ICSBE2016-116

# LOAD TESTING OF BRIDGES FOR LOAD RATING

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**Abstract:** The AASHTO load rating method, which is the standard for determining the live load carrying capacity of bridges in the United States, is based primarily on the as-built material and section properties of a bridge. This supplemented with the ASSHTO load distribution factors, generally underestimates the "actual" load rating for bridges. Numerous field studies have shown that field test data provides an accurate representation of the actual bridge, and accurately predict the load rating of bridges. Field testing generally returns higher load ratings than what is predicted by the AASHTO method because it accounts for the actual load distribution, end fixity and stiffness of the structure. With advances in data acquisition and sensor technology, the use of diagnostic testing to evaluate bridge load ratings has become increasingly popular. This paper examines how current technologies are used, principally magnetic and reusable strain gauges and wireless data acquisition, to rapidly load test and improve the load rating of single span steel girder bridges.

**Keywords:** Bridge testing; Load rating; Load testing; Strain gauges.

## 1. Introduction

The National Bridge Inspection Standards (NBIS) regulations in the United States require that all bridges on public roads be assigned a load rating. Bridge owners, typically the Department of Transportation (DOT) of each state, are responsible for the load ratings of these bridges based on the NBIS. Load ratings should be carried out in accordance with the guidelines provided in the American Association of State Highway and Transportation Officials (AASHTO) Manual for Bridge Evaluation (MBE) [1].

According to the MBE, the objective of a load rating is to evaluate the safe live load carrying capacity of a bridge, based on as-built construction plans and material properties while taking into account any structural damage that may exist. This is important because historically bridges have been designed using several different design truck and lane loads. The rating is expressed as a Rating Factor (RF) or in terms of a particular truck weight in tons. Two different ratings are carried out: an Inventory Rating and an Operating Rating. The Inventory Rating represents the live load that can traverse a bridge an indefinite number of times, while the Operating Rating represents the maximum permissible live load that can traverse a bridge safely. When a bridge of insufficient capacity is found, the truck loads on the bridge are restricted by load posting.

The theoretical load rating calculated based on the MBE tends to be conservative due to many of the assumptions made in the calculation process. The live load distribution in longitudinal beams is one assumption that is evaluated based on recommendations provided in the AASHTO Specifications for Highway Bridges [2]. In addition, numerous bridges on secondary roads lack construction plans from which material and section properties can be identified, which leads to estimation of these properties. As numerous studies [3-7] have shown, field testing bridges can provide more accurate and reliable information regarding their current condition. The MBE [1] lists several factors that increase the live load capacity of a bridge and which can be evaluated through a load test. These include the following:

1. Unintended composite action
2. Unintended continuity/fixity

3. Participation of secondary members
4. Portion of load carried by deck

Field testing can also identify possible structural deficiencies in the bridge that were not observed during routine inspections. Due to its inherent advantages, the MBE dedicates an entire chapter to guidelines on establishing load ratings through non-destructive load testing. With the advancement of strain gauge and instrumentation technology, rapidly and efficiently load testing bridges is becoming more feasible. The paper details three bridges that were field load tested in Kentucky and details the instrumentation used and the resulting load ratings obtained.

## 2. Bridge Load Rating

The AASHTO MBE details three rating methods, Allowable Stress Rating (ASR), Load Factor Rating (LFR) and Load and Resistance Factor Rating (LRFR). While the more recent LRFR provides more uniform safety margin in terms of reliability, most state DOTs prefer LFD when load rating bridges designed using either allowable stress or a load factor based design. Since all three bridges highlighted in the paper are steel girder bridges built prior to the implementation of LRFR, the load rating has been carried out using LFR. The MBE specifies the following equation to calculate the rating factor (RF) based on LFR [1]:

$$RF = \frac{C - A_1 D}{A_2 L(I+1)} \qquad Eq\ (1)$$

$C$ = Capacity of the member

$D$ = Dead load effect on the member

$L$ = Live load effect on the member

$I$ = Impact factor

$A_1$ = Factor for dead loads

$A_2$ = Factor for live load

The factors $A_1$, $A_2$ vary depending on the level of rating performed, Inventory or Operating. The live load effect, $L$, is significantly influenced by the live load distribution factor. While field load testing of bridges provides the opportunity to calculate the actual distribution factors for a given bridge, to preserve analytical tractability, this paper uses the more conservative distribution factors recommended by AASHTO [2]. In addition, only the inventory load rating for a standard AASHTO HS20 truck, shown in Fig. 1, is presented in this paper.

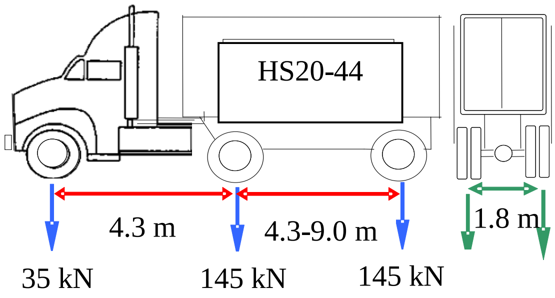


Fig. 01: HS20 truck specifications

The AASHTO standard trucks, including the HS 20, are hypothetical trucks developed for standardizing the design and load rating of bridges. While load rating is done using the standard truck types, when load testing, any suitable truck can be utilized loaded to a predetermined weight. Initial calculations are carried out to determine the required load of the test truck and the positions to be loaded to produce maximum effects on the bridge. It should be noted that based on local state regulations, bridges are typically load rated for several different truck types in addition to the HS 20 truck to include trucks with different axle configurations. The load posting, if required, would be based on the lowest of these ratings.

The load rating through field load test results can be calculated based on the NCHRP Manual for Bridge Rating through Load Testing [8]. The manual provides an adjustment factor (K), based on the field test results and other criterion, to modify the Rating Factor calculated based on the AASHTO MBE [1]. The rating equation is shown below:

$$RF_T = RF \times K \qquad Eq\ (2)$$

Where;

$RF_T$ = Load rating factor based on field test

$RF$ = Rating factor from Eq. 1

$K$ = Adjustment factor

Adjustment factor K can be calculated based on the following equation.

$K = 1 + K_a \times K_b$ *Eq* (3)

$K_a$ accounts for both the benefit derived from the load test, if any, and consideration of the section factor resisting the applied test load. It is given by the general expression below:

$$K_a = \frac{\varepsilon_C}{\epsilon_T} - 1 \qquad Eq\ (4)$$

Where;

$\varepsilon_T$ = Maximum member strain measured during load test

$\varepsilon_C$ = Corresponding theoretical strain due to the test vehicle and its position on the bridge

$K_b$ accounts for the understanding of the load test results when compared with those predicted by theory, the type and frequency of follow-up inspections, and the presence or absence of special features such as non-redundant framing and fatigue-prone details. The Kb factor is as follows:

$$K_b = K_{b1} \times K_{b2} \times K_{b3} \qquad Eq\ (5)$$

The three compounded factors can be evaluated using the tables provided in the NCHRP Manual for Bridge Rating through Load Testing [8].

## 3. KY 32 over Lytles Creek Bridge

The bridge over Lytles Creek, on State Route 32, in Scott County KY (referred to as the KY 32 Bridge), is a single span steel girder bridge (Fig. 2). The bridge is 6.96 m wide and has a deck length of 6.71 m. The reinforced concrete bridge deck is supported on five W14x30 steel girders and was cast non-composite with the girders. The bridge did not have any constructions plans, hence all dimensions and material properties had to be measured or estimated.

Fig. 02: KY 32 over Lytles Creek Bridge

For theoretical load rating the bridge is considered simply supported; the steel girders were embedded in concrete diaphragms at the abutments, which were cast integral with the deck. The concrete deck was also cast such that the top flanges of the steel girders were embedded. The purpose of performing a field load tests on the bridge was to incorporate the unintended composite action and the end fixity in to the load carrying capacity of the bridge and eliminate the need for the 124.6 kN (14 tons) load posting on the Bridge.

The primary instrumentation on the bridge consisted of reusable strain gauges attached to the girders and linear variable differential transformers (LVDT). Fig. 3 depicts the placement of the strain gauges and the LVDTs. Instrumentation was placed on girders G1, G3 and G5 at mid-span and at quarter span on girder G3. At each instrumentation point, three strain gauges were placed at the top, center, and bottom of the web and one strain gauge on the bottom flange. All four LVDTs, which measured the vertical displacement of the bridge, were placed adjacent to the strain gauge on the bottom flange. Fig. 4 shows the LVDTs and strain gauges at mid-span and quarter-span of the center girder (G3).

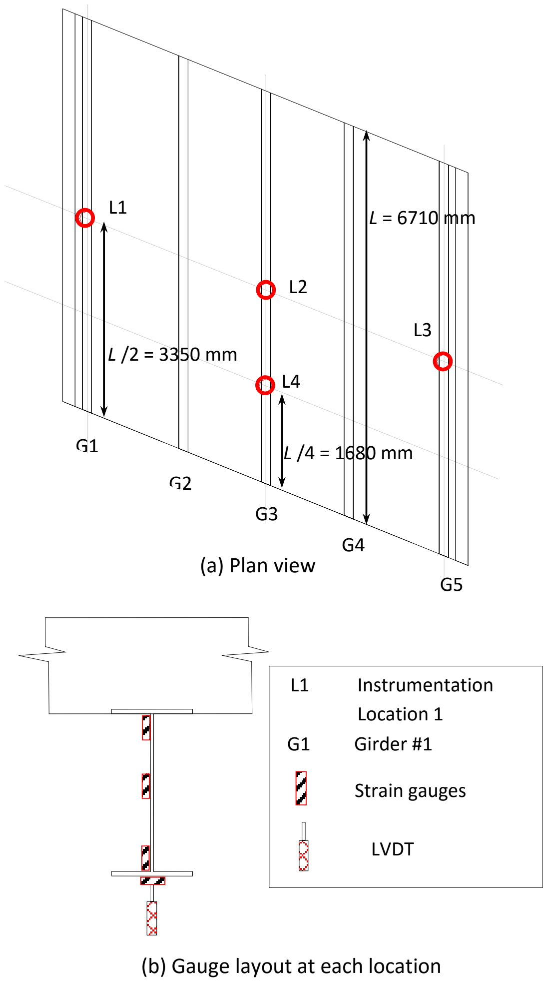


Fig. 03: Strain gauge and LVDT layout

Fig. 04: Instrumentation at location L2 and L4

A single axle dump truck, with measured individual axle weights, was used for loading the bridge. While multiple load positions were studied, only data for the load position that caused the maximum displacements and strains at mid-span of the bridge, which would in turn control the load rating, are presented here. The theoretical load rating factor (RF), for maximum moments at mid span of the center girder, from equation (1) was calculated to be 0.58 for an HS20 truck. Based on the maximum strains generated by the loaded test truck and its corresponding theoretical strain Ka was calculated to be 4.71. While Kb was evaluated to be 0.64, for the test truck and an HS20 load rating truck, resulting in the modification factor K being 4.02. Incorporating the modification factor in equation (2) provides the upgraded load rating factor based on the test results (RFT) as 2.34.

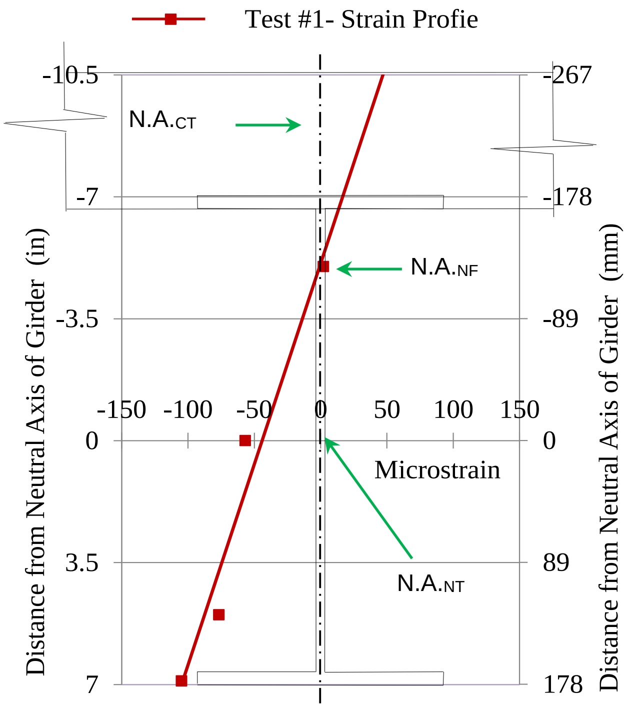


Fig. 05: Strain readings at mid-span

The effect due to the unintended composite action and end fixity is clearly visible through the strain profile at sensor location L2 at mid-span of the center girder (Fig. 5). For a non-composite bridge, the theoretical neutral axis (N.A.NT) would be at mid-height of the steel girder. The neutral axis from the field load test (N.A.NF) was between this point and the theoretical neutral axis for the fully composite section (N.A.CT). The theoretical elastic neutral axis for the composite section, found from section transformation, is estimated at 245 mm above the theoretical non-composite neutral axis.

## 4. KY 1068 over Laurel Fork Bridge

The Bridge over Laurel Fork on State Route KY 1068 in Lewis County, KY (referred as the KY 1068 bridge) is 18.7 m long and 8 m wide. The original construction date of the bridge is unknown. The bridge was expanded in 1954 from a single lane to a double lane bridge by adding two additional girders on one side. The current bridge construction includes four W33 x 130 girders with a 165 mm non-composite concrete deck. Each of the two original steel beams was constructed by splicing three smaller beams together, with diagonal cross bracing between the spliced beams. The two new beams lack splicing and have transverse floor beams as bracing at third points. All steel beams are spaced 1.8 m apart. While the steel beam ends were embedded in the abutment wall providing some partial fixity, due to the theoretical load rating, the bridge had a load posting of 151.2 kN (17 tons).

Fig. 06: Magnetic strain gauge calibration

To reduce the time spent on attaching strain gauges on the bridge and running wires to connect the gauges with the data acquisition system, the use of magnetic strain gauges interfaced with a wireless transmitter as a means to gather data wirelessly was evaluated on this bridge. Initial laboratory testing calibrated the magnetic strain gauges and evaluated their capabilities. Tension tests and beam tests were carried out to evaluate the strain limits. Tension tests were used to evaluate the effects of surface condition on the accuracy of the strain reading. Fig. 6 highlights one of the tension tests, where the magnetic gauges have been attached to a steel plate and the top two gauges on either side of the plate are attached to cleaned steel, while the bottom two gauges are attached to the areas with a corrosion cover. The foil gauge (covered in orange tape) used to calibrate the magnetic gauges and the wireless transmitters are also visible in the figure. The tests indicated that the magnetic gauges were accurate up to approximately 500 microstrain when the attached steel was clean. This fell to about 300 microstrain in the presence of rust or a coating. The values were acceptable for the expected strains in typical bridges to be load tested under the loading expected to be placed on the bridge.

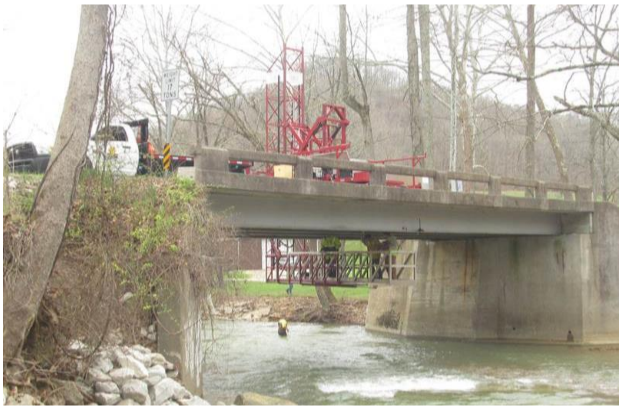

Fig. 07: Gauge application on KY 1068 bridge

Fig. 08: Loaded dump truck placed on deck

The gauge installation using a truck mounted mobile platform on the KY 1068 bridge is shown in Fig. 7. As the magnetic strain gauges were used for the first time on a bridge, foil strain gauges were also used as a backup. While the magnetic gauges could be installed in several minutes, the foil gauges took several hours, with installers having to run the wires back to the data acquisition system setup on the streambank. A dual axle dump truck weighing 274.5 kN (30.8 tons) was used to load the bridge with several load positions studied to evaluate the load distribution on to the girders. Girder lines were measured and marked on the deck to position the truck. The truck placed on the bridge deck is shown in Fig. 8.

Fig. 9 illustrates the strain readings for when the tire lines were on top of the center girders and the two rear axles were over the mid-span of the bridge. Foil gauge and magnetic strain gauge readings are shown with the layout of the two types of gauges next to each other shown in the photo within the figure. While the readings on the magnetic gauges were slightly higher than for the foil gauges, overall they correlated well, and were judged acceptable due to the rapid deployment and conservative nature of the load rating results. The expected results, had the bridge behaved as a simply supported structure are also included in the plot, showing that the partial fixity at the girder ends possibly assisting in carrying some of the loads on the bridge. In addition, the neutral axis is elevated above the mid-height of the steel girder, indicating some partial composite action, possibly due to the friction between the top flange of the steel girder and concrete deck.

The $K_a$ factor, showing the effectiveness of the load tests, was calculated to be 1.78. While $K_b$ was evaluated to be 0.80, for the test truck and an HS20 load rating truck, resulting in the modification factor $K$ being 2.42. While the theoretical load rating factor ($RF$) of the bridge was 0.59, the field test rating ($RF_T$) was 1.44 for an HS20 truck.

## 5. KY 220 over Martins Branch Creek Bridge

The KY 220 over Martins Branch Creek Bridge (referred to as the KY 220 bridge) in Hardin County, KY is a single span steel girder bridge with a non-composite concrete bridge deck. The bridge was built in 1935 and

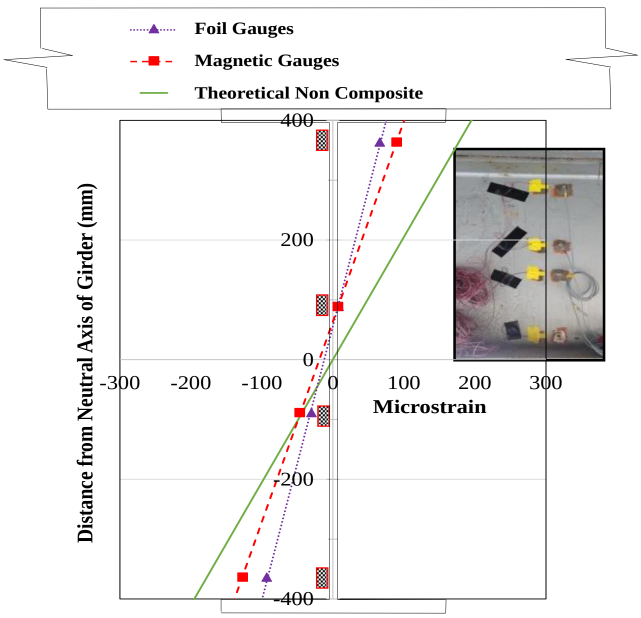


Fig. 09: Magnetic strain gauge results

did not have any design or construction plans. The 8.3 m (7.5 m clear span) long bridge is 6.1 m wide and rests on six steel beams similar to a W18×50 section. Unlike the previous two bridges, the KY 220 Bridge did not have the girder ends embedded in to the abutment nor were the top flange of the girders embedded in the concrete deck to provide a degree of compositeness. The close spacing between girders (1.2 m) coupled with lateral stiffeners every 2.4 m was expected distribute the live loads more evenly among the girders when compared to the AASHTO specified distribution factor.

The bridge was also selected to evaluate a novel method of displacement measurement using optical motion measurement. Because a relatively dry creek bed offered easy access, the girders were instrumented to evaluate displacement using Linear Variable Differential Transformer (LVDTs) to compare with the measurements obtained from the patented optical sensing technology using high-resolution video cameras. For load rating purposes, reusable strain gauges were also installed but interfaced with wireless transmitters so that information could be collected without having to lay down long cables. Foil type strain gauges were attached as a backup. Part of the instrumentation is seen in Fig. 10, which shows the wireless transmitter located on the creek bed floor. One of the high-resolution video cameras can be seen on a tripod on the embankment in the background.

Fig. 10: Sensor equipment underneath the bridge

The results from the load test on the KY 220 bridge is not produced here as certain parts of the research are still ongoing. But what was found during the testing was that the reusable strain gauges coupled with the wireless transmitters provided rapid deployment of the gauges along with accurate strain readings, cutting down on the test time.

**6. Summary and Conclusions**

This paper described field load testing carried out on three bridges, highlighting the types of sensors used and their effectiveness. On the KY 32 bridge and the KY 1068 bridge, load rating was carried out based on the strain readings obtained from the load test. These were compared with the theoretical load ratings. Table 1 summarizes the results for the inventory level rating for a standard HS20 truck. It should be noted that the rating is only based on strength (using the AASHTO load distribution factor). Serviceability criteria such as deflection limits and overloads are not factored into the load rating for the purpose of direct comparison.

As detailed earlier, both bridges were load posted due to the load rating factor for several truck types being less than one. The field load tests revealed the load rating factor for strength was adequate for both bridges for an HS20 truck load (Table 1).

Table 01: Load rating results

| Bridge | Analytical Rating Factor | Load Test Rating Factor |
|---|---|---|
| KY 32 – Scott Co. | 0.58 | 2.34 |
| KY 1068 – Lewis Co. | 0.59 | 1.44 |

While magnetic strain gauges performed well in the field and reduced gauge installation time, due to the rugged requirements of field testing, they will not be considered for any future deployments in the current status of the technology. Reusable strain gauges coupled with wireless transmitters balanced rugged performance with low installation times

The development of better, low-cost, wireless, and non-contact sensing technology, field load rating is now a more feasible option for evaluating bridge load ratings. In addition to accurately describing bridge behaviour and highlighting any unintended factors that may increase the load rating, this technology can also be employed to diagnose structural deficiencies.

**Acknowledgement**

The funding for the research is through a grant from the Federal highway Administration (FHWA) and the Kentucky transportation Cabinet (KYTC). The authors are grateful to the KYTC personnel that assisted with the field testing in numerous ways.

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