Load Rating of Timber Bridges through Physical Testing and Software Application

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Abstract— *This paper presents the applicability of a field testing and a commercially available structural testing system in load rating of timber bridges. Two bridges were instrumented with strain transducers and then controlled load tests were conducted with a 3-axle dump truck driven at a crawl speed across the bridges. The location of the instrumentation was selected such that maximum live-load response of the longitudinal girders could be determined. Strain transducers were placed at the midspans of the bridge to determine the flexural strains. Two transverse truck paths were established to develop the behavior characteristics of the bridges. As the truck continuously moved along the prescribed paths, strain data was recorded with respect to the loading position of the truck over the bridge. Finite element models of the bridges were developed and calibrated based upon the observed behavior of the bridge and the field measured strains. Results from the calibrated model were used to determine the load rating of the bridges.*

Keywords— Timber Bridges, Field Testing, Bridge Load Rating.

I. INTRODUCTION

Bridge evaluation or bridge load rating can be defined as a procedure to assess the adequacy of various structural elements of bridges to carry predetermined live loads. Load rating of existing and newly constructed bridges is typically carried out in accordance with the National Bridge Inspection Standards (NBIS) [1] and the American Association of State Highway and Transportation Officials (AASHTO) Manual for Bridge Evaluation (MBE) [2]. Studies have reported that the use of diagnostic load testing often results in higher bridge load ratings when compared to traditional analysis methods [3]. This is due to the fact that the traditional methods are based on information derived from theoretical analysis while the load rating through a diagnostic load testing is based upon field measured data that represents the actual bridge behavior.

This paper summarizes the efforts carried out in load rating of timber bridges through a series of field load testing and the use of Bridge Diagnostic Inc. (BDI) structural testing system. Load tests were conducted on two glued laminated girder bridges: Butler Bridge located in south central Alabama in Butler County, Alabama and Wittson Bridge in north central Alabama in Tuscaloosa County, Alabama. The main goals were to develop a realistic analytical model based upon field test data and conduct load ratings on the girders of these bridges. The BDI system consists of software and hardware components designed specifically for bridge load rating. The subsequent sections provide an overview of the field load testing, analytical model generation as well as a summary of the load rating results.

II. BRIDGE LOAD RATING METHOD

Among other alternatives such as the allowable stress design (ASD) and the Load and Resistance Factor Design (LRFD), the load rating method presented in this paper is based on the load factor design (LFD) method [4]. The BDI software applies the limit states for rating calculations by using the truck loads applied on the structure. The rating equation used by the BDI is of the same general format as the LFD method but the user needs to specify the load factors according to the following equation:

$$RF = \frac{C - \gamma_{DL} * DL}{\gamma_{LL} * LL * (1+I)} \quad (1)$$

where:

RF = Rating Factor
C = Capacity of section of interest
γ_{DL} = Dead Load Factor
γ_{LL} = Live Load Factor
I = Impact coefficient

The load rating through the BDI system can be completed in three phases: testing, modeling and rating of the bridges. Each phase requires its own tools and individual processes. The BDI system consists of five elements: a group of strain transducers used to measure the bridge response; a structural testing system (STS) unit for storing and transmitting data; a power unit for transferring commands to the system during the testing; an autoclicker for providing the load vehicle position over the span of the bridge; and an STS software for data processing and developing analytical models. The BDI software packet is mainly for analytical modeling part of the system and is composed of three components: WinGRF used for analysis and data presentation; WinGEN used to develop a finite element model of the bridge limited to beam and shell elements; and WINSAC used to analyze and calibrate the analytical model.

III. TESTING, MODELING AND LOAD RATING

3.1 Description of Bridges

The Butler bridge, shown in Fig. 1 (a), is a 84 ft x 24 ft – 7 in., two-span longitudinal glued-laminated girder bridge. Span 1 measure 24 ft while Span 2 is of 60 ft and each is measured from the center line of supports. As shown in the Fig. 2, each panel of the bridge deck measures 5 in. x 4 ft x 24 ft -7 in. These glued-laminated panels are tightly set against each other and attached to the girders with ring shank spikes. The bridge has a 3-in thick asphalt wearing surface. The girders selected for evaluation are 5 in. x 27.5 in. glued-laminated, and are simply supported. The bridge has no skew. The roadway width measured between the guardrails of the bridge is 27 ft – 7in. allowing two traffic lanes. The supporting substructure consists of reinforced concrete (RC) abutment back-walls, caps, RC wing walls, and steel H-piles with RC caps for the piers.

The Wittson bridge, shown in Fig. 1 (b), is a 232 ft x 16 ft, four-span longitudinal glued-laminated girder bridge, consisting of spans of 50 ft (Span 1), 50 ft (Span 2), 102 ft (Span 3), and 30 ft (Span 4) measured from the center line of supports. The bridge deck consists of several glued-laminated panels with each panel measuring 5 in. x 4 ft x 16 ft. These 4 ft wide transverse, non-connected, glued-laminated panels are tightly set against one another and attached to the girders with ring shank spikes. The bridge has a 2.5-in. thick asphalt wearing surface. The girders selected for evaluation are 6.75 in x 43 in. glued-laminated girders, and are simply supported. The bridge has no skew. The roadway width measured between the guardrails is 16 ft allowing one traffic lane in both directions. The supporting substructure consists of RC abutment back-walls, caps, wing walls, and steel H-piles with concrete caps for piers.

3.2 Instrumentation and Testing

In both bridges, Span 1 was instrumented at midspan to measure flexural strains. The location of the instrumentation was selected so that the maximum live-load response of the longitudinal girders in Span 1 could be determined, thus providing an understanding of the longitudinal and lateral load distribution characteristics. Several strain gages were installed with two gages on each girder – one near the top of each girder and one on the bottom of each girder. After the instrumentation was completed, controlled load tests were conducted with a loaded 3-axle dump truck driven at crawl speed across the bridge. Two transverse truck paths were utilized to establish the behavior characteristics of the bridges. Strain data was collected continuously as the truck crossed the bridge along the prescribed paths, and the truck position was monitored to measure strain as a function of vehicle position. For the Butler bridge, for load case 1, the driver side wheel was placed 2 ft from the longitudinal centerline of the bridge while, for load case 2, the passenger side wheel was placed 2 ft –3 in. from the south curb. The location of strain gages and load cases for Butler bridge are given in Fig. 3 while the axle weights and axle configuration of the load truck are given in Fig. 4. For the Wittson bridge, for load case 1, the driver side wheel was placed 2 ft from the east curb whereas for load case 2, the driver side wheel was placed 5 ft from the east curb.

(a) Butler Bridge

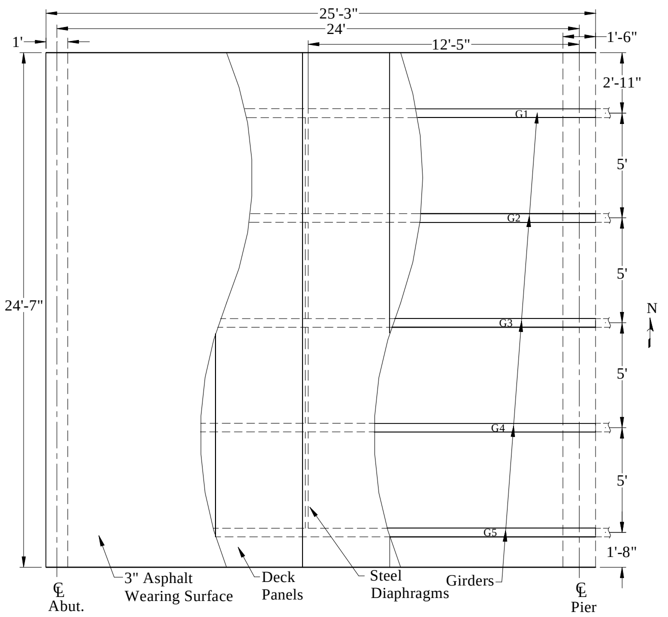
(b) Wittson Bridge

Fig. 1: Photograph of Butler and Wittson Bridges

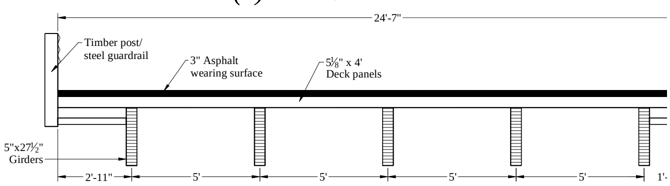
3.3 Preliminary Evaluation of Test Results

The measured strain data was first examined graphically to determine their quality and to provide a qualitative assessment of the bridges' response to the truck loading. In Fig. 5, the continuous lines represent the measured strain obtained at the prescribed gage location as a function of truck position as the load truck crossed the bridge. This qualitative assessment is important in that it provides considerable insight into the structure’s live load behavior and that the direction of quantitative investigation can be established. For both bridges, similar observations were made from the field data as follows:

- In general, all collected strains showed an elastic response.
- The neutral axis location was found to be slightly away from the mid-depth of the girder in the positive moment region, thereby verifying some unintended composite behavior of the girders.
- Although the girders were not continuous with the adjacent span, there were some continuity between Span 1 and Span 2. This behavior can be indicated by observing the tensile and compressive strains measured from the gages in Span 1 while the truck was crossing Span 2.
- For the Butler bridge, a maximum measured tensile strain of approximately 515 micro-strain was obtained from G5 at midspan of Span 1 for the load case 2. For the Wittson bridge, the largest measured strain was obtained from G1 at approximately 500 micro-strain.



(a) Plan View



(b) Typical Cross-Section of Span 1

Fig. 2: Details of Span 1 of Butler Bridge

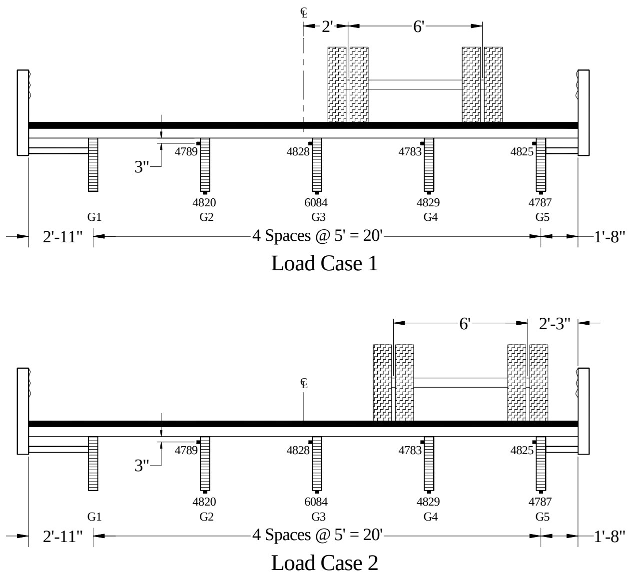


Fig. 3: Truck Load Cases and Strain Gage Location

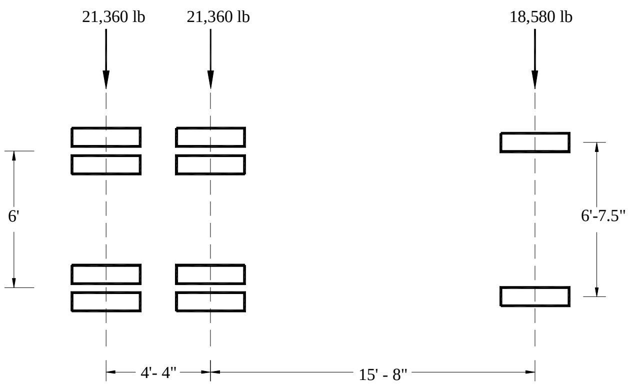


Fig. 4: Dimension and Weight of Load Truck

3.4 Modeling and Analysis

The computer model was developed to predict the overall live-load response of each girder. The purpose of generating the analytical model was to predict the structure's live load response as close as possible. Each bridge was modeled with beam, plate, and spring elements. Abutment support conditions were simulated by utilizing elastic springs at the ends of the bridge. Field measured information (e.g., span length, beam spacing, member cross-section, etc.) was used for defining the geometry of the computer model. Due to the observed continuity between Span 1 and Span 2, the entire bridge was modeled; although the magnitude of the unintended continuity was insignificant, it was necessary to develop a model that represents as close to the actual behavior of the bridge as possible since measured and theoretical strains were to be compared.

Once the computer model was generated, field tests were simulated in the model by applying the field truck weight and axle configurations to the model. After the initial analysis, analytical results were directly compared with the measured results to determine the accuracy of the model. In an effort to improve the accuracy of the model, a total of three different stiffness parameters (i.e., beam stiffness – I_y, deck stiffness – E, and support condition – K) were modified through an iterative optimization technique based upon comparison of the field and analytical strain results. To this end, an acceptable model was attained by minimizing errors between the measured and computed strains. Table 1 presents the statistical terms that summarize the accuracy of the computer model and Fig. 5 provides a comparison of the field measured and computer generated strain histories for the Butler birdge. In each graph, the computer generated strains are shown as markers at discrete truck intervals of 5 ft. The two sets of data for each gage indicate the two previously described different load cases. The analytical results for both bridges were quite similar.

Table .1:Model Accuracy

Statistical Term	Initial	Final
Percent Error*	7.6%	6.7%
Scale Error**	4.5%	4.6%
Correlation Coefficient***	0.96	0.97

*Sum of the strain differences squared, divided by the sum of the measured strains squared.

**Maximum error divided by maximum strain for each gage.

***Represents how well the shapes of the computed response histories match the measured response.

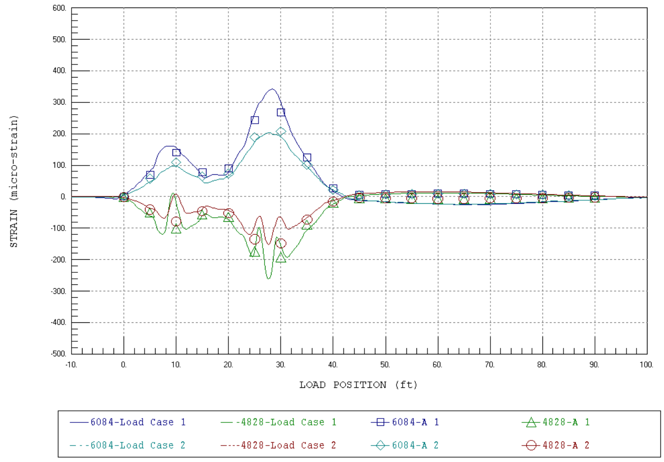
3.5. Load Rating Procedure and Results

The rating procedure is similar to standard load rating procedures except that a field calibrated computer model is used to predict the response of the bridge to the rating vehicle instead of using assumed load distribution factors and other assumptions.

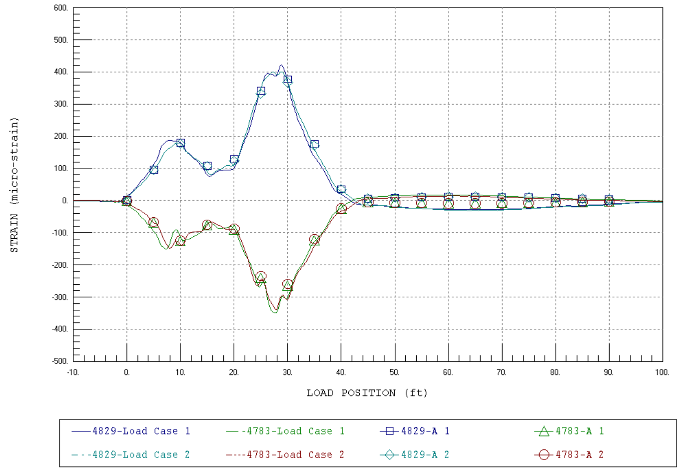
Member flexural capacities were computed for each girder following the appropriate AASHTO code equations for ASD rating methodology. Due to the lack of information on the actual condition of the girders, three different values of member unit stress in bending (an estimate of 2000, 2200, and 2400 psi) were utilized in computing member capacities; these values were adjusted by the applicable adjustment factors to obtain allowable unit stress of the member. Applied dead load included the member self-weight of the model plus an additional 0.027 ksf to account for the 3 in. thick asphalt wearing surface. Three ratings were performed based on the member capacities described above. Sample load rating results for each girder of the Butler bridge is given in Table 2.

Table .2: Rating factors based on HS-20 Design Vehicle

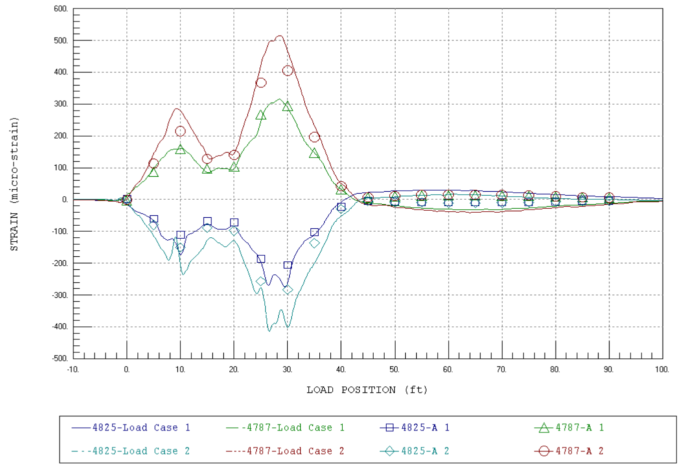
Unit stress used, psi	G2	G3	G4	G5
2000	1.61	1.20	1.24	1.50
2200	1.80	1.34	1.39	1.67
2400	2.10	1.46	1.53	1.84



(a) Strain Comparison in G3



(b) Strain Comparison in G4



(c) Strain Comparison in G5

Fig. 5: Sample comparison of Field Measured and Computer Generated Strains for the Butler Bridge

IV. CONCLUSIONS

After considering all the load conditions, it was found that the maximum tensile strain was attained only when the load is placed at or near the considered girder. The accuracy of the model for both bridges during the initial and the final stage was observed to be high as their correlation coefficients were well above 90% and the percent error being less than 10%. This indicates that an adequate live load response of the bridges was attained, which in turn allowed accurate bridge ratings.

Commercial software such as the BDI system is often considered by bridge owners as an effective tool for load testing and rating existing structures. Strength and stiffness parameters for most of the bridges are often higher than what traditional calculations produce and thereby resulting in increased load ratings with better accuracy. There is a need for identifying this “reserve” strength during the replacement or rehabilitation of the bridges which can result in significant cost savings over a long period of time. BDI load rating tools presented in this paper were found to be effective and accurate for the determination of load response characteristics. Furthermore, diagnostic load testing can also be used to accurately determine the maximum load carrying capacity of bridges. Although it is recognized that developing load ratings through diagnostic testing cost more initially when compared to the load ratings by traditional methods, the long term savings from a bridge’s useful life may offset these costs.

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