LOAD TESTING OF A STEEL THRU-GIRDER RAILROAD BRIDGE WITH BALLASTED TROUGH DECK

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ABSTRACT

A steel thru-girder bridge with ballasted steel trough deck and consisting of three simple spans was tested under static and dynamic train loads to ascertain stress distribution, particularly on the trough deck. Visible deflection of the deck under live load had been noted during an inspection of the bridge and a subsequent load rating analysis was performed. Two methods of live load distribution in the AREMA Manual for Railway Engineering were interpreted as applicable to a trough deck. It could be assumed that such a deck could behave either as a transverse beam per Article 15-1.3.4.2.3 with train axles applied as concentrated loads or as a regular ballasted deck per Article 15-1.3.4.2.2 with uniform load application of the train axles. Accordingly, two different load ratings and deflections were calculated, and based on ballasted deck behavior the load rating was lower than expected. With an aging bridge and considering the observed deflection, load testing was recommended to verify the safe passage of site-specific locomotive loads, and
to ensure that those loads do not overstress the deck and to determine actual load distribution on the deck.

The bridge was subjected to static and dynamic loads from train engines and resulting stains and deflections in the trough deck and the thru-girders were measured. Actual live load distribution was compared to the calculated distributions based on the transverse beam and deck theories, and the measured impact load was compared to the impact load as recommended in the AREMA Manual. A finite-element model of the bridge was analyzed, to explain the measured behavior. The article-presentation includes details and results of the live load testing, comparison of actual live load distribution to AREMA recommended distributions and suggested recommendations for load evaluation of a trough deck.

Key Words: Load Testing, Load Rating, Design Specifications, Steel Thru-Girders, Ballasted Deck, Trough Deck, Railroad Bridge.

PROJECT BACKGROUND

The subject bridge is located at Mile Post 61.36 along Metro-North Commuter Railroad (MNCR) Harlem Line and spans over the Croton River near Pawling, NY. Built in 1902, the bridge consists of three equal simple spans that are supported on concrete and masonry piers and abutments. Each span consists of two thru-girders and ballasted transverse troughs, all built from riveted steel plates and angles; span length is 19.61 feet between girder supports. Locomotive-powered passenger trains pass on this single track bridge. Various views of the bridge are shown in Figures 1a, 1b, 1c and 1d.
An inspection of the bridge (after 100 years of service) has shown visible deflection of the trough deck under live load, as well as corrosion in the bottom plates of the trough deck near drain pipes and in girder webs. While performing load rating calculations for the trough deck, two methods of axle load distribution in the AREMA Manual for Railway Engineering (1) were interpreted as being applicable to this deck system. The two methods are (i) beam distribution in the form of two equal and concentrated rail loads as per Article 15-1.3.4.2.3 and (ii) deck distribution in the form of uniform load as per Article 15-1.3.4.2.2. Two different load ratings and corresponding deflections were calculated from the two methods. The deck load rating based on transverse beam distribution was significantly higher than that based on deck distribution. If uniform distribution of load was assumed, the overall rating of the bridge would be controlled by the trough deck, otherwise girder rating would control. While girder rating was reported as the controlling rating, load testing of the bridge was recommended to ascertain actual distribution of site-specific live loads, particularly on the trough deck. Subsequently, the main focus of the testing was to verify that MNCR locomotive loads do not overstress the trough deck and can pass safely on the subject bridge.

**DISTRIBUTION OF LIVE LOAD ON THE BALLASTED DECK**

Based on AREMA Article 15-1.3.4.2.2, axle load is distributed uniformly on the deck. The longitudinal length of load distribution is 3 feet plus the minimum distance from bottom of tie to top of beam (or trough top plate in this case), but not to exceed 5 feet nor minimum axle spacing (5 feet for Cooper E80 load). The lateral or transverse distribution length is the length of tie plus the minimum distance from bottom of tie to top of beam. The
thickness of a steel deck is recommended to be a minimum of 1/2 inch, based on this article.

Figure 2a shows schematically the uniform distribution of live load on the trough deck of the subject bridge along the transverse direction. The thickness of trough plates is 7/16 inch for top and bottom plates and 3/8 inch for web plates. Even though both values are smaller than the recommended thickness of 1/2 inch, it was assumed that the recommended thickness is intended for the design of a new steel deck (for durability) and, therefore, uniform distribution of live load was applicable to the trough deck.

Based on AREMA Article 15-1.3.4.2.3, an effective axle load on a beam is calculated from the following equation:

\[
P = \frac{1.15AD}{S}
\]

\(D = d\) (no concrete deck)

where,

- \(P\) = load on beam from one track (effective axle load on a beam), kips
- \(A\) = axle load (maximum axle load), kips
- \(S\) = axle spacing (minimum axle spacing), feet
- \(D\) = effective beam spacing, feet
- \(d\) = beam spacing, feet

The calculated load \(P\) is divided into two equal concentrated loads \(P/2\) that are applied on each beam at each rail, without lateral distribution of the two loads. Figure 2b shows
schematically the distribution of axle load onto a transverse trough, hereby referred to as “beam distribution” in contrast to “deck distribution” in Figure 2a.

**As-Inspected Load Rating**

As-inspected load rating of the trough deck was calculated, based on each of the two methods of live load distribution and considering section losses. For both methods, it was assumed that the distributed axle load is resisted by a single trough section consisting of a bottom plate, two web plates and two top half-plates, because of small and varying vertical distance (±5 inches) between bottom of wood tie and top steel plate. Figure 3 shows a typical longitudinal section of the ballasted trough deck with dimensions measured at Span 1. For the deck distribution method, the longitudinal distribution length was limited by the width of a single trough section (2.73 feet), instead of the required length (3 feet + 5 inches). For both methods, the distributed axle load was applied on an assumed transverse beam, simply-supported by the girders and with a span length equal to girder spacing (14 feet). Later when evaluating load test data, the assumed beam span length was reduced (in manual calculations and preliminary finite element analysis), in order to represent more accurately existing conditions and measured structural behavior.

The deck distribution method resulted in 38 percent lower load rating as compared to the beam distribution method. The as-inspected deck rating based on uniform distribution of axle load was E 40 for normal load level and E 51 for maximum load level. However, based on beam distribution of axle load, the as-inspected deck rating was E 65 for normal load level and E 82 for maximum load level. Maximum moment was the controlling load effect for all deck ratings. It is worthy to note that, if the longitudinal distribution length
was assumed to be as recommended by the deck distribution method (3.42 feet for the subject deck), the deck rating based on that method would have increased to E 49 for normal load and E 63 for maximum load.

Considering the conservative assumptions of (i) limited longitudinal distribution length and (ii) a single trough section resisting the axle load, the beam distribution method was assumed to be more realistic for the subject deck and the load rating based on that method was reported to MNCR as the as-inspected deck rating.

As for the girders, the as-inspected rating was E 54 at normal load level and E 71 at maximum load level, which was reported as the controlling rating for the entire bridge. Maximum moment was the controlling load effect for all girder ratings. The yield strength of steel material was assumed to be 33 ksi for all the ratings, considering the year of construction of the bridge and as recommended in the AREMA Manual.

**LOAD TESTING PLAN AND PROCEDURES**

The primary objective of the load testing was to verify that the trough deck is not overstressed by MNCR locomotive loads and that the bridge can safely carry such loads. The secondary objective was to determine the structural response of the deck to static and moving loads and the actual distribution of axle loads to the deck.

Steel samples were removed by MNCR from the trough deck and later tested at an independent laboratory to determine material properties, such as modulus of elasticity for later use in the evaluation of field-measured displacements and strains, and yield strength for comparison with the value assumed in the load ratings.
Span 1, the southern span of the bridge, was selected as the most suitable of the three similar spans for conducting field instrumentation and load testing. With water depths of 3 to 5 feet, acceptable clearance and easy access to the shore, all necessary work could be performed at that span in a timely and practical manner.

MNCR provided (i) a temporary platform for placing testing equipment, (ii) flagman for safety and communication with train conductor and (iii) test locomotives. Figure 4a shows the temporary platform after installation at Span 1. Testing equipment, field instrumentation and monitoring were provided by Rutgers University (Rutgers) as subcontractor to Chas. H. Sells, Inc. (Sells), the design consultant to MNCR. Analysis and evaluation of all data were performed by Sells.

Testing equipment consisted of a portable data acquisition system, a portable computer, deflection transducers, strain gages and a power generator. The Structural Testing System (STS) was used for acquisition of deflection and strain data, which would be transferred to a computer. Figure 4b shows the STS equipment and the portable computer. Linear Variable Differential Transducers (LVDT) were installed at the bottom of the bridge deck and girders for measuring vertical displacements, as shown in Figure 4c. Demountable Strain Gage Transducers (DSGT) were installed at the bottom of the deck to measure transverse strains and at the bottom of the girder flange to measure longitudinal strains. Threaded nails were used to mount strain gages to the bottom and top plates of the trough deck as shown in Figure 4d. Because of limited clearance between trough webs, epoxy was used instead of threaded nails to adhere some strain gages to trough web plates. Strain gages at girder bottom flange were mounted with clamps as shown in Figure 4e.
Seven displacement transducers and twenty one strain gages were installed within the middle region of Span 1. The locations of the displacement transducers are shown in Figure 5a. Vertical displacements were measured at the bottom plates of three consecutive troughs and at the bottom flange of each of the girders. Figure 5b shows the locations of strain gages. Transverse strains were measured at the bottom, top and web plates of three consecutive troughs and longitudinal strains were measured at the bottom flange of each girder.

Two types of locomotives, Genesis and FL-9, were used during static testing (zero speed) and dynamic testing with speeds of 10 and 70 miles/hour (mph). The Genesis locomotive has four axles - two 69.5 kips axles spaced at 9 feet and two 66 kips axles also spaced at 9 feet, with 34.6 feet spacing between the interior axles. Figures 6a and 6b show the actual positions of the Genesis locomotive during static testing. The FL-9 locomotive has four 57 kips axles, with spacing of 9 feet between the front two axles, 6.79 feet between the back two axles and 22.7 feet between the interior axles. Figures 6c and 6d show the actual positions of the FL-9 locomotive during static testing.

The plan for the static testing was to apply a single axle load at the centerline of the fourth trough (or fourth bottom plate) from the south abutment, directly above the displacement and strain sensors at that location (sensors 8, 10 and 1 as shown in Figures 5a and 5b). The actual positions of the axles were slightly off the centerline of the fourth trough (6 to 14 inches), because small movements of the diesel locomotives used in the testing were difficult to achieve and there was limited time (less than 5 minutes) for completing each
test before the required continuation of regular traffic on the bridge. Regardless, the exact locations of the axle loads were accounted for in the analysis.

Deformations from the static tests would be compared with deformations from the dynamic tests in order to determine the extent of dynamic amplification of the static axle loads and thus the influence of locomotive speed on the subject bridge, and particularly its trough deck. The procedure for a dynamic test included the following steps. First, the data acquisition system was initialized five minutes before the scheduled arrival time of a locomotive. Second, the assigned time duration of each test was one minute, which was more than sufficient for the locomotive to pass the entire length of the bridge at the lower speed. Third, the data acquisition system was triggered manually 15 seconds before the scheduled arrival time. Fourth, the data was measured and stored automatically in the portable computer.

LOAD TESTING RESULTS

Only some of the essential results are presented, given the scope of this paper. Variation of static deflection of the trough deck, at its mid-span, with longitudinal distance is shown in Figure 7. Specifically, the deflections at locations DT 6, 10 and 9 for all four static load cases are shown in this figure. First, the deflections from each the two Genesis locomotives are larger than the deflections from each of the two FL-9 locomotives, understandably because of larger axle loads. Second, the fact that locations DT 6 and 9 each have a relatively significant deflection as compared to the deflection at location DT 10 indicates that more than a single trough section is resisting the axle load. Third, the measured deck
deflection at each location should be reduced by the average girder deflection at corresponding transverse locations in order to calculate a net deflection of the deck, as demonstrated in Figure 8.

Variation of static deflection of the bridge along a transverse direction is shown in Figure 8. Specifically, deck deflections at locations DT 1, 10 and 8 and girder deflections at locations DT 7 and 2 are shown. First, the Genesis locomotives (2996 Cases B and C) produce higher deflections than the FL-9 locomotives (2900 Cases B and C), because of higher axle loads. Second, for Genesis locomotive Case B and location DT 10, the measured deck deflection is about 0.205 inches but the net deflection is 0.105 inches, which is calculated by subtracting the average of the deflections at DT 7 (0.08 inches) and at DT 2 (0.12 inches) from the deflection at DT 10. Third, girder deflections at DT 2 are slightly larger than those at DT 7, most likely because the centerline between the two rails is slightly off by two inches from the bridge centerline and the girder where DT 2 was mounted had corrosion at mid-span and therefore reduced flexural stiffness.

Figure 9 shows dynamic deflections of the deck at sensor DT 10, for the Genesis locomotive load passing over the bridge at speeds of 10 mph and 70 mph. The maximum or peak deflection for each speed case occurs when the axle load is directly above the sensor location. The maximum deflection for the lower speed is 0.214 inches, which is approximately equal to the maximum deflection of 0.227 inches for the higher speed, and the same is true for the net deflections (0.111 versus 0.102 inches). This means that dynamic amplification of static axle loads or impact load is lower than expected for the subject deck. A similar pattern of low dynamic amplification is shown in Figure 10 for
dynamic deflections of the girder at sensor DT 7 under FL-9 locomotive loading. The maximum deflection for the slow speed is 0.094 inches as compared to 0.102 inches for the high speed.

Longitudinal variation of the dynamic deflection of the deck under FL-9 locomotive loading is shown in Figure 11 for each of the two speeds, 10 and 70 mph. The deflection pattern is almost identical for the two speeds, indicating low dynamic amplification.

Figure 12 shows dynamic strains in the deck due to FL-9 locomotive loading, measured at locations SG 6 (South 6097), SG 10 (Middle 5103) and SG 9 (North 6159) along the centerline of the bridge. The strain pattern for the low speed is similar to that for the high speed, indicating once again minimal dynamic amplification or small impact load. The strain at SG 6 is about 65% of the strain at SG 10, which means that axle load is resisted by more than a single trough section.

Dynamic deflections of the deck at sensor SG 10, for Genesis locomotive loading at speeds of 10 mph and 70 mph, are shown in Figure 13. The maximum strain for the low speed case (328.7 $\mu\varepsilon$) is almost identical to the maximum strain for the higher speed (325.5 $\mu\varepsilon$), confirming previous observations on small impact load.

**TEST RESULTS VERSUS ANALYTICAL RESULTS**

Field measured strains were converted to experimental stresses, by multiplying the strains by the modulus of elasticity of the steel material. The average modulus of elasticity for three steel samples removed from the deck and tested in a laboratory was 29,300 ksi. The average yield strength for the same samples was 33.4 ksi, as compared to the assumed
value of 33 ksi used in the load rating calculations, and the average ultimate strength was 56.4 ksi.

Analytical stresses were calculated for each of the deck and beam distribution methods and assuming either one or two trough sections. Table 1 shows experimental and analytical static stresses in the deck at the mid-span of the center trough. Consider for example Genesis Load Case C. The experimental stress is 8.32 ksi as compared to 19.28 ksi for deck distribution with one trough, 10.14 ksi for deck distribution with two troughs, 6.29 ksi for beam distribution with one trough, and 6.84 ksi for beam distribution with two troughs. The shown analytical stresses for both distribution methods are calculated manually, assuming that the deck acts as a simply supported beam with a span length equal to girder spacing (14 feet). With the beam distribution method, the calculated stress for a two-trough section is approximately the same as that for a single-trough section, because beam spacing and effective load are each increased by a factor of two (Equation 1) while as-inspected section modulus is almost doubled (for two troughs). Without section losses, the calculated stress using the beam distribution method would be the same for one trough as for two troughs.

For all four load cases, the analytical stresses based on the deck distribution method with a single trough are more than twice the corresponding experimental stress. In fact the difference varies from 129 to 156 percent, implying that this approach is not accurate. For deck distribution with two troughs, the difference between analytical and experimental stresses varies from 20 to 35 percent. As for beam distribution with one trough, the difference between analytical and experimental stresses varies from -16 to -26 percent. But
if two troughs are used with the beam distribution method, the difference would vary from -9 to -19%, implying that this is the most accurate approach. Preliminary three-dimensional finite element analysis of the subject deck seems to validate the accuracy of the beam distribution method with two troughs resisting the load. However, it should be noted that using manual calculations with a two-trough section, the deck distribution method provides slightly conservative stresses which is appropriate.

Experimental and analytical dynamic stresses in the deck for Genesis loading are shown in Table 2. The first three load cases represent the passage of the front axle of the Genesis locomotive over the center trough of the deck, while the last three load cases are for the rear axle. The analytical stresses include 42% impact for the high-speed load cases based on AREMA Article 15-1.3.5 and about 14% for the low-speed cases based on Articles 15-1.3.5 and 15-7.3.3.3. First, the beam distribution method is more representative of actual dynamic stresses than the deck distribution method. But it could be argued that deck distribution with two troughs results in reasonably conservative stresses. Second, the speed of the locomotive, in this case, does not have a quantifiable effect in terms of load amplification. Third, comparison of experimental dynamic stresses in this table to experimental static stresses in Table 1 for the same axle shows that the maximum impact is 9.5%, occurring with the third load case. Based on this and other data, the measured maximum impact for the subject deck is about 10%. Possible reasons for the difference between measured and calculated impact are (i) smooth rail surfaces and (ii) the AREMA recommended impact represents a maximum design impact with 1% probability of being exceeded in 80 years.
Finally, all of the experimental stresses caused by either static or dynamic loading are much less than the allowable stress of 18.4 ksi, calculated based on AREMA Article 15-1.4.1 ($0.55F_y$) and the laboratory measured yield strength of 33.4 ksi. The ratio between actual stress and allowable stress varies from 0.35 to 0.51, while the ratio between actual stress and yield stress varies between 0.19 and 0.28. The implication is that MNCR locomotives used in the testing do not overstress the subject trough deck.

CONCLUSIONS

First, the subject trough deck is not overstressed by MNCR Genesis and FL-9 locomotive loads, and it is safe to operate MNCR Genesis and FL-9 powered trains on the subject bridge. Second, the measured impact load or dynamic amplification of live load (about 10%) is smaller than the calculated impact load (42%) per AREMA Article 15-1.3.5. One reason for this difference is that the AREMA recommendation is for a maximum design impact that is based on 1% probability of being exceeded in 80 years. Also, the measured impact load is most likely influenced by smooth rail surfaces. Third, the effective section of the trough deck in resisting axle load consists of two consecutive troughs, based on measured deflections and strains. Fourth, actual load distribution on the trough deck is more accurately represented by Article 15-1.3.4.2.3 (beam distribution) than Article 15-1.3.4.2.2 (deck distribution), particularly if a two-trough section is assumed. However, deck distribution with two troughs provides conservative representation of actual stresses.
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### Tables 1: Experimental and Analytical Static Stresses in the Trough Deck

<table>
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<tr>
<th>Load Case</th>
<th>Measured Stress ksi</th>
<th>Deck 1 Trough ksi</th>
<th>Deck 2 Troughs ksi</th>
<th>Beam 1 Trough ksi</th>
<th>Beam 2 Troughs ksi</th>
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<tr>
<td>Genesis Case C (FA)</td>
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<td>19.28</td>
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### Tables 2: Experimental and Analytical Dynamic Stresses in the Deck for Genesis Loads

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<th>Deck 1 Trough Impact ksi</th>
<th>Deck 2 Troughs Impact ksi</th>
<th>Beam 1 Trough Impact ksi</th>
<th>Beam 2 Troughs Impact ksi</th>
<th>Measured Static 10%Impact ksi</th>
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(b) Beam Distribution

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Max Deflection $= 0.214''$
Max Deflection of Trough $= 0.111''$

Figure 10: Dynamic Deflections of Girder at LVDT 7 for FL-9 Locomotive Load

Max Deflection $= 0.037''$
Max Deflection of Trough $= 0.102''$
Figure 11: Dynamic Deflections of Trough under FL-9 Locomotive

Figure 12: Dynamic Strains in Trough at SG 6, 10, and 9 under FL-9 Locomotive
Figure 13: Dynamic Strains in Trough at SG 10 under Genesis Locomotive

Max Strain = 328.7 μe
Max Stress = 2.86 kpsi

Max Strain = 325.5 μe
Max Stress = 2.77 kpsi