Testing of a 1912-Vintage Steel Girder Span Under Heavy Axle Loads at the Facility for Accelerated Service Testing: Initial Results

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AREMA 2011 Annual Conference and Exposition
Minneapolis, Minnesota

ABSTRACT

The North American railway network has thousands of riveted steel deck plate girder spans in service, many of them nearly 100 years old. In 2009, a 1912 riveted steel girder span donated by Norfolk Southern was installed in the Facility for Accelerated Service Testing (FAST) loop near Pueblo, Colorado, to evaluate performance under heavy axle load (HAL) traffic and to develop recommended practices for life extension and cost-effective repair procedures.

After 18 months and 212 million gross tons of HAL traffic, results indicate corrosion (which is a concern for older steel bridges) is the most significant issue to overcome in order to extend the life of the span and to accommodate HAL traffic. Simple bolted splice repairs made on the span have performed well so far. Repair of the lateral bracing system helped limit lateral deflections of the girder under HAL traffic. Deflections are well within American Railway Engineering and Maintenance-of-Way (AREMA) recommendations. Comparisons are made to theoretical calculations, measurements made in revenue service, and to measurements of the previous welded girder span at FAST. Theoretical models predict higher stresses than measured, but lower deflections than measured. Additional testing on this vintage span includes performance of a
movable bridge rail joint, bridge deck tie fasteners, and alternative ties for open deck bridges. Brief updates are provided on these tests as well.

**Number of Words: 220 abstract; 3,600 + 4,500 (18 figures)**
INTRODUCTION

Transportation Technology Center, Inc. (TTCI) is investigating efforts to safely extend the service life of older railroad bridges. Norfolk Southern (NS) donated a 1912 vintage riveted steel bridge span to TTCI, which was installed in the steel bridge at the Facility for Accelerated Service Testing (FAST) in December 2009 for testing under heavy axle load (HAL) traffic. The span was originally constructed for the Wabash Railroad and was in service over Wildcat Creek in Lafayette, Indiana. The 55-foot-5 inch riveted girder span replaced the original 55-foot-6 inch welded girder from Conrail in the east span of the FAST steel bridge.

Thousands of riveted steel girder railway bridges are still in service in North America. Many are approximately a century old. While they have performed well thus far, there is concern about their sustained performance under HAL traffic.

The 1912 vintage span is being observed and tested in the controlled environment under HAL traffic at FAST. Performance observations and test information will provide a good insight on where and when older steel bridges in revenue service will require maintenance.

Prior to installation at FAST, TTCI crews instrumented this bridge span in revenue service on the NS. Comparisons between the vintage riveted span performance at FAST and in revenue service, as well as between the vintage span and the previous welded steel span are made based on measurements to date. Comparisons are also made to values calculated based on theoretical models.

The vintage span is being monitored for performance under HAL traffic including deterioration and maintenance requirements for bridge components. The vintage span will be used to develop and evaluate effective life extension, retrofit, and repair strategies for similar bridges.
deterioration is found through frequent inspections, repairs are made as necessary. Tests are performed on the bridge to compare span performance before and after repairs are made.

VINTAGE SPAN DESCRIPTION AND RATINGS

The 55-foot 5-inch vintage span is a typical example of a riveted steel deck plate girder span, similar to many of the steel bridges still in revenue service today. Figure 1 shows the steel span in revenue service.

![Figure 1. Vintage Riveted Steel Girder Span in Revenue Service on NS](image)

When installed in revenue service, the vintage riveted span had concrete floor pans and a ballasted deck, as Figure 2 shows. The live load with that deck was rated at Cooper’s E-42, following AREMA Chapter 15 recommended practice. (1) When the span was installed at FAST, it was installed as an open-deck span, with ties directly supported by the steel girders. With the open deck, the live load rating increased almost 50 percent, to Cooper’s E-61 including a 10 percent reduction for corrosion. For comparison, the previous welded steel span from Conrail rates at Cooper’s E-89. The loading of the HAL train is equivalent to Cooper’s E-62. Figure 3 compares the ratings with the HAL train loading.
TRAFFIC HISTORY

Prior to installation at FAST, the vintage riveted span carried very little HAL traffic as compared to the HAL traffic at FAST. Annual traffic for recent years of service on the NS was about 50 to 55 million gross tons (MGT).

In one and a half years at FAST, the span has been subjected to 212 MGT of traffic under the HAL train, which is made up of primarily 315,000-pound gross rail load cars. Note that the HAL train at FAST typically runs at 40 mph and does not include cars that generate significant wheel impact loads.
Approximately 75 percent of the traffic at FAST is heavier than any traffic measured on the vintage riveted span when it was in revenue service. Figure 4 displays the axle loads at FAST compared to revenue service. Figure 4 also shows the HAL load the 55-foot 6-inch welded steel span was carrying before it was removed from FAST.

![Figure 4. Comparison of Axle Load Distributions for Vintage Span in Revenue Service and at FAST](image)

PRE-INSTALLATION REPAIRS

Before the vintage riveted span could be installed in the steel bridge at FAST, an initial inspection was made. Several repairs were needed before installing the span. Figure 5 shows the areas where initial repairs were made to the top lateral bracing system at Locations A, B, C, D, and E.

![Figure 5. Vintage Span Repair Areas](image)
At Location A, the horizontal cross members were rusted all the way through. To correct this, the angles were replaced with the same size angle. Bolted connections were used instead of rivets. Figures 6a and 6b show the rusted horizontal member.

Rusted gusset plates at Locations B and C were replaced. Lateral bracing members on both ends of the bridge had broken off. The lateral braces were left off until baseline data was collected.

At Location D, the gusset plate and rivets were rusted through. Also, the rusted lateral brace was unattached. The gusset plate was replaced and prepared for a bolted connection for a brace after baseline data was collected.

At Location E, the lateral bracing member was missing. The gusset plates were prepared for a new member to be installed after baseline data was collected.

IN-SERVICE CRACKS AND REPAIRS

After 72 MGT of traffic at FAST, a broken bracing member was found on the vintage span. The brace was a bottom lateral angle at mid-span. It broke at the gusset plate connection to the bottom flange of the south girder. The horizontal leg of the angle, which connected to the gusset plate, was rusted through most of its cross section. There appeared to be about a 1/2-inch crack growth region near the corner of the angle. The vertical leg was fractured resulting in a pull-apart.
gap of about 3/16-inch. Horizontal displacement was also about 3/16-inch. Vertical displacement was about 1 inch, which was primarily due to the dead weight of the member (Figure 7a).

The train ran one night (109 train passes) before the brace was repaired. No additional damage was noted. The repair made was a simple splice of the vertical leg of the angle (Figure 7b).

**BRACING ANGLE**

After 83 MGT of traffic at FAST, another broken bracing member on the FAST vintage span was found. The brace was a top cross frame member at the east intermediate diaphragm. It broke at the gusset plate connection to the top flange of the south girder. Figure 8a shows the horizontal leg of the angle, which connected to the gusset plate, was rusted through for most of its length. The train ran one night (127 train passes) before the brace was repaired (Figure 8b). No additional damage was noted. The repair was once again a simple splice of the vertical leg of the angle.
After approximately 100 MGT, a splice repair was made to a top lateral brace. The horizontal leg of the brace was mostly corroded, with a crack beginning from the corrosion. The crack propagated about 2 inches into the vertical leg before repair, leaving only about 1 inch of the cross section remaining. Figure 9 shows the repair to the vertical leg.

LATERAL BRACING TEST

In addition to frequent inspections and subsequent repairs, TTCI conducted lateral bracing tests before and after initial repairs were made to the vintage span. Previous work regarding lateral bracing and lateral forces on bridges has been performed by Canadian National (CN) and at FAST (1,2,3). Two reference frames were fabricated and installed near mid-span outside of both
the north and south girders. String potentiometers were attached to the top flange as well as the bottom flange to monitor the lateral deflections of the girders during dynamic testing. String potentiometers located on the bottom of each girder at mid-span were used to measure vertical deflections. These lateral tests were performed before repairs were done and again after repairs were installed. Test runs were made using different train speeds from 5 to 45 mph. This allowed comparisons to be made in measured lateral deflections.

Figures 10a and 10b show the top flange lateral deflections of the north and south girders before and after repairs to the lateral bracing. As expected, the amount of deflection increased with increase in train speed. Note that the bracing limits the amount of lateral deflection. Without the bracing repairs, the maximum lateral deflections were about 50 percent higher under the HAL train.

Both the north and south girder measurements for lateral deflection are within the AREMA recommended value of 3/8 inch for a span of this length (4). Repair of the lateral bracing system helped limit lateral deflections of the girder under HAL traffic.

Even after the lateral bracing was repaired, the measured lateral deflections are higher than those measured on three CN steel bridges (J). The FAST train has heavier cars, and the post-repair
data was collected under the entire FAST train of more than 100 cars. The CN data was collected at higher train speeds with a short test train.

STRESS MEASUREMENTS

Figure 11 displays the vintage riveted span installed at FAST and the location of the instrumentation for measurement of girder strains and vertical displacements.

![Figure 11. Vintage Riveted Span at FAST](image)

Figure 12 compares the average peak stresses in the vintage riveted span measured at FAST and in revenue service. As expected, because of the higher axle loads at FAST, the corresponding stress is higher. The typical peak live load stresses experienced at FAST are about 80 to 90 percent greater than those in revenue service.

![Figure 12. Comparison of Average Peak Live Load Stresses for Vintage Riveted Span](image)
Theoretical stress values were calculated using conventional bending beam models. Because the FAST train had no flat wheels, there were no rail joints on the span, and the bridge approaches were smooth, no impact was included in the theoretical calculations. Figure 13 compares the measured stresses in the vintage riveted span and the previous welded steel span to theoretical values.

![Figure 13. Comparison of Typical Peak Stresses of Vintage Riveted Span and Welded Steel Span at FAST](image)

The measured stress values are about 7 to 15 percent less than the theoretical calculated stress values for both the vintage riveted span and the previous welded steel span at FAST. Similar results have been reported by Sweeney et al. (5). Possible contributing factors include partial fixity of the bridge bearings and distribution of wheel loads by the rails.

Information provided by the NS and Conrail indicates that the yield strength of the steel in the vintage span is 30 ksi and in the welded span is 36 ksi. So while the live load stresses in each span at FAST are similar, they are 20 percent higher in the vintage riveted span as a portion of the yield strength.

MID-SPAN DEFLECTION MEASUREMENTS

Comparison of the mid-span live load deflections at FAST to revenue service indicates the span had smaller deflections in revenue service than at FAST, as expected. This is due primarily to the
heavier axle loads operated at FAST. It might also be due in part to additional span stiffness from the concrete floor pans supporting the ballasted deck. In revenue service, the span experienced many lighter weight and empty train cars. Figure 14 displays the typical measured peak deflection and calculated deflection for the vintage riveted span. The mid-span deflection of the vintage span at FAST is approximately 50 percent greater than the deflection of the span in revenue service.

![Figure 14. Comparison of Mid-Span Live Load Deflections for Vintage Riveted Span](image)

Theoretical values were calculated for the vintage riveted span and the previous welded steel span at FAST. A theoretical value was not calculated for the span in revenue service. Figure 15 shows the typical peak deflection measured for the vintage span at FAST is less than that for the previous welded steel span. This result is predicted in the theoretical models as well. The primary reason is that the vintage riveted span is 15-percent deeper than the previous welded span.

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Figure 15. Comparison of Live Load Deflections of Vintage Riveted Span and Welded Steel Span at FAST

Note that the theoretical deflection values are less than the measured values. There are several possible contributing factors that might explain this. First, the theoretical deflection model considers only beam bending. It does not include shear deformations. Nor does the theoretical model at this point consider displacements in the bearings, bearing pads, or foundations.

OPEN DECK CONVERSION EFFECTS

When the vintage span was in service on the NS, five strain gages were attached to each girder near mid-span. These gages were placed on the bottom angle, near ¼ height, near ½ height, near ¾ height, and on the top angle. Figure 16 shows the results from these strain gages from the bottom of the girder to the top. Note that the neutral axis (location of zero bending stress) is lower in the span at FAST than it was in revenue service. This is most likely due to the composite action of the concrete floor pans when the span was in revenue service. With the concrete floor pans removed for service at FAST, the resulting neutral axis shifted lower. Because more of the steel is in compression without the floor pans, bracing becomes more important to provide stability to the span. The presence of the concrete floor pans in revenue service provided redundancy in the case of corroded or missing bracing. (The north girder when this span was installed on the NS is the same girder that is on the north side when it was installed at FAST.)
OTHER TESTS

Figure 17 shows a moveable bridge joint that was installed for about 100 MGT over the center pier of the FAST steel bridge (on one end of the vintage span). The joint was installed as if the 65-foot welded span was the fixed span and the 55-foot 5-inch vintage span was the movable span. The increased impact loading from the joint resulted in no noticeable changes or degradation in the vintage span. However, the adjacent 65-foot welded steel span experienced a significant increase in fatigue crack growth during this time. All the fatigue crack issues were associated with welded details in the 65-foot span. The riveted construction of the vintage span might be better suited to the high-impact environment beneath a bridge joint. Perhaps the configuration of the joint, with all of the rail surface discontinuities on the fixed span, resulted in greater impacts being imparted to the fixed span (65-foot welded span) rather than the movable span (55-foot 5-inch vintage riveted span).
The vintage riveted span has girders spaced on 8-foot centers. This spacing is very typical and is well suited to testing of ties for open deck bridges. (The previous Conrail welded span in this location has 6-foot 6-inch girder spacing. The ties were subjected primarily to bearing rather than bending.) TTCI has installed several different open-deck bridge ties to compare performance and to evaluate ties from various alternative materials and designs. The ties on this span include three types of traditional timber ties (white oak, Douglas fir, southern yellow pine) as well as two alternative ties (glulam fir and fiber-reinforced foamed urethane (FFU), a fiberglass-like composite). Figure 18 shows the FFU ties.
FUTURE WORK

The vintage riveted span is still being observed and tested. Corroded top lateral braces and gusset plates, as well as a couple of bottom lateral members with noticeable bowing, are a concern. The span provides the opportunity to test a variety of different repair techniques, including some new methods that might be accomplished without taking the span out of service or without requiring track time on the bridge, which, if successful, may be applied to similar steel spans in revenue service.

SUMMARY AND CONCLUSIONS

- The 1912-vintage riveted steel span at FAST continues to perform satisfactorily after 212 MGT of HAL traffic. Thus far only minor repairs have been required.

- Corrosion of gusset plates and connecting bracing are a concern for older steel bridges.

- Most of the corrosion is in or near horizontal gusset plates, where moisture did not drain quickly. The vertical gusset plates show comparatively little corrosion.
- The simple bolted splice repairs made on the span have performed well so far.

- The HAL environment to which the vintage riveted span is subjected at FAST is considerably more severe than the axle load environment this span experienced in revenue service. Likewise, the stresses and deflections in the vintage span at FAST are considerably greater than those measured in revenue service.

- Measured stresses in the vintage riveted span are similar to the measured stresses in the previous welded steel span at FAST; however, the stresses in the vintage span are 20 percent higher as a fraction of the steel strength.

- Measured deflections in the vintage riveted span are somewhat less than those measured in the previous welded steel span; however, this is expected, because the vintage span is 15 percent deeper than the welded span. Deflections are well within AREMA recommendations.

- For both the vintage riveted span and the welded steel span, theoretical models predict higher stresses than measured, but lower deflections than measured.

- Conversion of the span from a ballasted deck in revenue service to an open deck at FAST resulted in a significant increase in live load capacity, but increased the importance of bracing members.

- Installation of bracing repairs limited the maximum lateral deflection of the bridge span.

ACKNOWLEDGEMENTS

The authors are grateful to the NS for the donation of the vintage riveted span and assistance with field testing. In particular, this work would not have been possible without the support of
James N. Carter, Jr., Chief Engineer – Bridges and Structures and Howard C. Swanson, Assistant Chief Engineer – Bridges.

The movable bridge rail joint was donated by Progress Rail Services, with support from Russell R. Hein – Technical Director.

Bridge deck ties were donated by NS, Canadian Pacific Railway, BNSF Railway, Union Pacific Railroad, and Sumitomo Corporation of America.

This study was funded through the Association of American Railroads’ Strategic Research Initiatives Program and the FAST Program.

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Testing a Vintage Steel Span at FAST

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Overview of Presentation
- Introduction and background
- Span description, rating, and history
- Repairs before and after installation
- Stresses and deflections
- Bridge joint test
- Open deck tie tests
- Summary and conclusions
Background

- 1912-steel DPG span
- Wabash RR, now NS
- Lafayette, Indiana
- 55 ft 5 in riveted span
- Thousands of similar DPGs in service
- Span removed by NS in October 2009, installed at FAST December 2009

How will these old spans perform under heavy axle load (HAL) traffic?
**Span Ratings**

- Ballast deck on NS Rated E-42
- Concrete floor pans
- Open deck at FAST Rated E-61
- FAST Train ~ E-62
- Operating near calculated span capacity
- Previous Conrail span at FAST rated E-89
Traffic History

- NS annual tonnage 50-55 MGT
- At FAST – 212 MGT in 18 months
- NS traffic – Triple Crown trains, automotive, mixed freight, grain
- FAST traffic – 315,000 lb cars
Pre-Installation Repairs
- Several corroded bracing members and gussets
- Concrete floor served as lateral bracing on NS
- For open deck use, some repairs needed
- Repairs made only to members completely broken or missing
Vintage Steel Span Testing at FAST

**In-Service Repairs**

- 72 MGT: Broken bottom lateral angle
- Horizontal leg corroded at gusset plate
- Bolted splice repair of vertical leg
**In-Service Repairs**

- 83 MGT: Broken top cross frame angle
- Horizontal leg corroded at gusset plate
- Bolted splice repair of vertical leg
In-Service Repairs

- 100 MGT: Broken top lateral angle
- Horizontal leg corroded at gusset plate
- Bolted splice repair of vertical leg
**In-Service Degradation**

- 180 MGT: Broken top horizontal gusset plate
- Significant corrosion near top flange connection
- Much of crack hidden between tie and angle
In-Service Degradation

- 201 MGT: Broken top end cross frame angles
- Significant corrosion near gusset connection
- Horizontal surfaces have most corrosion (drainage, drainage, ...)

VINTAGE STEEL SPAN TESTING AT FAST
Vintage Steel Span Testing at FAST

Lateral Displacements – Effects of Bracing

- Before and after bracing repairs completed
- Mid span displacements top and bottom flanges
- Bracing limits maximum displacements
- All displacements less than AREMA recommended max values
Vintage Steel Span Testing at FAST

Stress Measurements – On NS and FAST
- Span instrumented during service on NS
- Strain gages near mid span
- Average peak live load stresses shown
- Higher stresses at FAST as expected
Vintage Steel Span Testing at FAST

Stress Comparisons at FAST
- Comparison to previous Conrail welded span
- Comparison to theoretical calculations
- Similar span stresses, but 30 ksi versus 36 ksi steel
- Lower stresses than calculated
Vintage Steel Span Testing at FAST

**Vertical Deflections – On NS and FAST**

- Displacements at mid span
- Average peak live load displacements shown
- Higher displacements at FAST as expected

![Graph showing comparison of live load deflections between FAST and Revenue Service.]

**Graph Details:**
- **Vertical Axis:** Live Load Deflection (inches)
- **Horizontal Axis:**
  - **FAST:** 0.40
  - **Revenue Service:** 0.26
Vintage Steel Span Testing at FAST

Deflection Comparisons at FAST
- Comparison to previous Conrail welded span
- Comparison to theoretical calculations
- Vintage has less deflection (75-inch versus 63-inch depth)
- Greater displacements than calculated
**Vintage Steel Span Testing at FAST**

**Ballast Deck versus Open Deck Comparison**
- Intermediate strain gages to locate neutral axis
- Strain readings indicate n.a. is lower for open deck at FAST
- Concrete floor pans in revenue service likely acted with composite action in compression
Movable Bridge Rail Joint

- Installed as if vintage span was lifting span
- Progress Rail rider-style joint
- Crane rail used for longer rider section
- 106 MGT life at FAST
- Base plate weld cracks
- Generated local impacts
- Increased maintenance
  - Hook bolts
  - Walkway & handrails
  - Cracks in welded span
Vintage Steel Span Testing at FAST

Alternative Ties for Open Deck Bridges

- Vintage Span has 8-foot girder centers
- Alternative Bridge Ties in test include:
  - Glued-Laminated Douglas Fir (Union Pacific RR)
  - FFU Fiberglass (Sekisui/Sumitomo) – shown
- Timber ties include:
  - White Oak (NS)
  - Southern Yellow Pine (BNSF)
  - Douglas Fir (CP)
- Lab tests also conducted
- AREMA 30 Letter Ballot
Summary and Conclusions

- Span performing satisfactorily with 212 MGT of FAST HAL traffic
- Repairs to bracing have been required
- Bolted splice repairs performing well to date
- All broken members to date have had significant corrosion
- Corrosion is worse in horizontal members
- Stresses and deflections compare reasonably to previous span and theoretical calculations
- Tests are ongoing
Acknowledgments

Thanks to:

- Norfolk Southern (Jim Carter, Howard Swanson) – donation of span, deck ties & hardware
- Progress Rail Services (Russ Hein) – donation of movable bridge rail joint
- BNSF, CP, UP, Sekisui/Sumitomo – deck ties
- AAR – research sponsor
- AAR Bridge Technical Advisory Group (John Unsworth, chairman)