ABSTRACT: The status of highway infrastructure in the U.S. is in a critical state. Nearly 1/3 of U.S. bridges are nearing the end of their design life and one in ten bridges is categorized as structurally deficient. While the design and construction of the next generation of US highway bridges is underway, existing structures must remain operational through proper visual inspection and load rating. The current bridge evaluation paradigm is based on a visual inspection process, which can be subjective and potentially could lead to overly conservative bridge load ratings. Given that asset allocation for bridge management is highly dependent on load rating, the accuracy of these rating factors is vital for effective bridge management. In this paper, a baseline structural model is created and calibrated using field collected structural health monitoring data. The structural model is then used to calculate load rating factors of the bridge at both current and possible damaged conditions.

INTRODUCTION

The development of infrastructure management protocols in the United States is at a critical juncture. Many bridges are nearing the end of their original design life and a country-wide rehabilitation, replacement and retrofit phase is underway. Recognizing the need to capitalize on advances in structural health monitoring and assessment technologies, the Federal Highway Administration (FHWA) began a 20-year study in 2007 entitled the Long-Term Bridge Performance (LTBP) program, which will assess bridge performance changes over time using instrumentation, structural modeling and advanced data management techniques (Ghasemi, 2009). This effort focuses on the “work horse” highway bridges, i.e. the short to mid-span highway bridges, as opposed to larger signature bridges. Such spans comprise the majority of US bridges.

One of the goals of LTBP program is to improve inspection standards and condition assessment through non-destructive testing (NDT) and structural health monitoring (SHM) (Ghasemi, 2009). Recent research by Huang (2010), Barr, et al. (2006) and Yost, et al. (2005) built on Lichtenstein (1998) to show the viability of using a calibrated structural model to calculate periodic load rating factors. Concurrently, the International Society for Structural Health Monitoring of Intelligent Infrastructure (ISHMII) defines SHM as “a type of system that provides information on demand about any significant change or damage occurring in the structure” (ISHMII, 2010). SHM typically includes data collection through sensor based instrumentation, post-processing the collected responses with respect to predicted responses from both hand calculations and structural modeling and then monitoring of any changes to the structure. This process can be effectively integrated with a structural model of the instrumented bridge for the purpose of objective condition assessment, parameter estimation, design verification and bridge management.

In coordination with the goals of the LTBPP and ISHMII, the integration of instrumentation and modeling into the initial design and construction of a bridge structure will shift the design paradigm from an opening day focus to the long-term focus. This paradigm shift will create a quantitative performance measure for the new fleet of bridges that will be available throughout their service life. This information can also be used to develop a more accurate load rating for in-service evaluation.

This paper presents an integrated load rating protocol that includes 3D structural modeling, SHM, and NDT beginning during the design phase and continue throughout the bridge life. This approach will create an objective, quantitative, effective and efficient asset allocation tool for long-term bridge decision-making. (Santini-Bell and Sipple, 2009).
BRIDGE LOAD RATING

1.1 Background
Concurrent with the aging and deterioration of the nation’s bridges is the increased use and dependence on the highway system. In 2010, FHWA reported an increase of 50 billion miles traveled on U.S. roads each year from 1985 to 2008 (FHWA, 2011). Tonnage carried by trucks is also expected to double in the next 20 years (AASHTO, 2008a). Increased truck weights have been found to expedite the deterioration and decrease the remaining useful service years of highways bridges (Fu, et al., 2008). Combined with the fact that 74% of goods in the U.S. are transported by trucks on the interstate highways system, it is essential to the national economy that highway bridges remain in service. Therefore, inspection and load rating procedures need to accurately reflect the capacity and condition of existing bridges.

The level of deterioration or damage noted during the visual inspection is used to determine section loss and reduce component stiffness values. Strength calculations based on the reduced section properties, similar to those calculated during the design process, are then completed to determine the proportion of design live load that the critical damaged member can safely support. These calculations form the basis of a load rating, which is a ratio of available strength to the response from the chosen live load scenario. Based on inspection results and the rating factors, a decision can be made as to whether the bridge needs repair or replacement, may remain in service without repair but with a posted weight limit on the bridge (AASHTO, 2003), or may remain in service with no action.

Over the life of the bridge, SHM through instrumentation and baseline modeling can provide objective information to verify inspection reports, provide supplemental information between routine inspections, or trigger a special inspection to address a change in the response of the bridge.

1.2 Load Rating Procedure
Traditional load rating was developed to replace inventory and operating stress levels used for posting with rating levels based on available information including statistical descriptions of load and strength to create a reliability formulation to assess the safety index (NCHRP, 1987). Load Resistance Factor Rating (LRFR) assessment is a reliability-based methodology to determine the capacity of each bridge element (NCHRP, 2001).

Although bridge inspectors are highly trained, and every attempt is made to obtain objective reports, bridge inspection is still a highly subjective process, and results can be highly irregular. A study into the reliability of bridge inspections showed that visual inspection is the most common form of bridge inspection, bridge inspectors are not required to undergo any vision testing, and professional engineers typically do not conduct the inspections (Phares et al., 2000). Load ratings are calculated based on the results of inspection condition ratings (AASHTO, 2008c). Load ratings are a more objective measure of bridge health, but they are still calculated based on the results of subjective observations.

Repair funds are allocated, to an extent, based on results of bridge inspections. The measured section loss in an inspection report is also used to update live load rating factors (RF). A live load RF is a ratio of a bridge’s live load capacity to a worst case scenario loading condition.

The calculation of the load rating using the LRFR method of the 2008 AASHTO Manual for Bridge Evaluation is given by Equation 1.

$$RF_{LRFR} = \frac{C - \gamma_{DC} DC - \gamma_{DW} DW - \gamma_{P} P}{\gamma_{LL} LL (1 + IM)}$$

In this equation, RF is the rating factor of a structural element; C is its capacity in terms of a given force effect; DC is the effect of dead load structural components of the element; DW is the effect of dead load wearing surfaces and utilities; P is the effect of other permanent or superimposed dead loads; LL is the effect of a worst-case live load condition. IM and each of the γ-factors are the dynamic load allowance and LRFD load factors, respectively, according to AASHTO LRFD bridge design specifications (AASHTO, 2008c). The calculation of Inventory and Operating RF’s differ only in the value of the live load factor, γLL.

It is important to note that in current bridge management practice, load ratings are calculated for each structural member—an elemental approach, just like the design process—and does not take advantage of system wide behavior. Although the baseline structural model may not enhance the accuracy of the bridge inspection process, it can provide insight into the capacity of a bridge in terms of a RF by inputting the structural deficiency noted in the inspection into the calibrated structural model.

The load rating protocol described in this paper accounts for system structural behavior as opposed to the traditional component-by-component method. System behavior is evaluated by using a 3D structural model that is calibrated with NDT data from a load test and visual inspection information for a three-span, steel girder highway bridge in central Massachusetts. The bridge was instrumented during construction for long-term SHM. A baseline structural model was created and calibrated using NDT data collected from a controlled load test prior to
bridge commissioning (Sanayei, et al., 2012). The calibrated model was then used for bridge management as part of an objective load rating protocol. Load ratings are then compared to an AASHTO load rating calculated using the LRFR method (AASHTO, 2008c) as a benchmark.

2 CASE STUDY: VERNON AVENUE BRIDGE

2.1 Vernon Avenue Bridge Description

The Powder Mill Pond Bridge, referred to at the Vernon Avenue Bridge (VAB) in this paper, is located in Barre, Massachusetts, see Figure 1. The bridge carries Vernon Ave over the Ware River at its junction with Massachusetts State Route 122. The VAB is a 3-span continuous composite steel girder bridge with a reinforced concrete deck. The bridge is 12.7m (41.7 ft) wide in the south and center spans and widens to 19m (62.3 ft) at the north abutment. The bridge is 47m (154 ft) long with a 23.5m (77 ft) center span and two 11.75m (38.5 ft) outer spans. The concrete deck is a 200mm (8 in) thick deck and is made of high performance concrete with a nominal design strength of 30MPa (4.35ksi).

Figure 1. The Vernon Avenue Bridge (VAB)

There are six main girders that run the length of the bridge, evenly spaced at 2.25m (7.4 ft), giving a deck overhang of 732.5mm (28.8 in). At the quarter point of the north span, two new fascia girders are mechanically connected to the exterior girders and fan out linearly at a 10° angle to the edge of the bridge in order to accommodate a widened deck at the junction of SR 122. In this paper, the main girders will be referred to by number with girder 1 being the westernmost girder, and girder 6 being the easternmost, as in Figure 2.

All steel girders are wide-flange shapes made of weathering steel conforming to AASHTO M270M Grade 345W. The interior girders are W920x238 (W36x160), the exterior girders are W920x345 (W36x232), and the short fascia girders of the north span are W920x201 (W36x135). At the abutments, the steel diaphragms are encased in concrete, and are poured integrally with the deck concrete. Each girder is supported by elastomeric bearing pads at the abutments and at the piers. The grade 3 60-durometer neoprene pads with A1011 Gr250 steel shims are circular in shape with a 350mm (13.8 in) diameter and a 61mm (2.4 in) thickness. The substructure consists of the concrete abutments and two concrete piers. Each pier consists of three columns with a 915mm (36 in) diameter circular cross-section beneath a pier cap of rectangular cross section 1.2 m (47 in) wide by 1.0 m (39.4 in) tall.

The VAB instrumentation data used in this paper includes information collected from strain gauges and temperature sensors. One hundred strain gauges were installed on the bridge on all six steel girders at stations 2, 4, 6, 8 and 10, see Figure 2. At each instrumented station, four strain gauges were installed: two on the top side of the bottom flange, and two on the underside of the top flange. The only exception to this pattern is on the exterior girders where the gauges were installed on the interior side of the girders to protect the gauges from the weather. The gauges were installed on the girders during fabrication in the steel fabrication yard, prior to steel erection (Sanayei, et al., 2012).

Figure 2. The VAB Instrumentation Plan

2.2 2009 Load Test

A pseudo-static load test was conducted on September 3, 2009, prior to the bridge being opened to traffic, to calibrate a baseline structural model for bridge management and experimental load rating calculation. Static load tests were conducted by positioning a truck loaded with aggregate at various locations across the bridge. The truck was a tri-axle dump truck with tandem wheels in the rear and one rear axle raised for maximum concentration of wheel loads. Each front tire had a contact patch of 203 mm (8 in) long by 292 mm (11.5 in) wide and each rear tire had a contact patch of 203 mm long by 546 mm (21.5 in) wide. The gross vehicle weight was measured at 320 kN (72 kip), with a distribution of 86 kN (19.3 kip) to the front axle and 116 kN (26 kip) to the rear axles. The truck was positioned at
The EDM consisted of frame elements for the steel girders and shell elements for the concrete deck, see the extruded view in Figure 3. The structural model was created based on design documents and shop drawings and refined using observations made during construction, as well as material testing results and previous research. The bridge cross section was enhanced to include steel reinforcement in the deck by utilizing a layered shell element available in SAP2000®. The material properties of the steel were assumed as specified in the design plans and those of the concrete deck were representative of as-built conditions according to material testing results and concrete mix design specifications. The EDM is created using all available objective information to increase model accuracy for bridge load rating and long-term structural health monitoring. (Lefebvre, 2010).

Figure 3. Structural Model of the VAB in SAP 2000®

The four modifications described above served to calibrate the EDM to reflect the instrumentation data collected during the load test. The calibrated baseline EDM was found to correlate well with measured strain and deflection measurements (Santini-Bell, et al., 2011), accurately predicting bridge behavior. The calibrated model was verified through parallel research on structural health monitoring (Sanayei, et al., 2012) for use as part of an alternative method for load rating.

3 LOAD RATING OF THE VAB

3.1 Rating Overview

Load rating was calculated for the VAB for bending moment under the Design Inventory and Operating: Strength I limit state as defined by the 2008 LRFD bridge design specifications. The inventory rating will also be extracted from the response of the EDM under the same live load conditions. In order to make a direct comparison between the RF’s, the dead load components included in all calculations are limited to the components included in the EDM, as described in the next section. Therefore, the DW and P components are zero.

A live load RF is the proportion of available live load capacity to overall load capacity, after considering dead load effects. The 2008 AASHTO Manual for Bridge Evaluation defines three types of ratings: Design, Legal, and Permit. There are two types RF under the Design rating type, which are inventory and operating level RF. This paper will focus on the Design Inventory and Operating Level RF, which is the first level of evaluation. Only a bridge with a Design RF greater than one would be evaluated for Legal or Permit Load Rating. The Design RF reflects the existing bridge and material conditions and is based on the load, for which the bridge was designed. (AASHTO, 1996) The load rating factors calculated in this paper use HL-93 as per LRFR standards (AASHTO, 2008c), which is a design truck with three axles, each axle weighing 35.6 kN (8 kip), 142 kN (32 kip) and 142kN (32 kip) and an additional lane load of 9.34 kN/m (640 lbf/ft) (AASHTO, 2008b).

3.2 Rating Overview

To determine the feasibility of using the calibrated EDM for load rating, the LRFR design inventory rating factor was first calculated. The first step in this process was to determine the capacity of each of the girders. Since the geometry of the exterior girder-slab system differed from that of the interior, two capacities were calculated. The capacity of the
composite girders were calculated at its plastic moment, \( C = M_p \), using the strain compatibility method. The capacities for both interior and exterior girders are 6150 kN*m and 7414 kN*m, respectively. These capacities are used for the calculation of load rating factors in both the LRFR and the EDM rating.

After the capacities were calculated, the applied loads were used to obtain the maximum applied moment on the bridge, starting with dead loads. The structural components included in the EDM were the steel girders and diaphragms, the concrete haunch, deck, and the safety curb. A number of other components exist on the actual bridge but were only included in the EDM as additional dead load, such as the steel railing and the asphalt wearing surface. A common design assumption is that dead load distributes itself evenly across the girders, and therefore all components can be summed and divided by the number of girders. When multiplied by the dead load factor \( \gamma_{DC} = 1.25 \), the dead load per girder is \( w_0 = 21.5 \) kN/m (1,470 plf).

After dead loads were calculated, HL-93 factored live loads moments were determined. LRFD bridge design live load distribution factors consider span length, bridge width, girder spacing, and the depth and relative stiffness of the girders and deck (AASHTO, 2008b). The distribution factors also differentiate between interior and exterior girders, and the loading of one lane of traffic versus multiple lanes. Therefore, four different distribution factors are calculated: two for the interior girders and two for the exterior girders. The maximum distribution factors of 0.676 for exterior girders and 0.587 for interior girders were used in the live load analysis. The live loads are multiplied by the load factor of \( \gamma_{LL} = 1.75 \) for the inventory rating and \( \gamma_{LL}=1.35 \) for the operating rating, a dynamic impact factor of 1.33 (1+IM), and the appropriate distribution factor.

The dead loads are applied uniformly across the entire girder length producing a moment of 695 kN.m (512 kip.ft). The live loads are patterned for maximum force effect. Using influence lines for positive bending moment at the midspan, the worst case positive bending moment occurs at the midpoint of the center span with the lane load applied across the length of the center span and the center axle of the truck at the midpoint of the center span producing moments for 1440 kN.m (1063 kip.ft) and 1660 kN.m (1225 kip.ft) for interior and exterior girders, respectively. The resulting LRFR inventory factors are 3.8 and 4.1 for the interior and exterior girders, respectively. The LRFR operating factors are 4.9 and 5.3 for the interior and exterior girders, respectively.

3.3 Load Rating Using Baseline EDM

In order to use the EDM for load rating, equivalent loading conditions were entered into the analysis program. The self weight of the bridge, scaled by \( \gamma_{DC} = 1.25 \), was used as a dead load. The bridge width inside the curbs is 11.725 m (38.5 ft) and dictates that there are three 3.05 m (10 ft) traffic lanes to be loaded. Therefore, three trucks were applied to the bridge, using wheel loads instead of axle loads with a \( \gamma_{LL} = 1.75 \) scaling factor. No distribution factors were used in the EDM, since the live load distribution was accomplished through system behavior inherent in the EDM, see Table 1.

Therefore each girder has an individual load rating. The equation for the LRFR factor is manipulated to include the response of the calibrated EDM, as shown in Equation 2. Load factors are no longer included in the calculation of the RF, as they were included in the EDM load cases.

\[
RF_{Model} = \frac{C-DL_{Model}}{LL_{Model}+IM}
\]  

The most significant assumptions made in the loading of the bridge were associated with the lane loading. The lane load was applied only to the center span, such that the entire width of the bridge center span minus the safety curbs was loaded with a factored traffic load of 5.37 kN/m² (112 psf). The ratings were also found to be close to the LRFR inventory factors, see Table 1.

<table>
<thead>
<tr>
<th>Girder</th>
<th>Moment (kN.m)</th>
<th>Rating Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Dead Load</td>
<td>Live Load</td>
</tr>
<tr>
<td>1</td>
<td>1059</td>
<td>1580</td>
</tr>
<tr>
<td>2</td>
<td>808</td>
<td>1211</td>
</tr>
<tr>
<td>3</td>
<td>836</td>
<td>1291</td>
</tr>
<tr>
<td>4</td>
<td>836</td>
<td>1291</td>
</tr>
<tr>
<td>5</td>
<td>812</td>
<td>1211</td>
</tr>
<tr>
<td>6</td>
<td>1058</td>
<td>1579</td>
</tr>
</tbody>
</table>

The more evenly distributed ratings can be attributed to the system behavior that is accounted for by EDM. For example, the deck overhang on the VAB is a relatively small at 732.5 mm (28.84 in) and the effective overhang is even less at only 237.5 mm (9.35 in). A small effective overhang corresponds to an exterior girder that is not required to carry much of its own load.

3.4 Load Rating Using Non-Destructive Test Data

There were two portions of the diagnostic load tests that were performed on September 3, 2009, prior to the bridge opening. The first one was the crawl
speed load test and the other was the stop locations load test. The crawl speed load test had three load patterns along the length of the bridge, X1, X2 and X3. Load path X1 was 0.61m (2ft) off of the northwest curb and X3 was 0.76m (2.5ft) off of the southwest sidewalk. Load path X2 was located on the girder 3 in the center of the travel way width, see Figure 4.

Figure 4. Load Paths for the 2009 PMPB NDT

The results from 2009 diagnostic load test showed that the test truck was not placed on the bridge to obtain maximum load effect from all girders. Therefore, the benefit of 2009 load test could not be used to evaluate load factors of girder 1, girder 4, and girder 6. However, girders 2, 3, and 5 were stressed at sufficient levels to calculate K factors larger than one and result in higher rating factors as shown in Table 2.

Table 2. Adjustment Factor and Load Rating for the 2009 Load Test of the VAB

<table>
<thead>
<tr>
<th>Girder #</th>
<th>K</th>
<th>Inventory RF</th>
<th>Operating RF</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>4.1</td>
<td>5.3</td>
</tr>
<tr>
<td>2</td>
<td>1.45</td>
<td>5.5</td>
<td>7.1</td>
</tr>
<tr>
<td>3</td>
<td>1.5</td>
<td>5.7</td>
<td>7.4</td>
</tr>
<tr>
<td>4</td>
<td>1</td>
<td>3.8</td>
<td>4.9</td>
</tr>
<tr>
<td>5</td>
<td>1.56</td>
<td>5.9</td>
<td>7.7</td>
</tr>
<tr>
<td>6</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Adjustment factor of K represents the benefit of load test. If K factor is equal to 1, the bridge response agrees with theory. If K factor is greater than 1, the response of the bridge is more favorable than the theory (AASHTO, 2008c). After processing the 2009 diagnostic load test data the K factors of girder 1 and girder 2 were found equal to 1 therefore theoretical rating factors were used for these girders. Because of the sidewalk, the position of the truck was not close enough to girder 6. Since the distribution factor for girder 6 was really low, NDT data could not be used to rate girder 6.

4 DISCUSSION

The NBIS provides a set of standards to ensure consistent bridge condition evaluation practices. Most evaluation is currently based on routine visual inspections which are, to a degree, subjective. With increased use of instrumentation and development of SHM systems it is important for each instrumented bridge to have an associated structural model to take full advantage of the collected data. These structural models can then be calibrated to reflect the current condition of the bridge. A properly calibrated structural model can be used to calculate changes to load distribution due to changes in member section properties and thus obtain a more comprehensive and objective load rating. This calibrated model can also be used for load rating in conjunction with measured data.

This case study supports the use of a calibrated EDM for load rating, showing that the EDM can be used to effectively calculate a baseline load rating for a new bridge. This inclusion of system-based load rating method for bridge management would require additional evaluation guidelines for all bridge elements, including elements such as bridge parapets and railing that are not traditionally considered to participate in the structural performance of the bridge.

This study also showed the adjustment factor calculated based on the 2009 Load Test per Section 8 of the MBE allows for load rating through diagnostic testing, which includes system behavior and the impact of non-structural elements (AASHTO, 2008c). The design of the NDT including load path and truck weight has a significant impact on the applicability of the adjustment factor and its value.

By adding specifications for structural model creation, instrumentation and monitoring, the MBE would provide bridge owners with proper guidance for the appropriate use of diagnostic testing and analytical modeling for load rating and decision-making relating to bridge management.

5 CONCLUSIONS

A system-based load rating procedure can help bridge managers objectively allocate the limited funding available to maintain the US infrastructure. Combining instrumentation, structural modeling, structural health monitoring, and model calibration with inspection reports will produce an objective and quantitative load rating that both reflects the observed condition of the bridges as well as the 3D system behavior. Once the EDM is verified using measurements collected during a controlled load test, bridge managers can be confident that it reasonably reflects bridge behavior. This EDM can be updated for field observations, such as section loss in a girder, or via structural identification using NDT data for parameter changes such as bearing pad stiffness values (Santini-Bell, et al., 2007).
Further adjustments are required in the overall process of bridge design, inspection, reporting and load rating protocols in order for bridge managers to take full advantage of the benefits of instrumentation, testing and modeling protocols. For example, a model created by the bridge designer and calibrated to the initial condition of the bridge can be maintained through each inspection cycle and used to load rate the bridge throughout its service life.

The additional cost of an instrumentation program, load testing and modeling will be offset by the ease of inspection documentation, increased consistency between inspection intervals, improved condition documentation between physical inspections, and capability for rapid and accurate load rating. For the case study presented in this paper, the additional cost of the instrumentation, load testing, and modeling, analysis, and evaluation was less than 10% of the construction cost. In the past decade, modeling, instrumentation, and data management capabilities have greatly improved, while their relative costs have decreased. It is expected that this trend will continue in the future making bridge load rating using NDT and calibrated FEMs even more feasible.

Currently it is not practical to instrument and install SHM systems on most existing bridges. But with the trends of improving technology, ease of application, and decreased costs, one can envision a time in the near future when the approach suggested by this article will not only be feasible, but desirable for many bridge structures.

6 ACKNOWLEDGEMENTS

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7 REFERENCES


