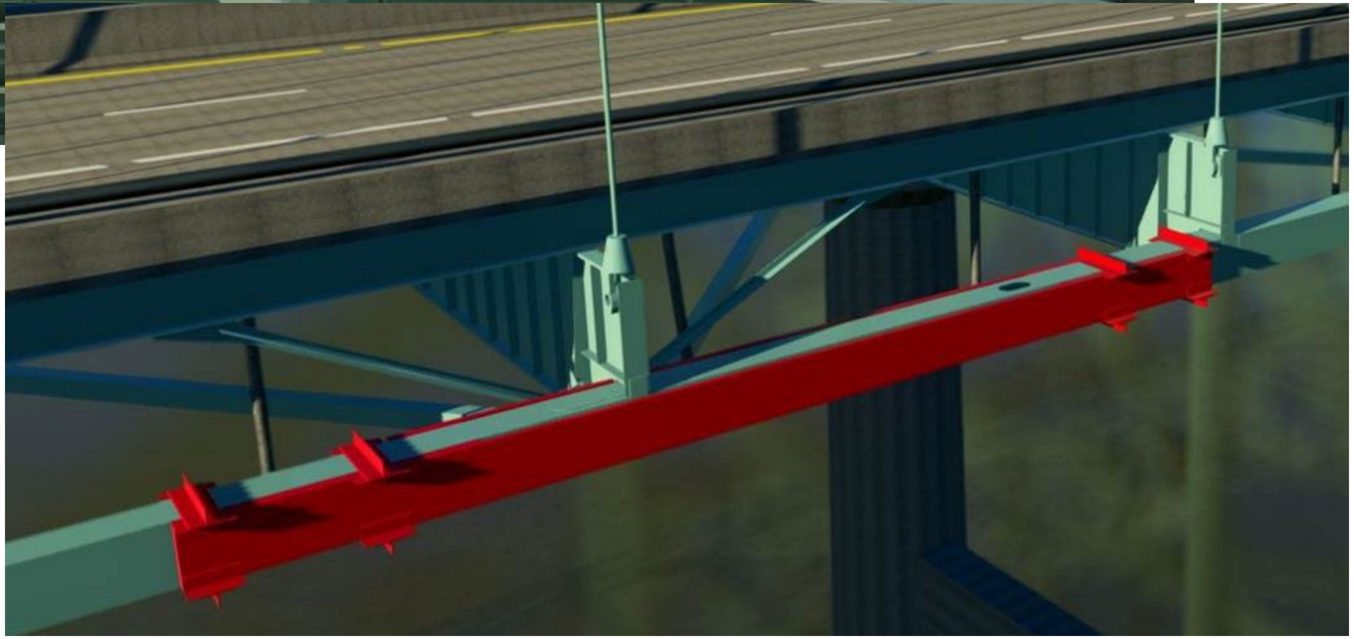
Outside Surface of Tie Girder Box Section

Phase 1: Stabilize the Structure Day 6

Design Collaboration

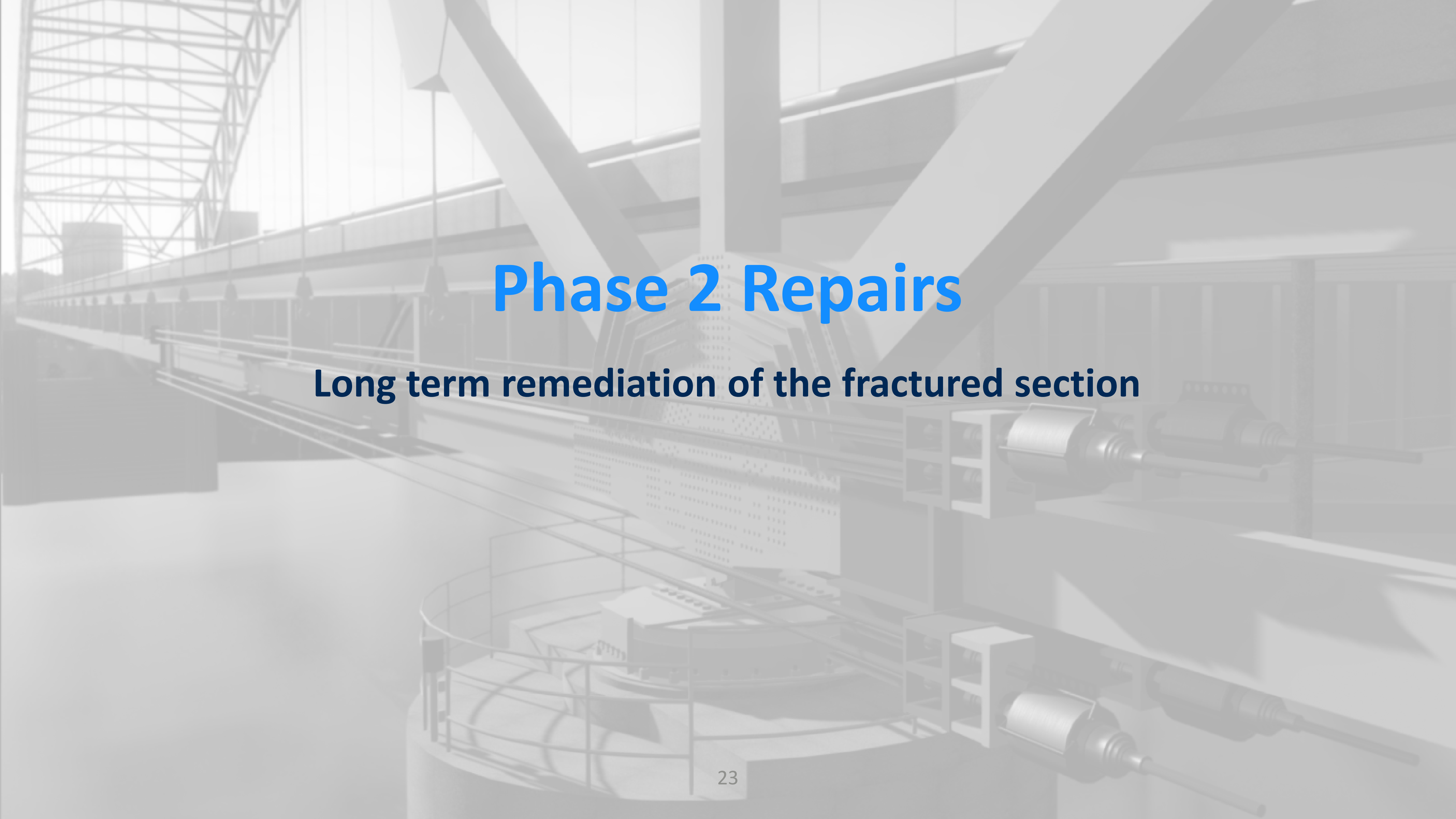
- CMGC Contractor Selected
- **Reduce Risk…**
  - Add capacity
  - low impact operations (drilling not bolt removal)
  - No attempt to straighten the tie





Phase 1: Plate Installation

- Kiewit Infrastructure began installing the plates Saturday, May 22nd. **Day 11**
- Phase 1 plate installation was completed on Tuesday, May 25th. **Day 14**
- Completing Phase 1 allowed for starting the Phase 3 inspection work

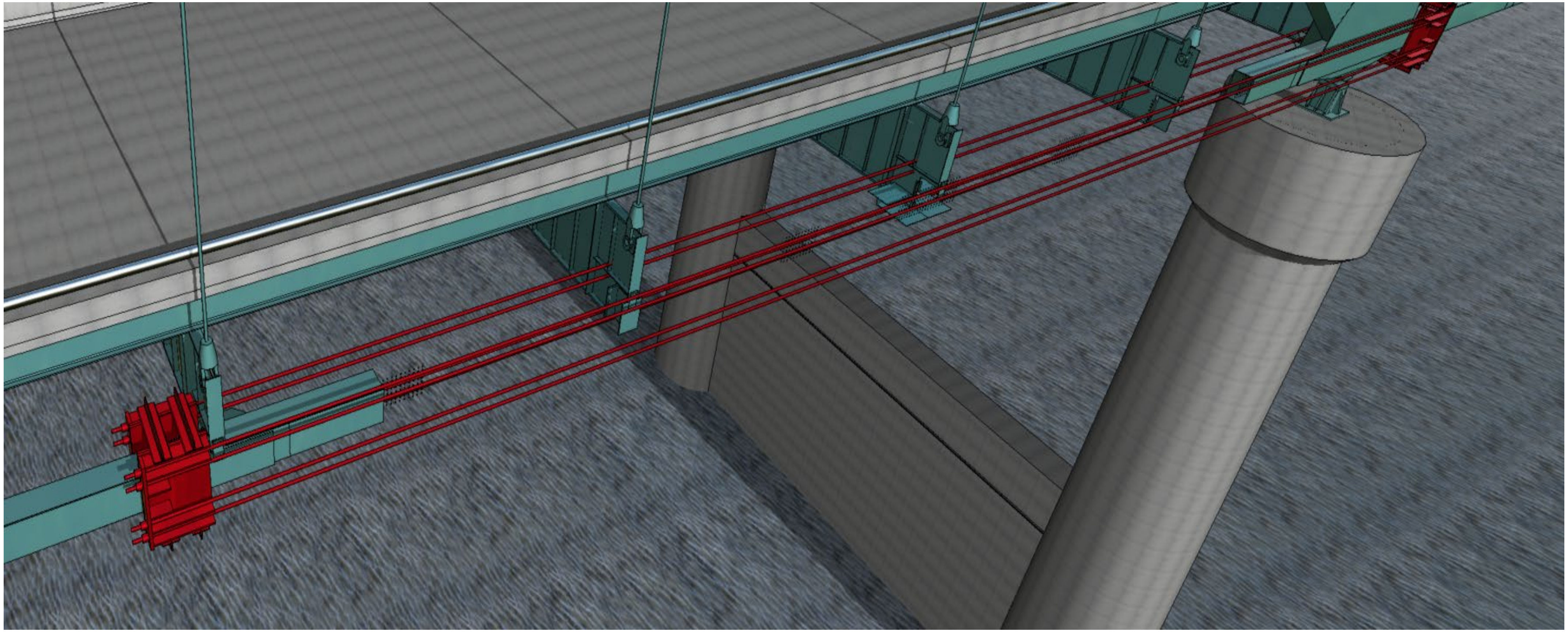


Phase 2 Repairs

Long term remediation of the fractured section

Tie Girder Complete Replacement

- Initial direction: completely remove the old/fractured tie
- End Result: Cut out Fracture and Plate back to connections



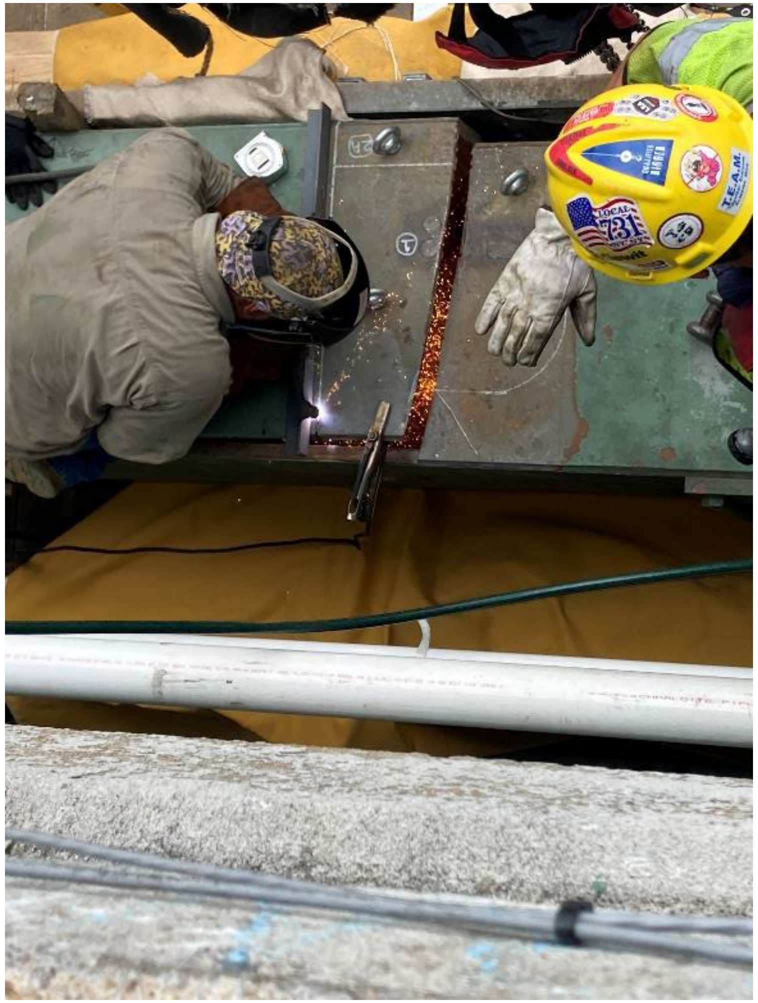
Stressing

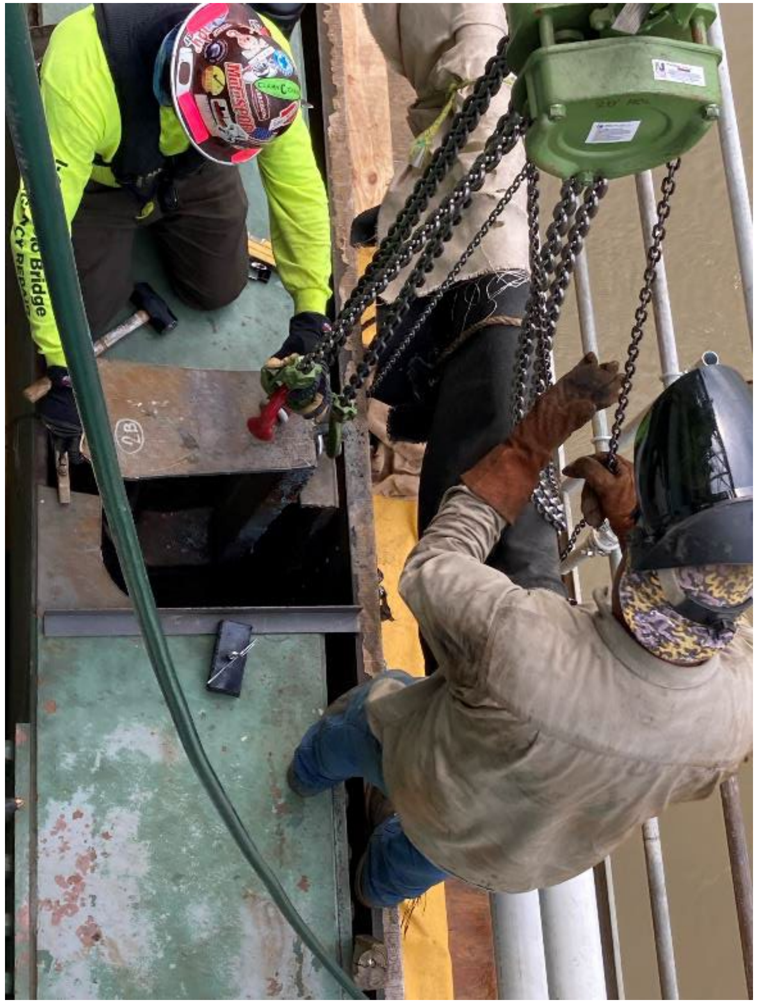
Day 42

PT Stressing began Sunday, June 20th and was completed June 22nd

Removing the Fracture

Day 43





Phase 2 Completion

- Final Painting
- Phase 2 Complete

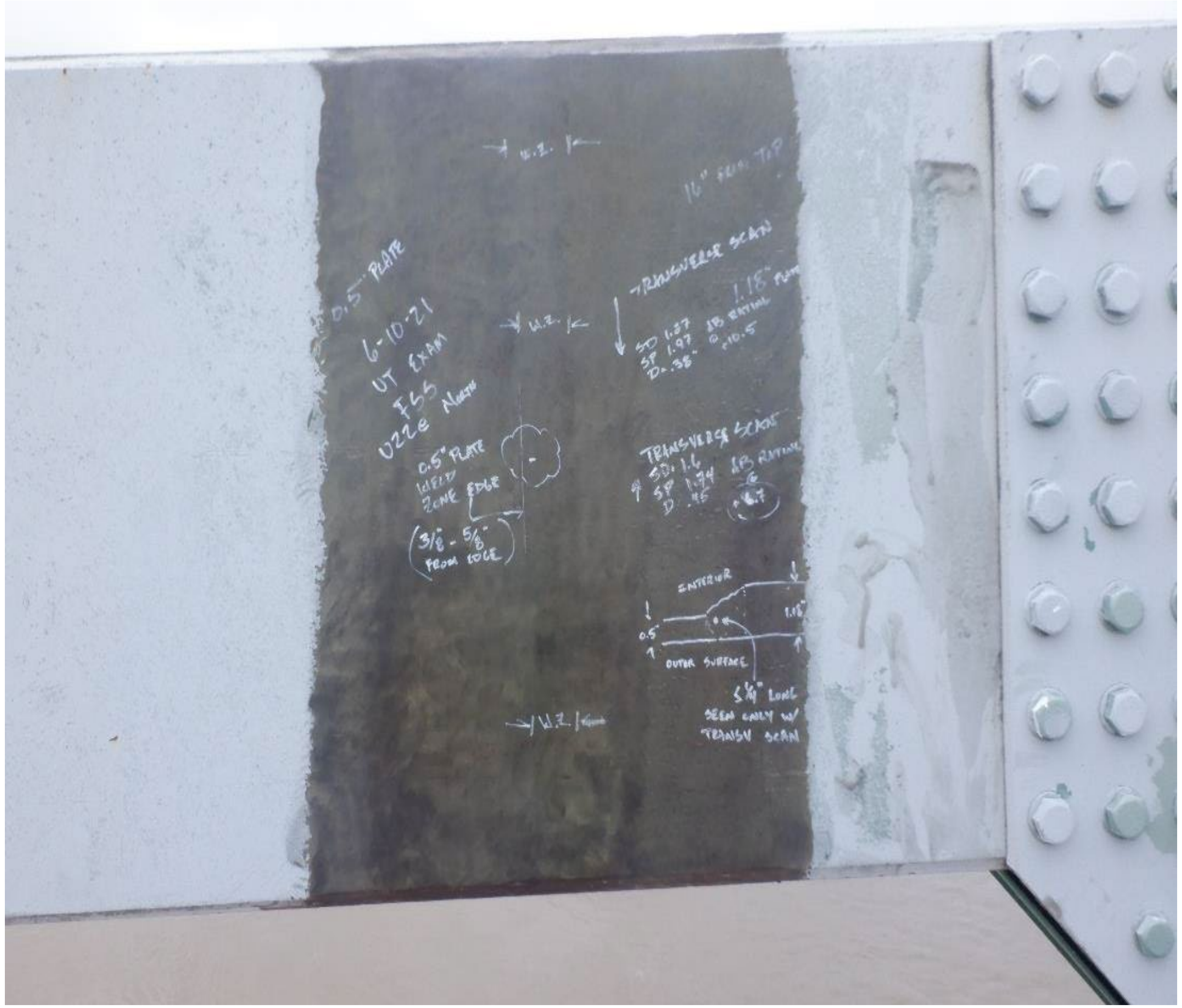
Phase 3 Repairs

Inspection, Testing, and Repair for long-term reliability

Phase 3 – Inspection

Day 15 - Day 59

- Full Penetration Butt Weld detail typical throughout structure
  - Potential for similar defects
  - Prevent future failure
- Arch Tie Members and Hanger Pins (Approx. 500 welds)
  - HNTB contracted CAN-USA
  - June 1st to June 23rd
- Arch Truss Members (Sampling)
  - MBI contracted Fickett
  - June 7th to June 11th and June 23rd to 25th



NDT Inspection of Arch Rib Member

Fracture Analysis

Figures obtained from WJE Fracture Investigation Report

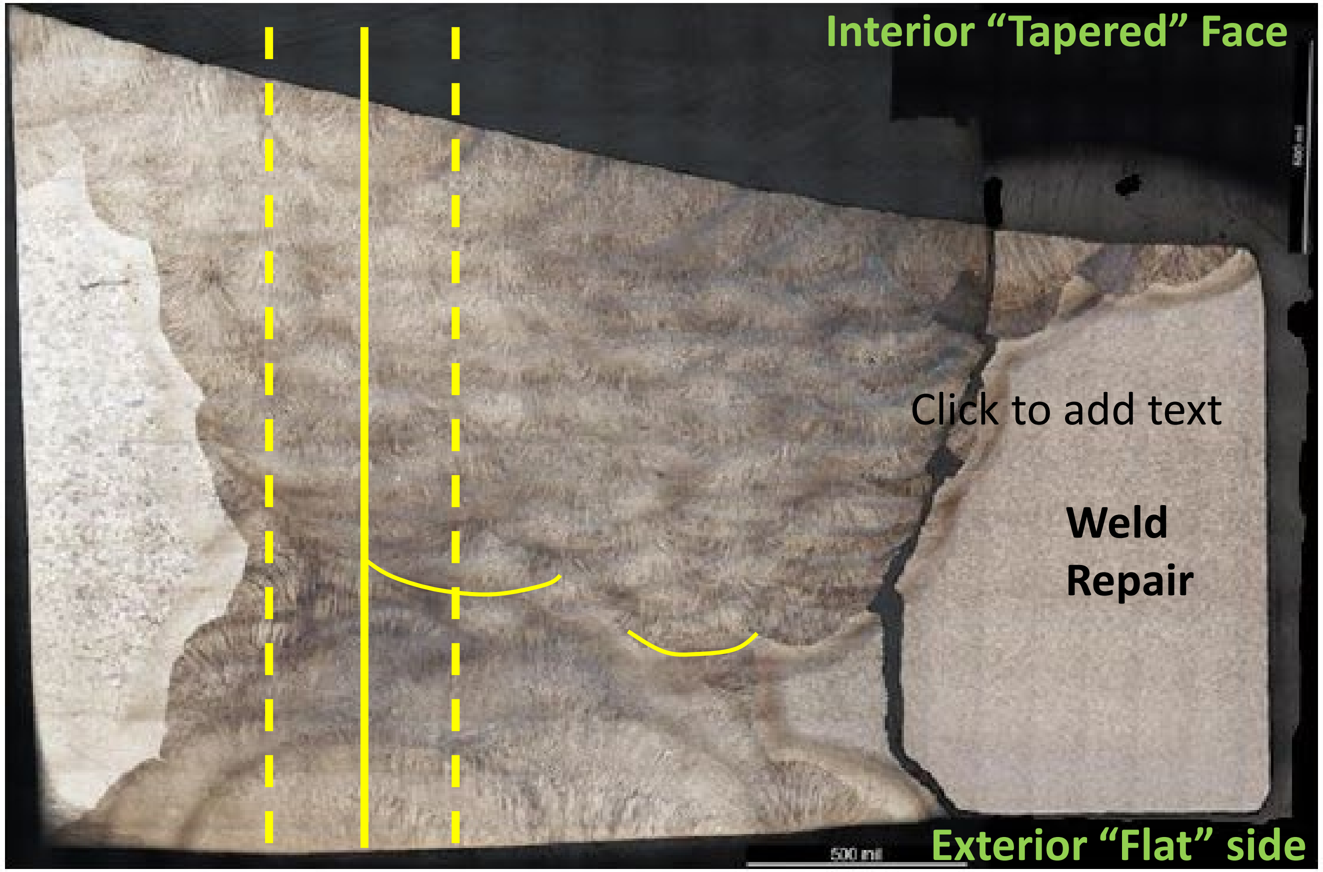


Figure 39. Primary preexisting crack weld profile.

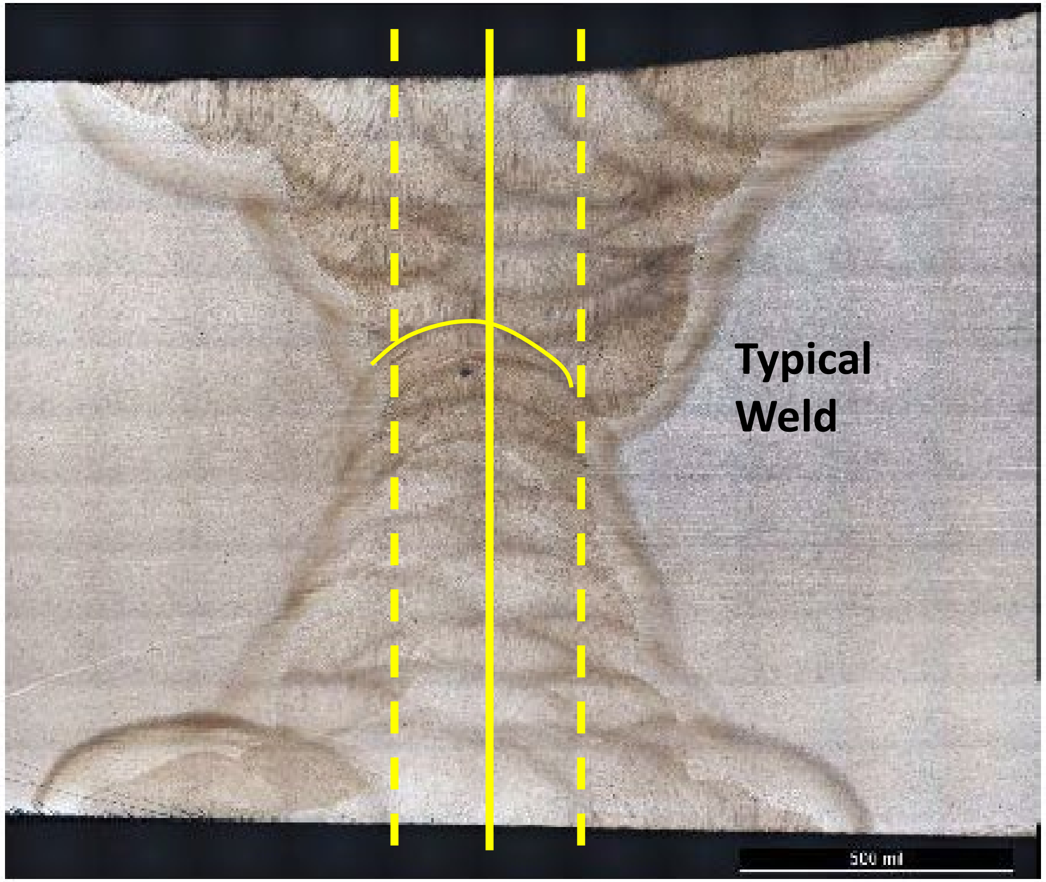
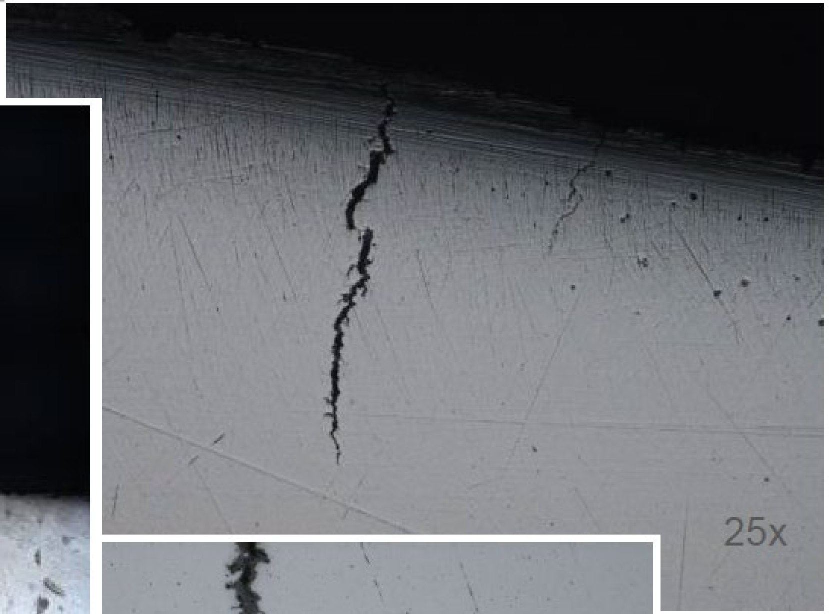


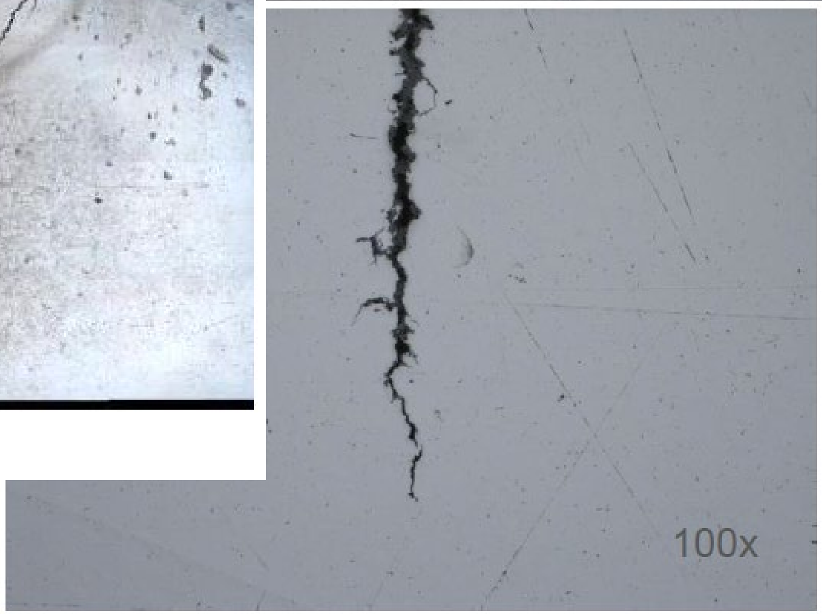
Figure 40. Core Sample SA008E weld profile.

Fracture Analysis

Figures obtained from WJE Fracture Investigation Report

Figure 54. Cracks in the top weld passes at location SA168W





Fracture Analysis

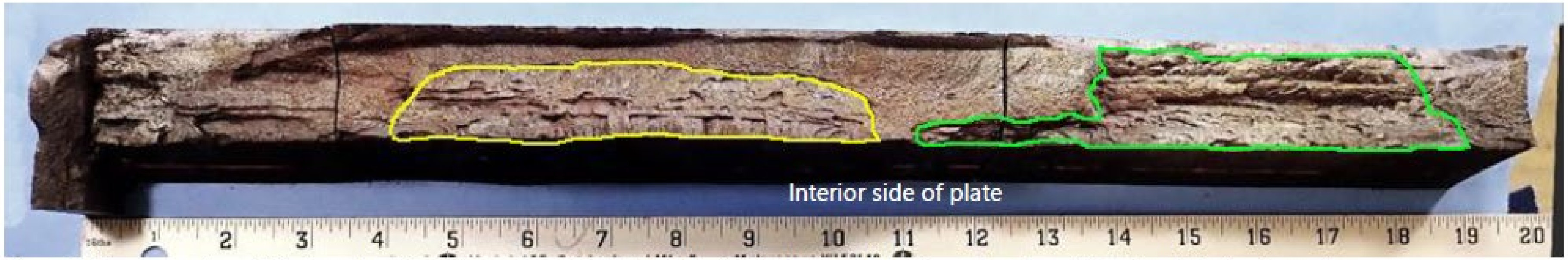


Figure 20. Lower portion of fracture showing Primary Preexisting Crack Region (yellow) and the Secondary Preexisting Crack Region (green).

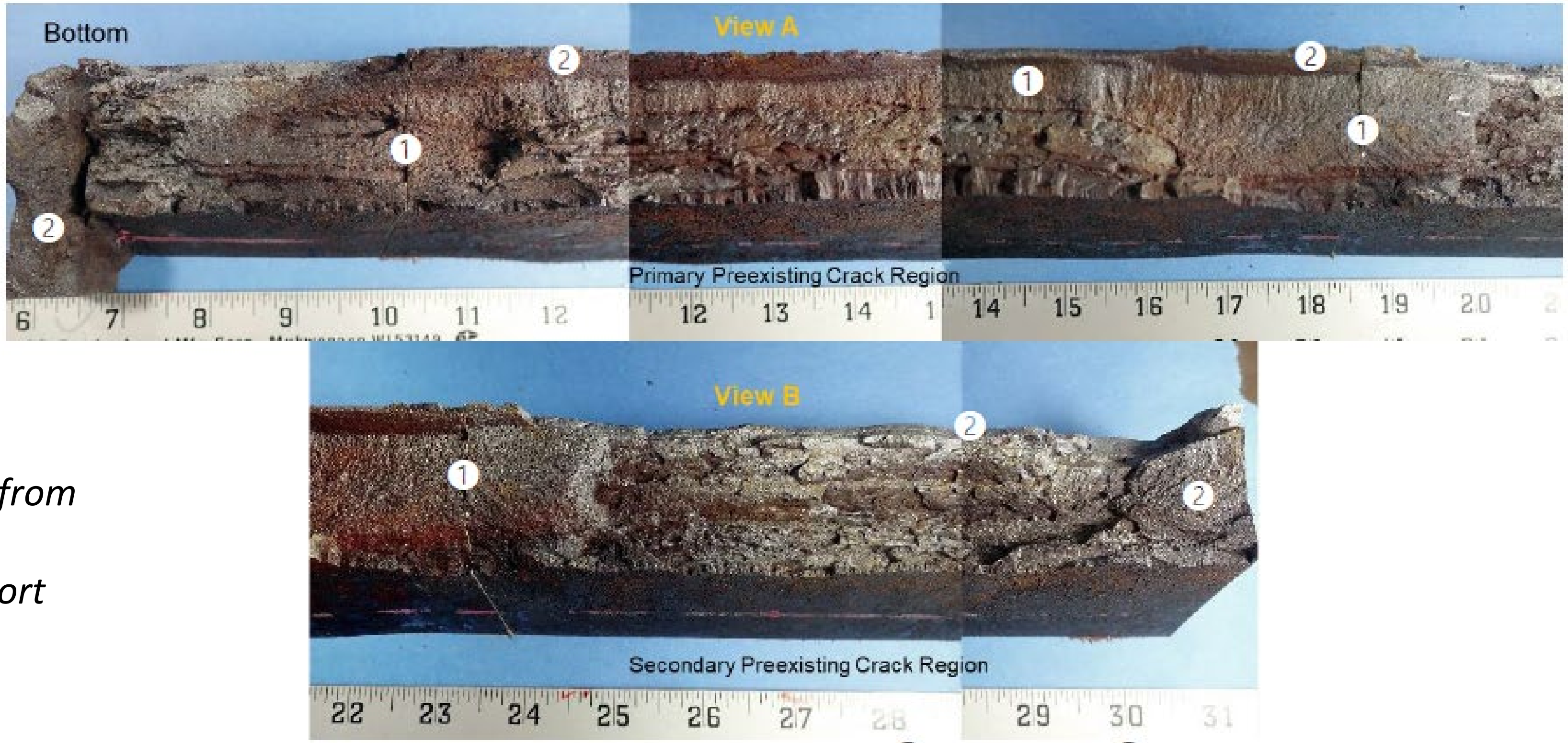


Figure 21. Higher magnification of the two preexisting crack regions after cleaning (❶ first fracture and ❷ second fracture).

Figures obtained from WJE Fracture Investigation Report

Fracture Analysis

Figures obtained from WJE Fracture Investigation Report

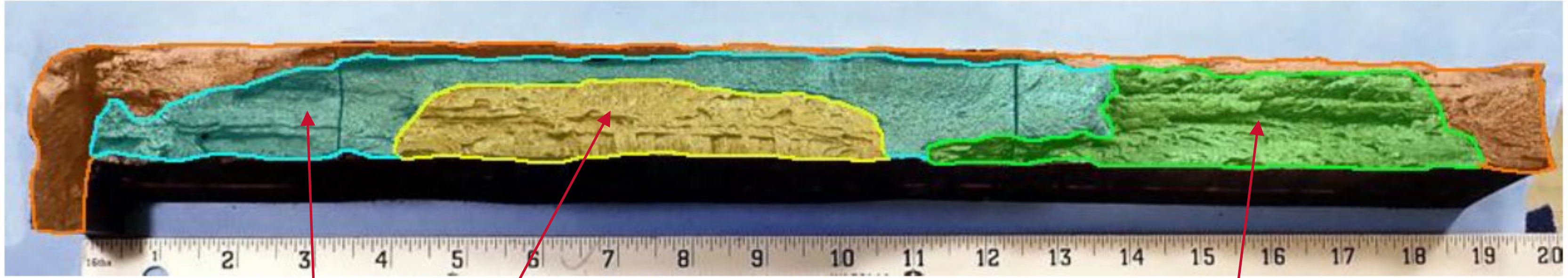


Figure 61. Lower portion of fracture, color-coded to indicate failure sequence.

Fracture Event #1

Secondary Pre-existing Crack

Primary Pre-existing Crack

Fracture Event #2

Fracture Event #3

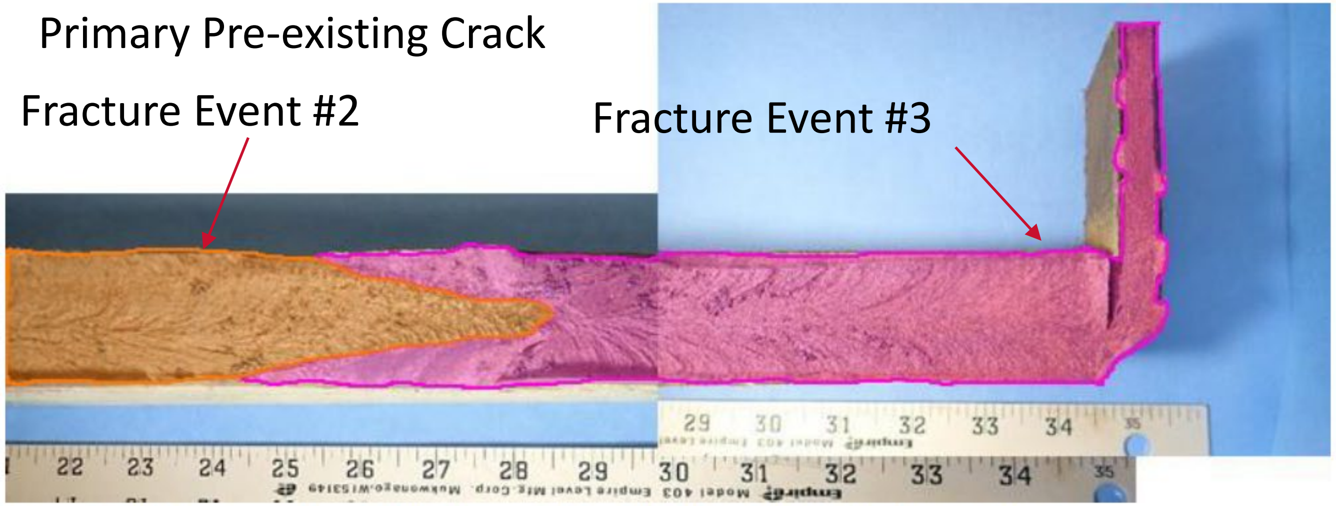
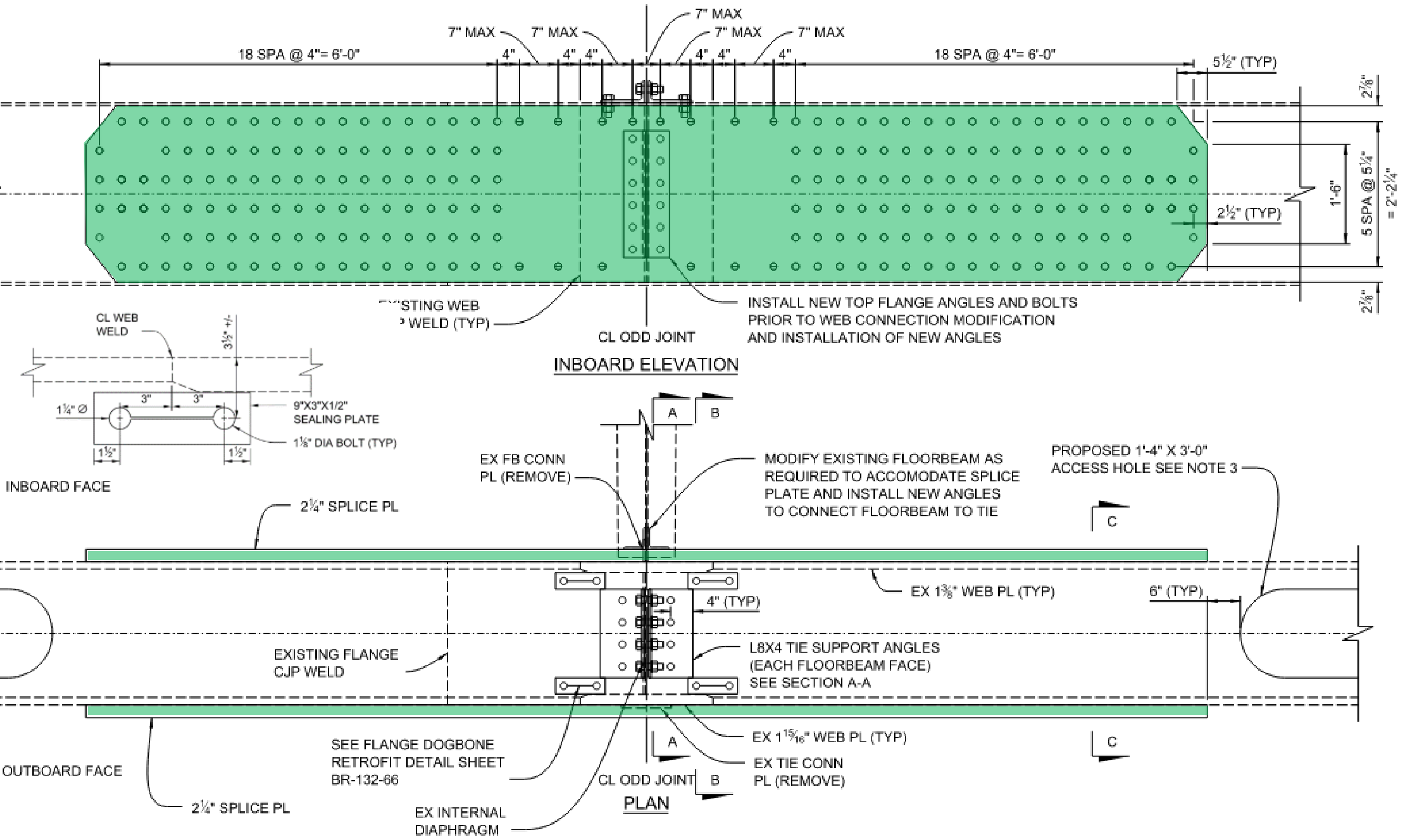


Figure 62. Upper portion of fracture, color-coded to indicate failure sequence.

Phase 3 – Repair Types (Typical Odd)



[Plating with “Dogbone” (17 locations)]

Phase 3 – Bridge Reopening

Westbound
August 2nd

Arkansas

Tennessee

Eastbound
July 31st

Questions ?

U.S. Department of Transportation
Federal Highway Administration
Office of Infrastructure

Non-Destructive Evaluation of Fracture Critical Members Fabricated from AASHTO M244 Grade 100 (ASTM A514/A517) Steel

FHWA Office of Bridges and Structures

November 2022

Disclaimer

- Except for any statutes or regulations cited, the contents of this presentation do not have the force and effect of law and are not meant to bind the public in any way. This presentation is intended only to provide information to the public regarding existing requirements under the law or agency policies.

Agenda

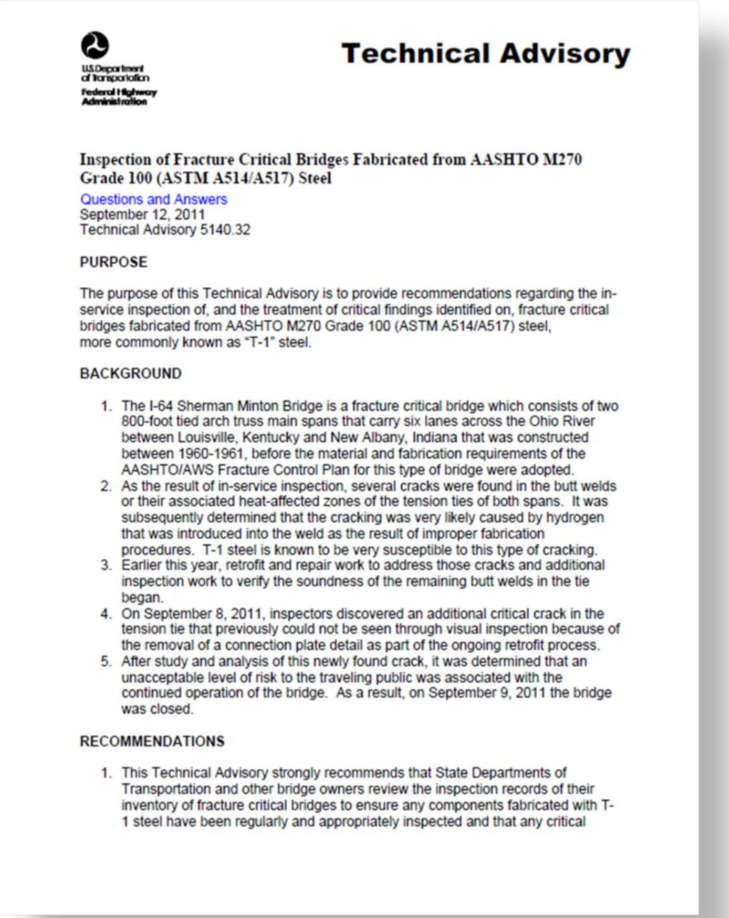
- Background
  - Technical Advisory 5140.32
  - December 13, 2021 Memo "Non-Destructive Testing of Fracture Critical Members Fabricated from AASHTO M244 Grade 100 (ASTM A514/A517) Steel"
- Memo Implementation Discussion

U.S. Department of Transportation
Federal Highway Administration

Background

Technical Advisory 5140.32

- Released September 12, 2011, shortly after Sherman Minton Bridge closure
- *Recommends* that bridge owners:
  - Review inspection records to ensure components fabricated from T-1 steel have been regularly and appropriately inspected,
  - Follow up on deficiencies, prioritizing components primarily in tension (arch ties, hangers, truss members), and
  - Verify the soundness of all butt welds in tension in members fabricated prior to FCP adoption



U.S. Department of Transportation
Federal Highway Administration

Technical Advisory

Inspection of Fracture Critical Bridges Fabricated from AASHTO M270 Grade 100 (ASTM A514/A517) Steel

Questions and Answers
September 12, 2011
Technical Advisory 5140.32

PURPOSE

The purpose of this Technical Advisory is to provide recommendations regarding the in-service inspection of, and the treatment of critical findings identified on, fracture critical bridges fabricated from AASHTO M270 Grade 100 (ASTM A514/A517) steel, more commonly known as "T-1" steel.

BACKGROUND

1. The I-64 Sherman Minton Bridge is a fracture critical bridge which consists of two 800-foot tied arch truss main spans that carry six lanes across the Ohio River between Louisville, Kentucky and New Albany, Indiana that was constructed between 1960-1961, before the material and fabrication requirements of the AASHTO/AWS Fracture Control Plan for this type of bridge were adopted.
2. As the result of in-service inspection, several cracks were found in the butt welds or their associated heat-affected zones of the tension ties of both spans. It was subsequently determined that the cracking was very likely caused by hydrogen that was introduced into the weld as the result of improper fabrication procedures. T-1 steel is known to be very susceptible to this type of cracking.
3. Earlier this year, retrofit and repair work to address those cracks and additional inspection work to verify the soundness of the remaining butt welds in the tie began.
4. On September 8, 2011, inspectors discovered an additional critical crack in the tension tie that previously could not be seen through visual inspection because of the removal of a connection plate detail as part of the ongoing retrofit process.
5. After study and analysis of this newly found crack, it was determined that an unacceptable level of risk to the traveling public was associated with the continued operation of the bridge. As a result, on September 9, 2011 the bridge was closed.

RECOMMENDATIONS

1. This Technical Advisory strongly recommends that State Departments of Transportation and other bridge owners review the inspection records of their inventory of fracture critical bridges to ensure any components fabricated with T-1 steel have been regularly and appropriately inspected and that any critical

Source: FHWA

December 13, 2021 Memo

- *Requires* that State DOTs:
  - Identify bridges with fracture critical members fabricated from T-1 steel without requirements to meet the provisions of the AASHTO/AWS FCP and document them in the FCM inspection procedures[1]
  - Supplement hands-on inspection of T-1 FCMs with Non-Destructive Evaluation verifying the soundness of butt welds in tension[2]
    - Unless previous verification has been documented
    - Previous verification needs have been performed a minimum of 48 hours after original welding (≤ 2" thick, 72 hours for > 2" thick)
    - Complete testing by March 31, 2024
  - Classify rejectable indications (using AASHTO/AWS criteria) as critical findings[3]
  - By March 31, 2022, Report an inventory of bridges with T-1 FCMs and actions taken to perform verification and follow up on findings[4]
    - Update reporting data at six-month intervals

[1] 23 CFR 1.36, 23 CFR 650.313
[2] 23 CFR 1.36, 23 CFR 650.313
[3] 23 CFR 1.36, 23 CFR 650.313
[4] 23 CFR 1.36, 23 CFR 650.315

Memo Implementation

What Type and Coverage of NDE is Required?

- Refer to clauses 8 and 12 of AASHTO/AWS D1.5:2020 (note: Clause 8 was previously Clause 6 in D1.5:2015), which covers welding inspection
  - Clause 12.16.2.1 Requires 100% RT and UT of butt welds in tension for fabrication
  - Radiographic testing will be physically difficult in-situ → UT will satisfy the requirements of the memo
- Part C of Clause 8 sets forth procedures and standards for ultrasonic testing of groove welds
  - Equipment qualification and calibration
  - Evaluation procedures
  - *Challenge – D1.5 does not provide minimum performance qualifications for NDE personnel*
- Annex J of AASHTO/AWS D1.5:2020 (previously Annex K in D1.5:2015) sets forth procedures and standards for phased array ultrasonic testing

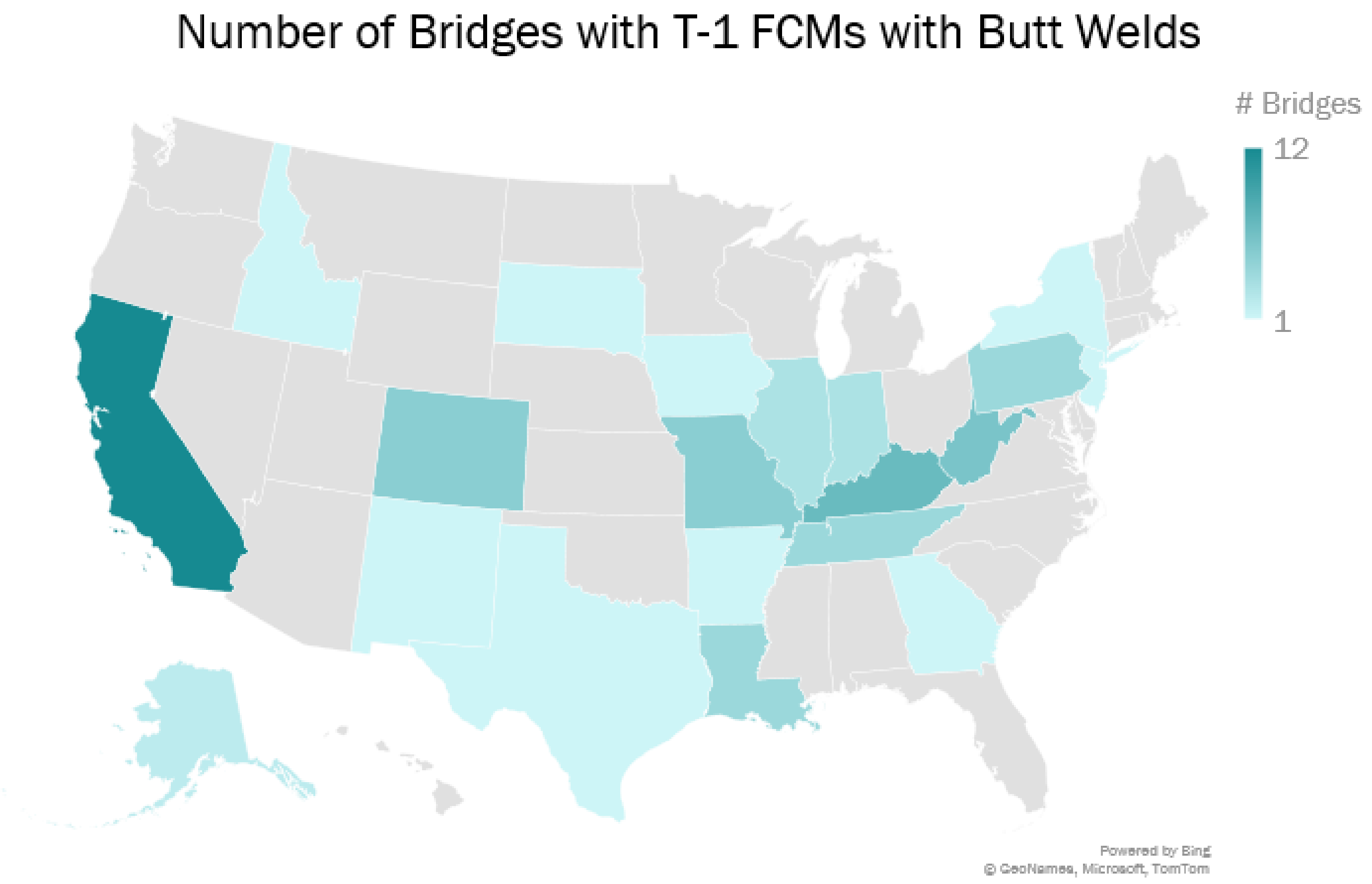
What is a “Rejectable Indication”?

- Refer to Clause 8 of AASHTO/AWS D1.5:2020
  - Part D – Weld Acceptance Criteria
  - Table 8.3 – “UT Acceptance-Rejection Criteria – Tensile Stress”
- For Phased Array UT, refer to Annex J, Clause J10.2
  - Table J.3 – “PAUT Acceptance Criteria”

Inventory of Bridges Subject to Memo Requirements
(as of September 30, 2022)

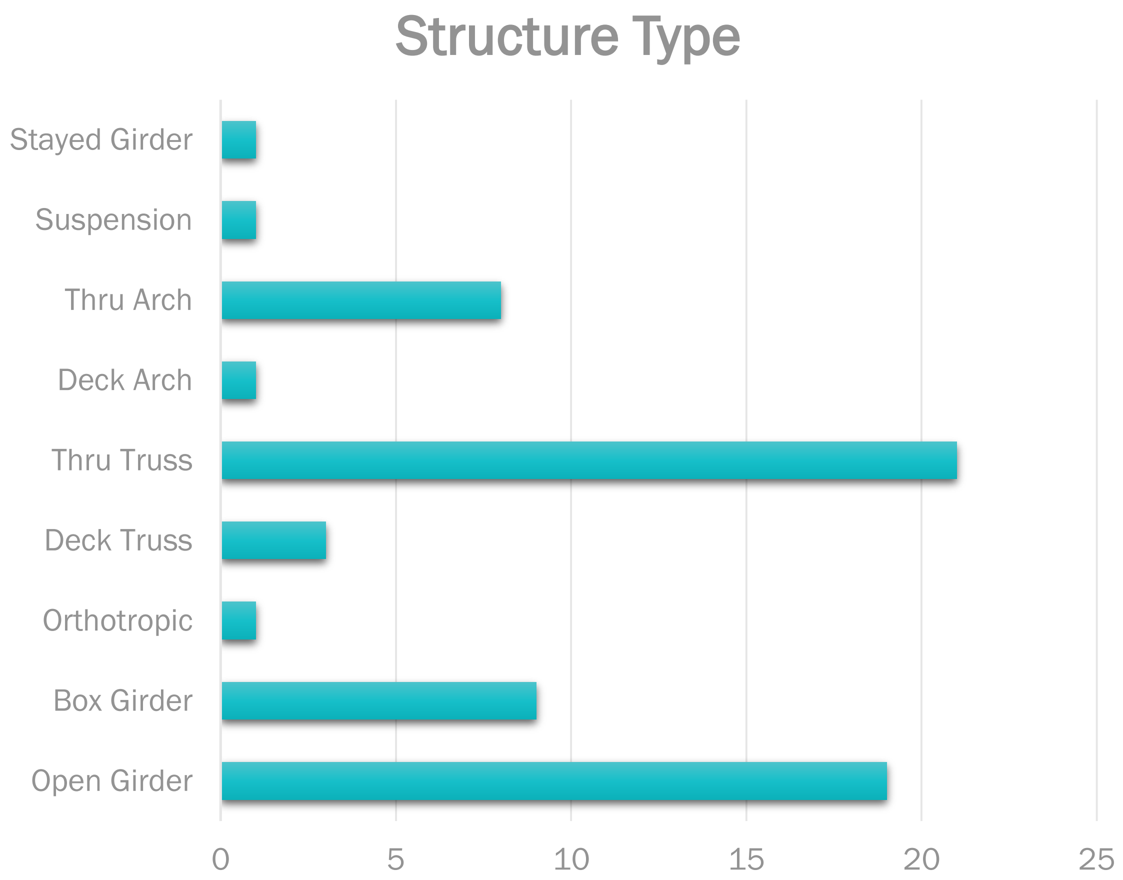
Alaska	2	Louisiana	4
Arkansas	1	Missouri	5
California	12	New Jersey	1
Colorado	5	New Mexico	1
Georgia	1	New York	1
Iowa	1	Pennsylvania	4
Idaho	1	South Dakota	1
Illinois	3	Tennessee	4
Indiana	3	Texas	1
Kentucky	7	West Virginia	6

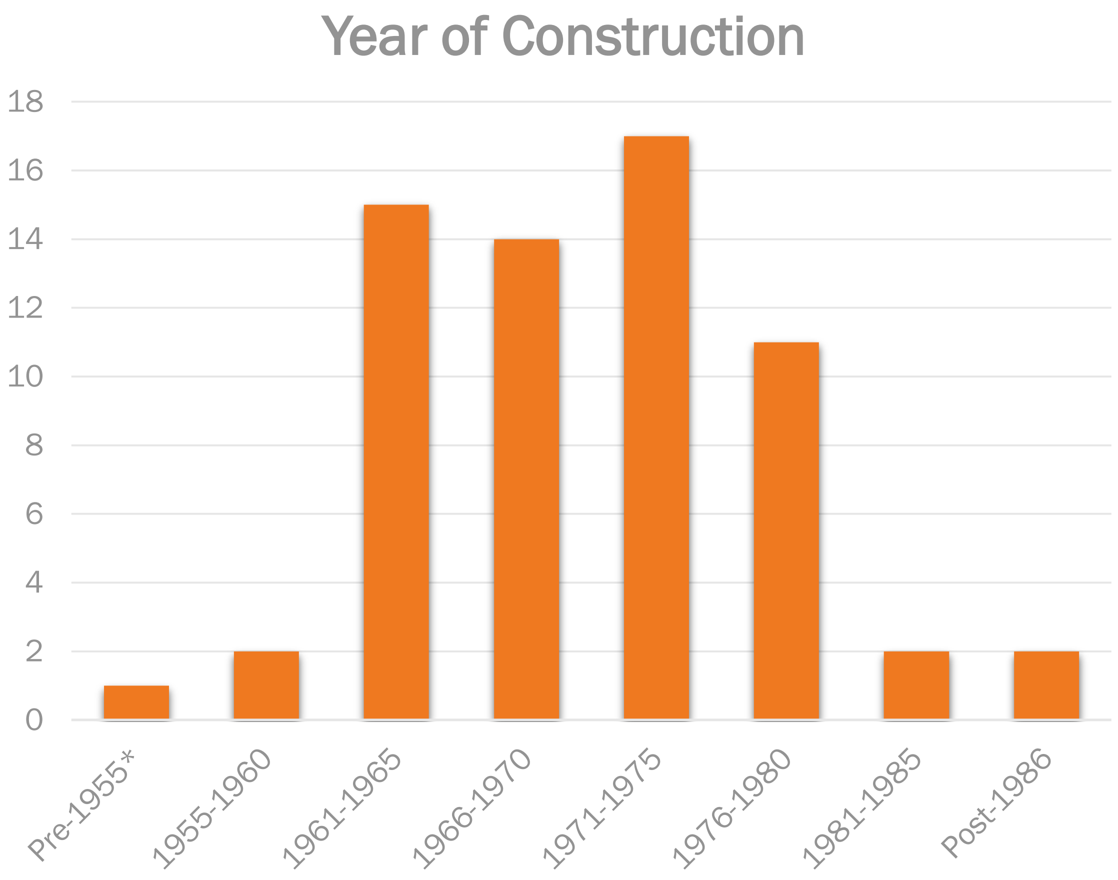
Total: 64



Based on data States submitted under the memorandum requirements.

Bridge Type and Age





Based on data States submitted under the memorandum requirements.

Tested Bridges

- Number of bridges tested prior to memo release - 6:
  - 1970s – 1
  - 1980s – 2
  - 2010s – 2
  - 2020-2021 – 1
  - Of these, the number with rejectable indications: 4 (66%)
- Number of bridges tested since memo release – 3
  - Of these, the number with rejectable indications: 2 (66%)

Based on data States submitted under the memorandum requirements.

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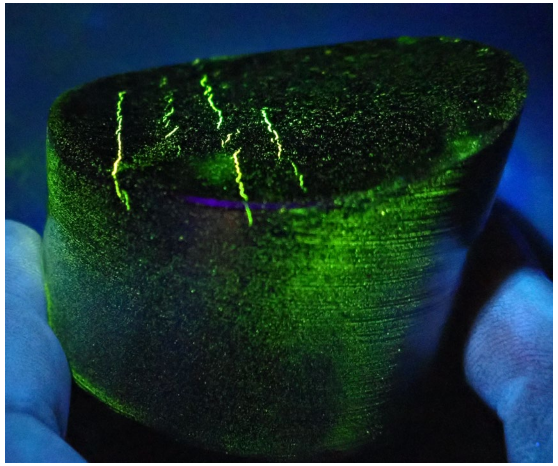
Federal Highway Administration

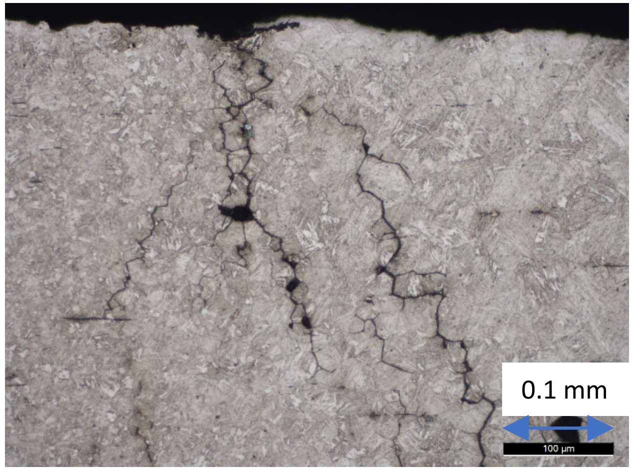
Thank you!

NDT of T-1 Steel
and Lessons Learned

Removal of Cores at Rejectable Flaws

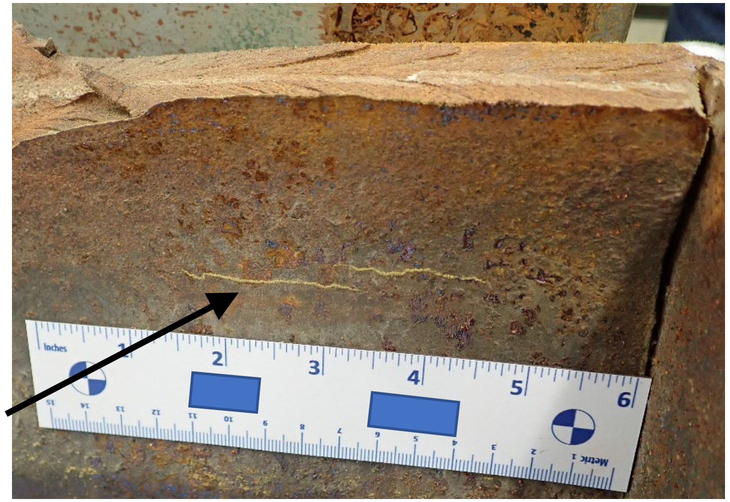
- CAN-USA located rejectable indications during PAUT inspection
- WJE removed 10 cores from welds with rejectable indications
- 4 cores contained crack-like defects on the inside surface of the weld
  - Wet, florescent MT
- Crack heights varied with maximum up to 3/16”
- Cracks had branching morphology following grain boundaries
  - Worse UT reflector than a flat plane

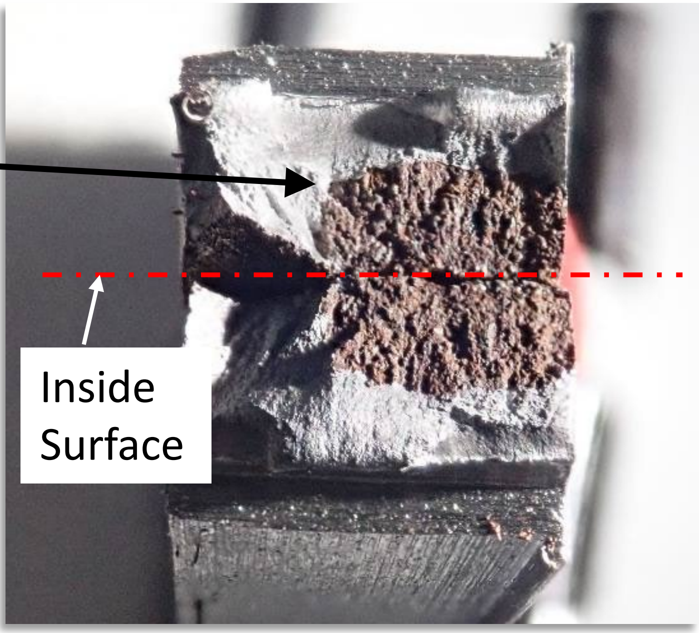




Conventional UT of Extracted Fracture Pieces

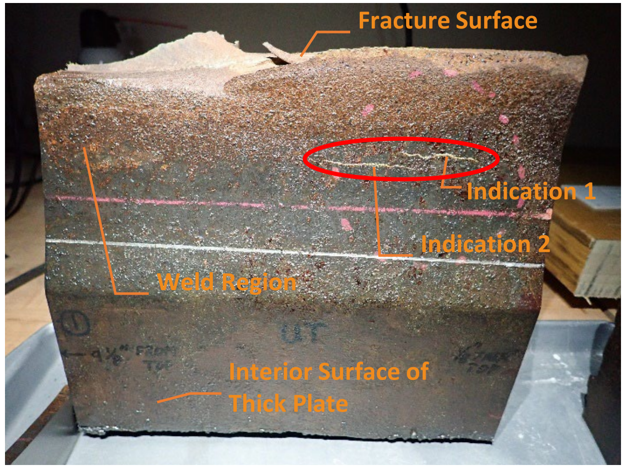
- Fracture specimen had hydrogen cracks in weld region which remained attached to thick base plate
- Initially scanned with conventional UT with 2 rejectable indications using 70° transducer
  - Group of two surface-breaking cracks (Shown in top photo)
    - Class A; UT Combined Length: 3.3”
    - Missed in 1982 UT Inspection
  - Smaller surface-breaking crack (Shown in bottom photo)
    - Class B; UT Length: 1.1”
    - Destructive Evaluation
      - 3/32”H x 5/32”L
    - Detected in 1982 with same Indication Rating
      - Reported as “Accept” and characterized as “Slag”
      - Although exceeded the UT acceptance criteria limits

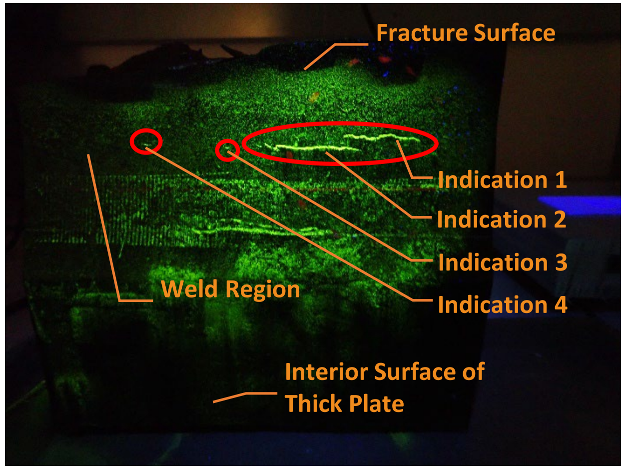


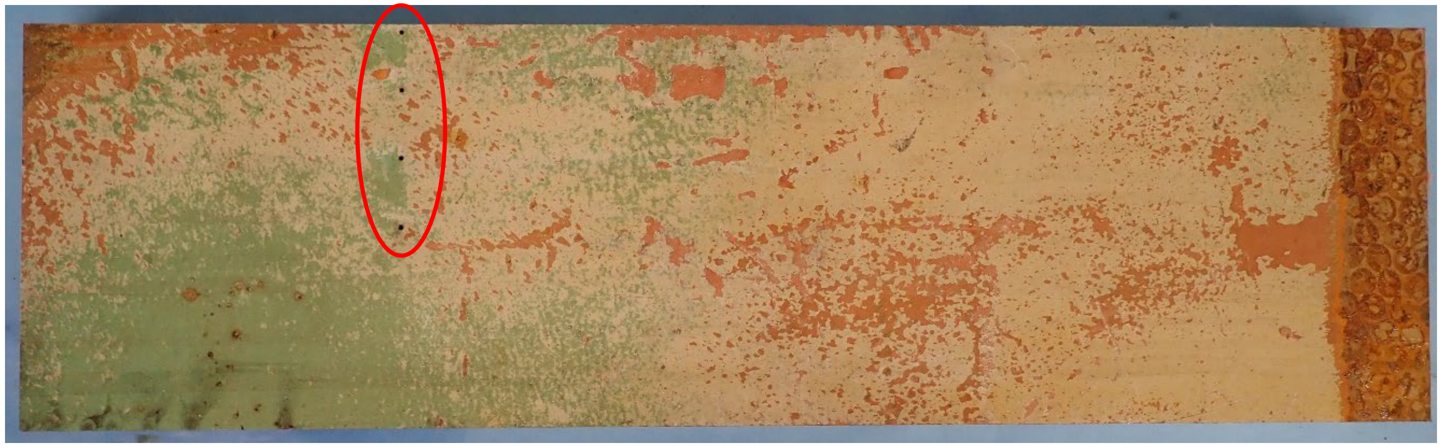


MT and PAUT of NDE Verification Sample

- Saved portion of weld as a NDE Verification Sample
  - Containing 4 MT indications
- Performed PAUT per AWS D1.5 Annex J
- Time-Corrected Gain (TCG)
  - Used in lieu of standardized attenuation factor for PAUT
  - Fabricated calibration block from I-40 base metal
  - Much lower attenuation in T-1 material than 1018 IIW block



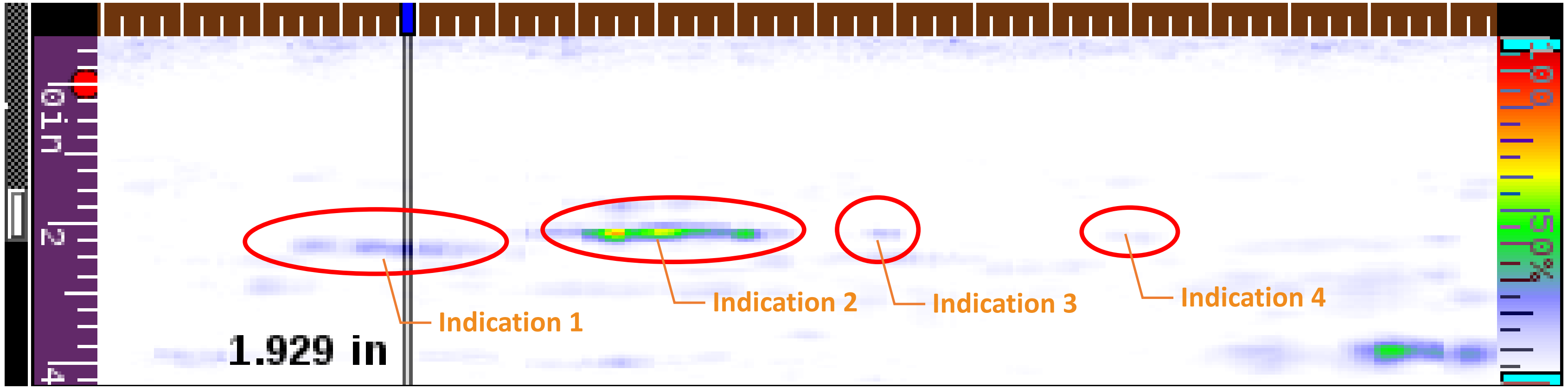




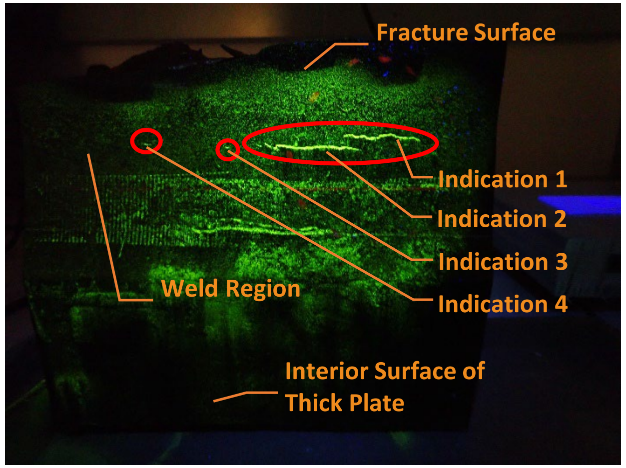
PAUT of NDE Verification Sample

NDE Method	Indication 1	Indication 2	Indication 3	Indication 4
Conventional UT	Reject (Class A) {Grouped with Ind. 2}	Reject (Class A) {Grouped with Ind. 1}	Not Recorded	Not Recorded
PAUT Annex J (Line Scanning)	Accept (Class D)	Reject (Class B)	Accept (Class D)	Accept (Class D)
PAUT (Raster Scan Modifications[1])	Reject (Class B)	Reject (Class A)	Accept (Class C)	Accept (Class B)

[1]Peak amplitude measured during raster scanning and length measured as region exceeding disregard limit (DRL)



PAUT Annex J C-Scan (Pseudo-Top View)



Why is Transducer Raster/Rotation Important?

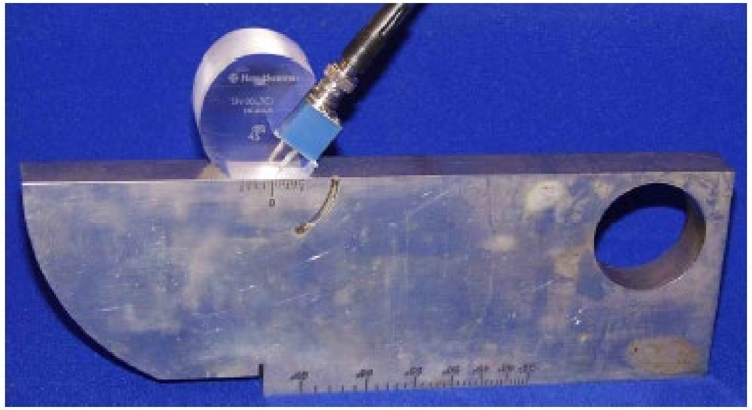
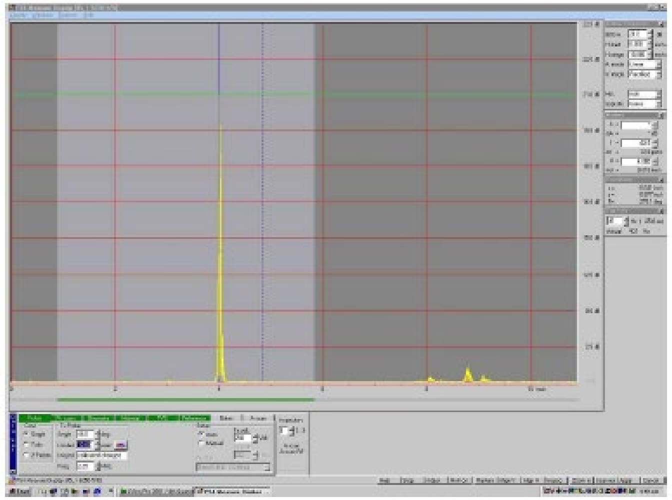


Figure 19. Sound entry point check: Photograph shows the transducer position on the IIW reference block.



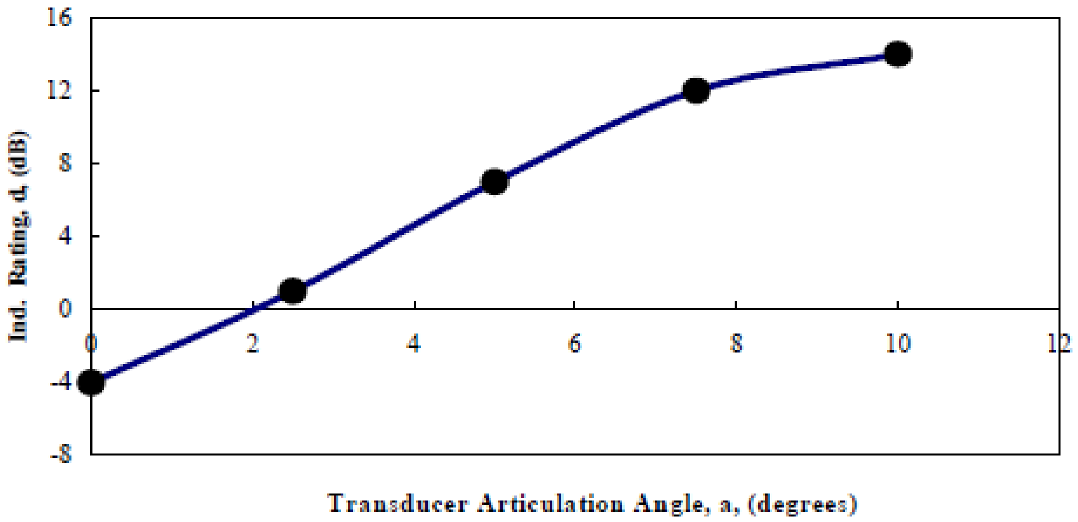


Figure 147. Influence of transducer articulation angle on the maximum amplitude of the reflected signal.

Washer et al.

Effects of Lack of Transducer Raster/Rotation

- If rejection criteria are based on maximum amplitude, then the scanning technique must ensure maximum amplitude is obtained

- This will <u>not</u> be guaranteed simply by using PAUT (*you will likely be told otherwise…*)
  - Effect of fixed index offset
  - Effect of not raster/rotation scanning

Takeaway on PAUT Applications for T-1

- Need to account for differences in attenuation when performing PAUT
  - 5 MHz PAUT transducers are more susceptible to attenuation differences
- Line scanning using AWS D1.5 Annex J
  - Pros
    - Record and keep the full encoded line scan as part of permanent bridge file
  - Cons
    - Does not measure peak amplitude of indication which results in lower defect classification and lower rejection rate
    - Recognized that the rejection criteria in Annex J are less stringent
  - Recommendations
    - Lower acceptance criteria amplitude limits (thus increasing sensitivity)
      - 8-10% FSH may be a potential surface crack
    - Follow-up with raster scanning on suspect indications
      - Amplitude >DRL (>25% FSH) may be a potential surface crack

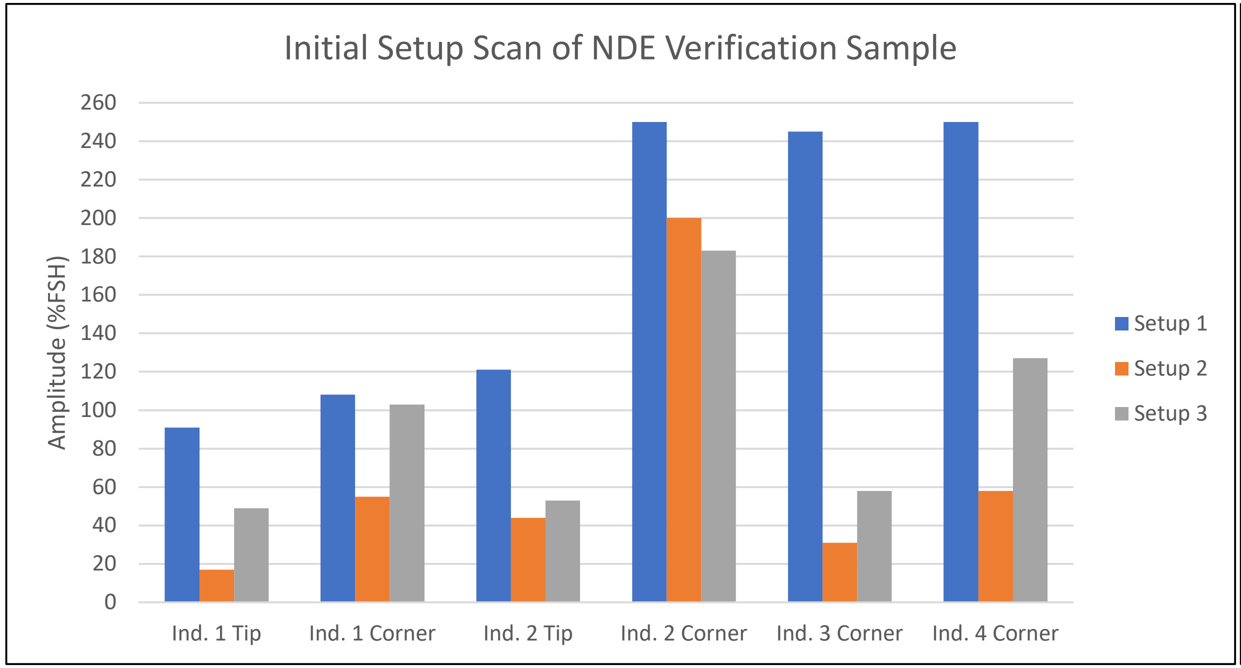
Procedure Verification

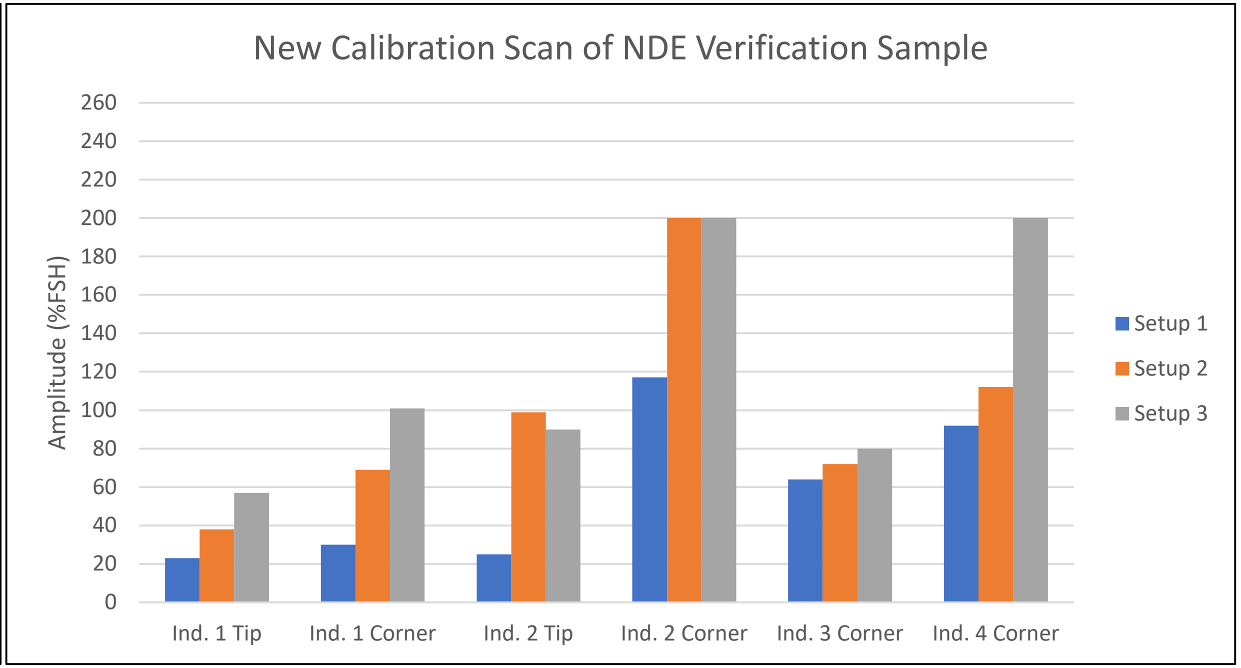
- CAN-USA reevaluated PAUT data based on laboratory observations
  - Susceptibility of hydrogen cracking on inside surface of the weld
  - Hydrogen cracks may have low amplitude
- CAN-USA scanned NDE Verification Sample using field inspection PAUT setups to verify inspection sensitivity
  - Used 3 different transducer, wedge, and instrument combinations
  - Various frequencies and probe parameters (number and size of elements, etc.)

	Setup 1	Setup 2	Setup 3
Frequency	5 MHz	5 MHz	2.25 MHz
Number of Active Elements	16 elements	32 elements	16 elements
Active Aperture x Elevation	9.6 mm x 10 mm	32 mm x 10 mm	9.6 mm x 10 mm

Calibration and Probe Parameters

- Consistency of raster scan peak amplitude slightly improved after calibration using T-1 calibration block
- Differences in peak amplitude for various transducer/instrument combinations are likely due to inherent transducer characteristics





Experiences From Other T-1 Bridge Inspections

- Expect to find rejectable indications
  - Welds were not UT inspected at initial fabrication
  - Some welds will likely have rejectable indications which will be considered critical findings (general rule of thumb is ~10% of welds will be rejectable)
  - Other flaw types observed in T-1 welds: Cracks unassociated with hydrogen, lack of fusion, slag inclusions, and porosity
- PAUT can aid in characterization, but core removal and metallurgical evaluation is preferred
  - Real world weld flaws have variable morphology
    - Lack of Fusion ⟷ Crack
- Eddy Current can be used to verify or detect weld locations
  - Actual weld location may vary from shop drawings
  - Simple application; not using for evaluation of weld quality

Recommendations for Future T-1 Inspection

- Conventional UT
  - Recommend as primary inspection technique
  - Prescribed raster scanning procedure
  - Consistent and prescribed transducer characteristics (size, frequency, wedge)
  - Prescribed attenuation factor
    - Likely conservative for T-1 steels (my opinion)
  - Acceptance criteria limits are different (more conservative) than Annex J
    - Hydrogen cracks may be low in amplitude

- PAUT
  - Use for supplemental scanning to characterize rejectable or suspect indications
  - If performing primary scans with PAUT
    - Raster scan to maximize amplitude
    - Calibrate on material of similar acoustic properties

Recommendations regarding Ultrasonic Testing, Calibration, and Performance Testing

Robert J. Connor
Purdue University

Glenn A. Washer
University of Missouri

November 10, 2022

Discussion Points in this Presentation

- Amplitude-based acceptance/rejection criteria of AWS
  - Importance of proper calibration
- Conventional UT vs PAUT
  - Should I specify one over the other and why
- Importance of removing coatings
- Importance of performance testing

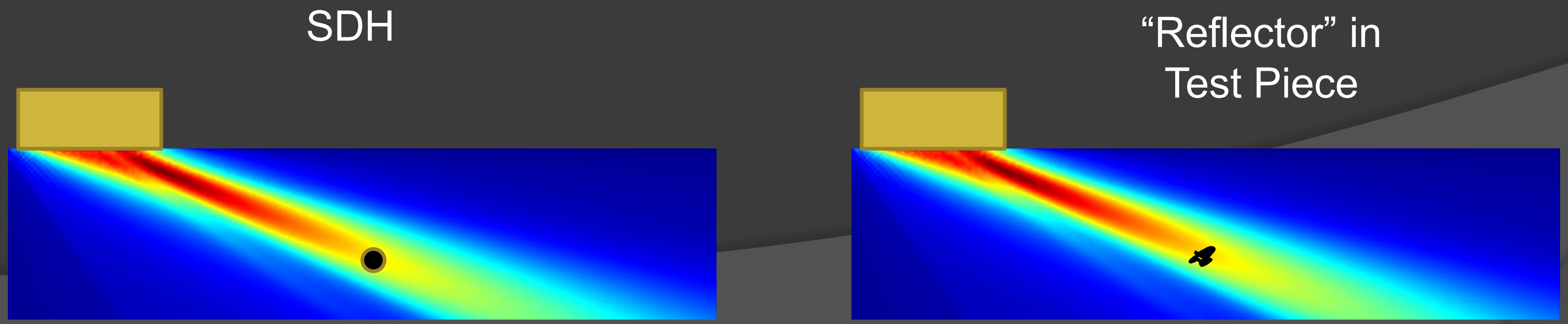
Amplitude-based Criteria

- AASHTO/AWS D1.5 conventional UT acceptance/rejection criteria are amplitude based
- Basic concept:
  - Sound is introduced into the test piece (the joint)
  - “Defects” reflect sound back to the technician
    - Many “things” can reflect sound, we just *assume* they are defects
  - No sound reflected = no defect
  - Assume defect criticality is proportional to the amount of sound reflected in amplitude-based methods

Amplitude-based Criteria

- Approach for conventional UT references the sound reflected from a 0.06” diameter side drilled hole (SDH) to sound from unknown reflector (defect) in the test piece

- Technician adjusts gain (volume) on the UT machine until reflector in test piece produces the same signal as SDH in Calibration Block



Amplitude-based Criteria

- Obvious that there are several major assumptions in this approach

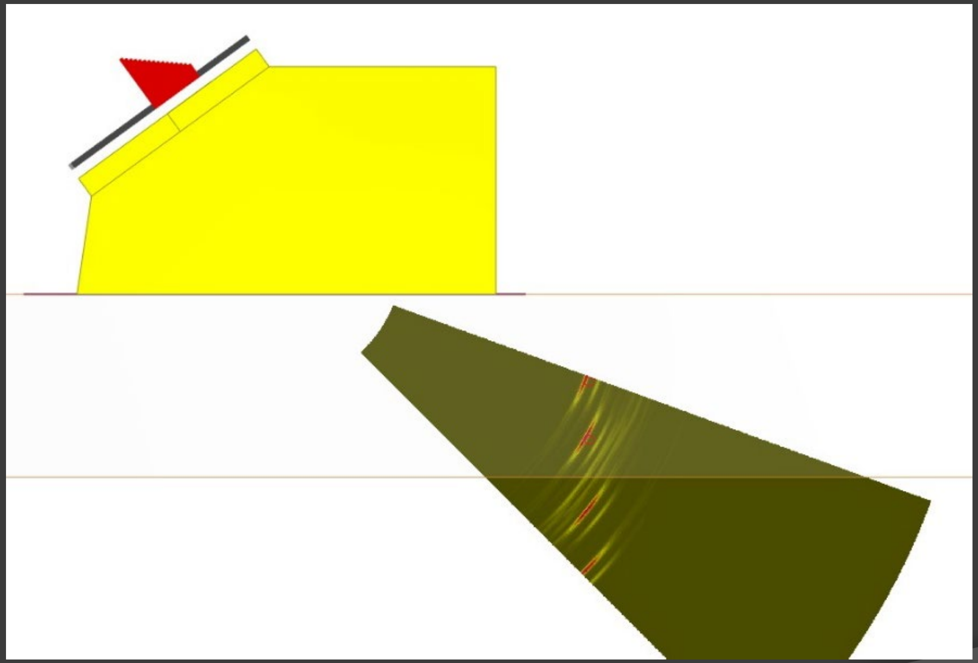
1. The rejection criteria are meaningful in terms of the performance of the structure
   - i.e., -8 dB compared to sound reflected from a SDH is critical
2. Acoustic properties between test piece and calibration block are the same
   - Attenuation of sound in steel
   - Velocity of sound in steel

The take away?

- Ensure that the steel calibration block is acoustically similar to the steel you are inspecting
  - Minimizes potential for over- or under-rejection rates
  - Overly conservative is expensive > unneeded repairs?
  - Unconservative results can be expensive > fracture?

- Current S-BRITE study indicates that using a modern Q/T calibration block (e.g., HPS 100 or similar) will most likely be acoustically similar to close to older T1 steels

Use of Conventional UT vs PAUT?



Use of Conventional UT vs PAUT

- S-BRITE recommends using conventional UT with existing AWS D1.5 criteria
- Why?

1. Line scan of PAUT gives up raster/rotation scanning if D1.5 Annex J is specified
   - Technician “may” maximize dB through manual manipulation of the probe ….but is not required
   - Critical when amplitude-based criteria are used in order to maximize the reflected amplitude
2. Current PAUT rejection criteria in Annex J of D1.5 will accept indications that conventional will reject
   - There is no rational for this that has been documented
3. More technicians are available for conventional UT work

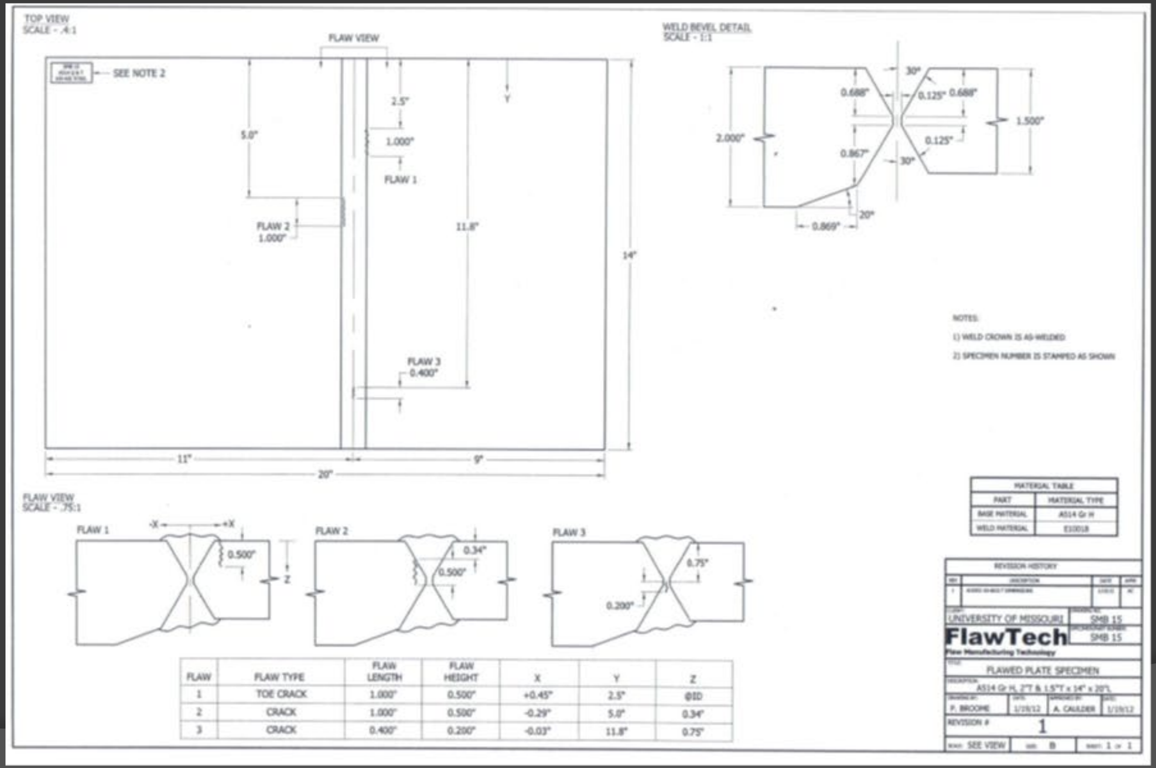
Use of Conventional UT vs PAUT

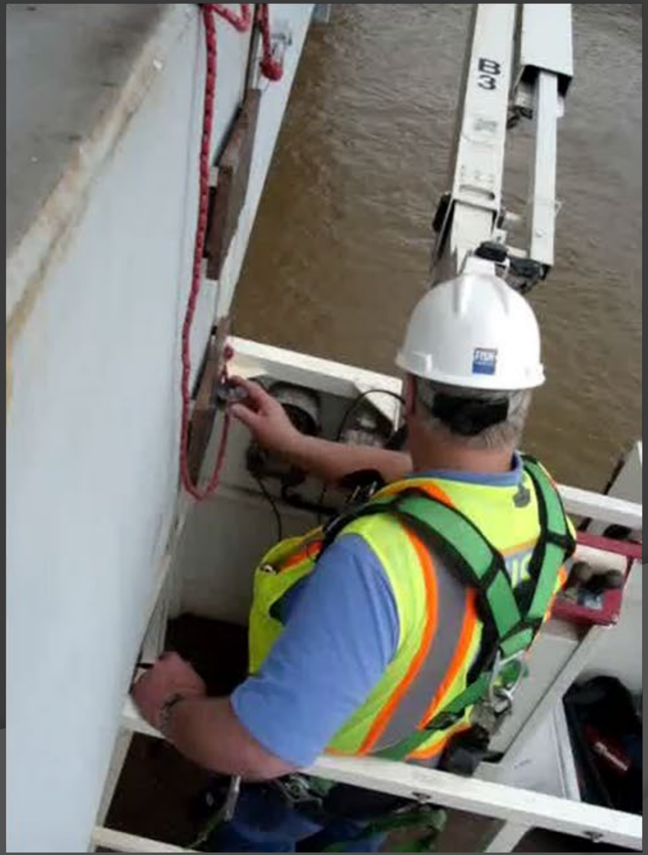
- The take away?

  The effects of no raster scan and probe rotation, fixed index offset, and less stringent rejection criteria mean you will accept larger defects if PAUT is used soley based on Annex J of D1.5

- If PAUT is used:
  - Record and keep the full encoded line scan as part of record
    - This is an advantage of PAUT
  - Require manual manipulation of the probed to maximize the dB response when
    - This is suggested for any indication that is >10% screen height
    - Take a screen shot of this indication maximum and document location
  - Recognized that the rejection criteria in Annex J are less stringent

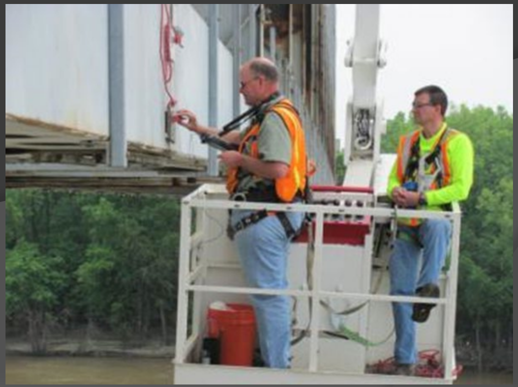
The Need for Performance Testing



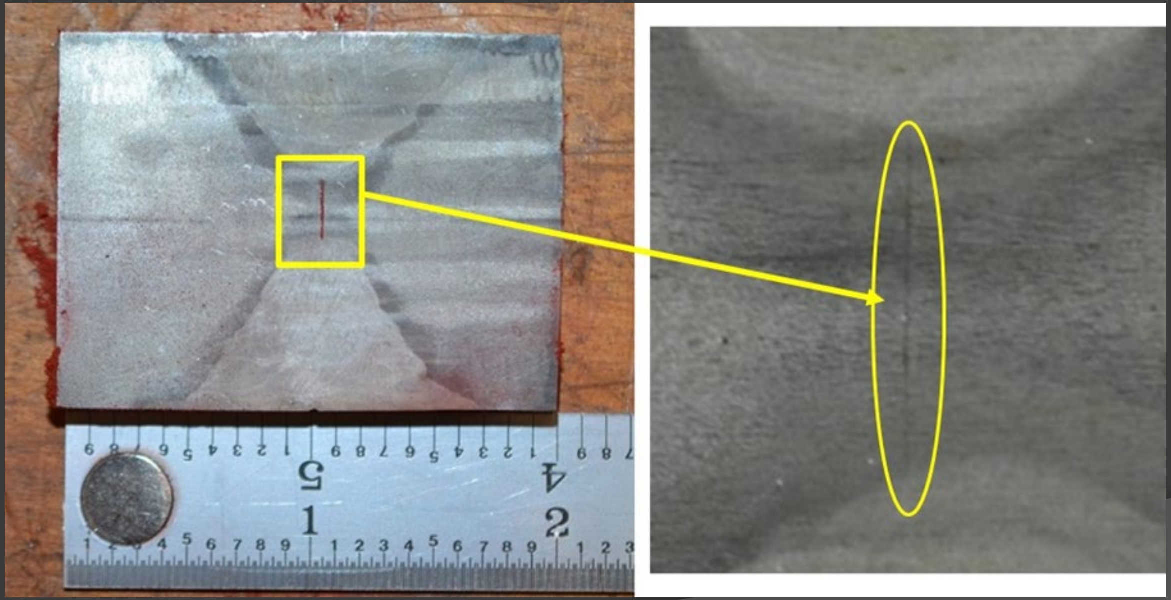


In-situ Performance Testing is Strongly Encouraged

- Why?
  - When implemented, results confirm that there is tremendous scatter in data from current work force
    - True on real bridges and in lab
    - "Our guys are good" should not be assumed
      - And if the are "good" then they will have no problem with the test…right?
    - Are the technicians following the procedures that you specified?
- Must recognize that very serious and possibly costly decisions will be made based on the results of the NDT
  - You don't want to miss real defects
  - You don't want to fix defects that are not there
- Let's look at some round robin data



What has performance testing shown?



Why is dB and Length Scatter a Concern?

Note how close Defect Classes are!

Table 6.3
UT Acceptance-Rejection Criteria—Tensile Stress (see 6.26.3.1)

Flaw Severity Class	Weld Thickness[a] (mm [in]) and Search Unit Angle										
	8 [5/16] through 20 [3/4]	>20 [3/4] through 38 [1-1/2]	>38 [1-1/2] through 60 [2-1/2]			>60 [2-1/2] through 100 [4]			>100 [4] through 200 [8]		
	70°	70°	70°	60°	45°	70°	60°	45°	70°	60°	45°
Class A	+10 and lower	+8 and lower	+4 and lower	+7 and lower	+9 and lower	+1 and lower	+4 and lower	+6 and lower	–2 and lower	+1 and lower	+3 and lower
Class B	+11	+9	+5 +6	+8 +9	+10 +11	+2 +3	+5 +6	+7 +8	–1 0	+2 +3	+4 +5
Class C	+12	+10	+7 +8	+10 +11	+12 +13	+4 +5	+7 +8	+9 +10	+1 +2	+4 +5	+6 +7
Class D	+13 and up	+11 and up	+9 and up	+12 and up	+14 and up	+6 and up	+9 and up	+11 and up	+3 and up	+6 and up	+8 and up

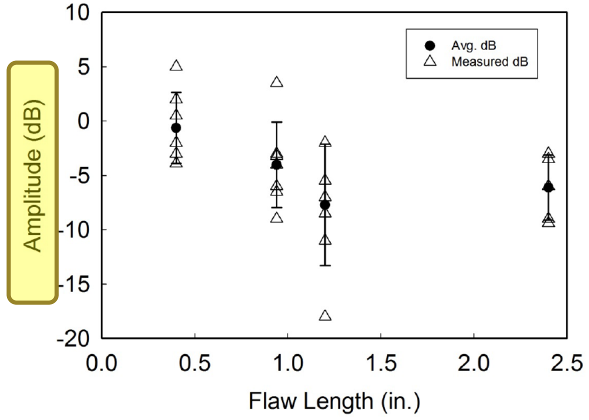


Figure 7. Reported amplitudes for rejectable flaws.

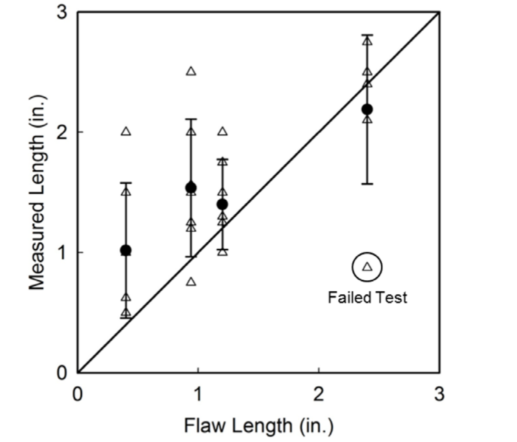


Figure 6. Flaw length measurement data from the UT performance tests.

Class A (large flaws)
Any indication in this category shall be rejected (regardless of length).

Class B (medium flaws)
Any indication in this category having a length greater than 20 mm [3/4 in] shall be rejected.

Class C (small flaws)
Any indication in this category having a length greater than 50 mm [2 in] in the middle half or 20 mm [3/4 in] length in the top or bottom quarter of the weld thickness shall be rejected.

Class D (minor flaws)
Any indication in this category shall be accepted regardless of length or location in the weld.

Flaw Characterization (NCHRP 908)

- PAUT techs were unreliable when characterizing flaw type (same with conv. UT)
  - Less than 50% of cracks were reported correctly
  - Cracks and LOF sometimes reported as volumetric

Flaw Characterization

Actual Flaw Type	Reported Flaw Type			
	Crack	Planar (Non-Crack)	Volumetric	No Type Reported
Crack	22%	44%	11%	22%
LOF	21%	71%	4%	4%
Porosity	25%	25%	50%	0%
Slag	9%	64%	27%	0%
False Calls	71%	0%	29%	0%

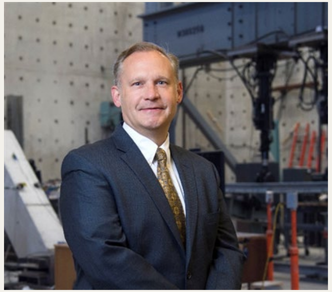
Does the surface condition of the steel need to be consistent with that required by AASHTO/AWS D1.5:2015 Clause 6.19.3?

In other words, does the in-situ coating need be removed prior to NDT?

- Yes
  - The presence of a coating is well known to potentially result in highly unreliable/variable UT data, especially when amplitude-based rejection criteria (i.e., AWS D1.5 criteria) are used
  - *Very specific and detailed calibration procedures are required if inspection is done through paint!!!!!*
    - *Good paint vs bad, well-adhered vs unknown adhesion etc.*

Discussion

Today’s presenters



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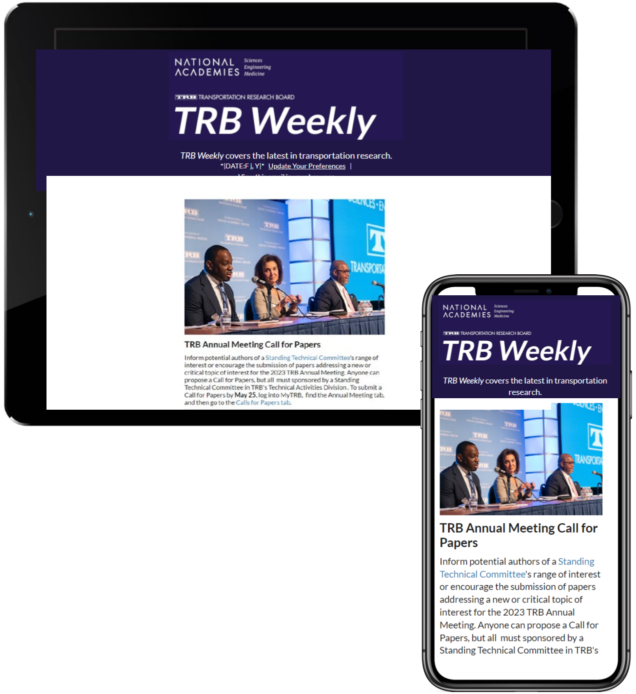

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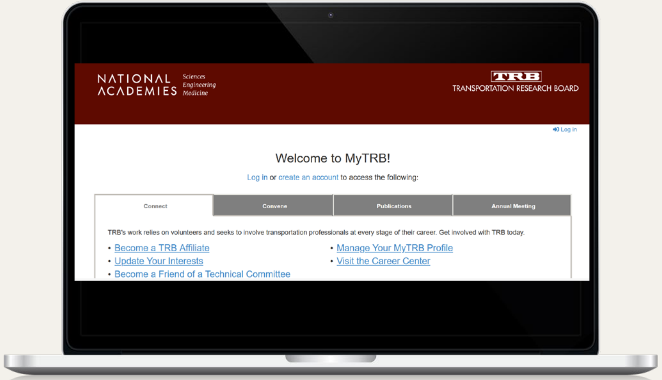

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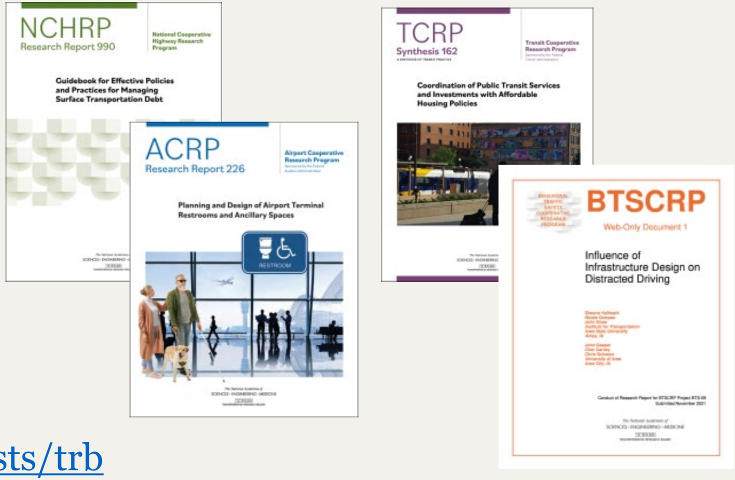

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