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Load Rating Strategies for Bridges With Limited or Missing As-Built Information

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Load rating is the process of determining the safe load-carrying capacity of a bridge; however, when plans and details are insufficient to determine the overall capacity of the structure, alternative methods must be used to infer what the live load capacity is. Two viable methods allowed by the AASHTO Manual for Bridge Evaluation are the commonly used but subjective engineering judgement and the experimentally based proof testing. However, these methods suffer from limitations. Engineering judgement typically is not based on physical phenomena and creates a degree of risk in unconservative estimates or unnecessarily restricts traffic and commerce if estimates are overly conservative. On the contrary, proof testing can cause damage during testing, tends to be expensive, and cannot be extrapolated to future performance.

Thus, the objective of this study was to develop rational engineering approaches for load rating structures within the Virginia Department of Transportation (VDOT) inventory for which limited as-built information is available. The initial phase of the investigation focused on categorizing the VDOT inventory to determine the types of structures that are likely to be missing information necessary for an analytical load rating, which were identified to be short span reinforced concrete slab or T-beam designs. Subsequent phases emphasized two main approaches to load rating: (i) structural identification frameworks based on finite element model updating; and (ii) leveraged vibration response characterization. Both approaches emphasized estimating unknown characteristics of these types of structures for use in a traditional analytical load rating. These unknown parameters include modulus of elasticity and strength of concrete as well as cross-sectional area of steel reinforcement. These estimates can ultimately be used to provide a rational estimate of load ratings.

All approaches were evaluated on two slab and two T-beam structures in varying condition states, which had sufficient plans available, but were treated as having varying degrees of unknown details. The results illustrated that the finite element model updating method generated load ratings that were within 0% to -17% of the load ratings developed according to conventional calculations, with negative differences indicating lower rating factor estimates; and the vibration-based simplified method led to results with a percent difference ranging from 16% to -16%. It was also shown that instrumenting bridges with a limited number of sensors is sufficient for successful implementation of the developed methods. The results from the study have been synthesized into recommendations for VDOT to perform load ratings of structures with insufficient plans or information, with the goal of minimizing the degree and complexity of experimental measurement as well as simplifying the tools for performing the analyses of these structures as much as feasible.

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FINAL REPORT

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ABSTRACT

Load rating is the process of determining the safe load-carrying capacity of a bridge; however, when plans and details are insufficient to determine the overall capacity of the structure, alternative methods must be used to infer what the live load capacity is. Two viable methods allowed by the AASHTO Manual for Bridge Evaluation are the commonly used but subjective engineering judgement and the experimentally based proof testing. However, these methods suffer from limitations. Engineering judgement typically is not based on physical phenomena and creates a degree of risk in unconservative estimates or unnecessarily restricts traffic and commerce if estimates are overly conservative. On the contrary, proof testing can cause damage during testing, tends to be expensive, and cannot be extrapolated to future performance.

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All approaches were evaluated on two slab and two T-beam structures in varying condition states, which had sufficient plans available, but were treated as having varying degrees of unknown details. The results illustrated that the finite element model updating method generated load ratings that were within 0% to -17% of the load ratings developed according to conventional calculations, with negative differences indicating lower rating factor estimates; and the vibration-based simplified method led to results with a percent difference ranging from 16% to -16%. It was also shown that instrumenting bridges with a limited number of sensors is sufficient for successful implementation of the developed methods. The results from the study have been synthesized into recommendations for VDOT to perform load ratings of structures with insufficient plans or information, with the goal of minimizing the degree and complexity of experimental measurement as well as simplifying the tools for performing the analyses of these structures as much as feasible.

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INTRODUCTION

Load rating is a process for temporal condition assessment of a structure and determination of its safe load-carrying capacity. Typically, load ratings are developed in accordance with the rules of structural mechanics using design drawings and details that define the geometry and material properties of the bridge. Figure 1 provides a breakdown of the various components that are used to arrive at the rating factor (RF) for a given bridge (AASHTO 2015). This rating factor represents the fraction of additional live load capacity remaining above the target live load, when removing the effects of dead load.

The information needed to carry out this load rating includes the latest safety inspection report that documents deterioration affecting as-designed structural capacity, prior load rating files, and design plans or as-built drawings of the structure. However, there are cases where the design plans are missing or incomplete due to lack of documentation at the time of construction, improper storage, or the evolution of data management practices. When this information is missing, the determination of the nominal capacity (R) is extremely challenging for certain structure types. For these structures, the Manual for Bridge Evaluation (MBE) provides limited guidance on the process for load rating (AASHTO 2015). The language in the MBE indicates that an inspection by a qualified inspector and evaluation by a qualified engineer may be sufficient to establish an approximate load rating (AASHTO 2015). This guidance does not explicitly state, but does imply, that engineering judgment may be necessary, in which an experienced engineer considers relevant factors, such as the original design live loads, the past performance and current physical condition of the bridge, current loads, and age, to arrive at a judgment-based load rating. Such techniques or guidelines have the potential to overestimate or underestimate the load bearing capacity of bridges without structural plans.

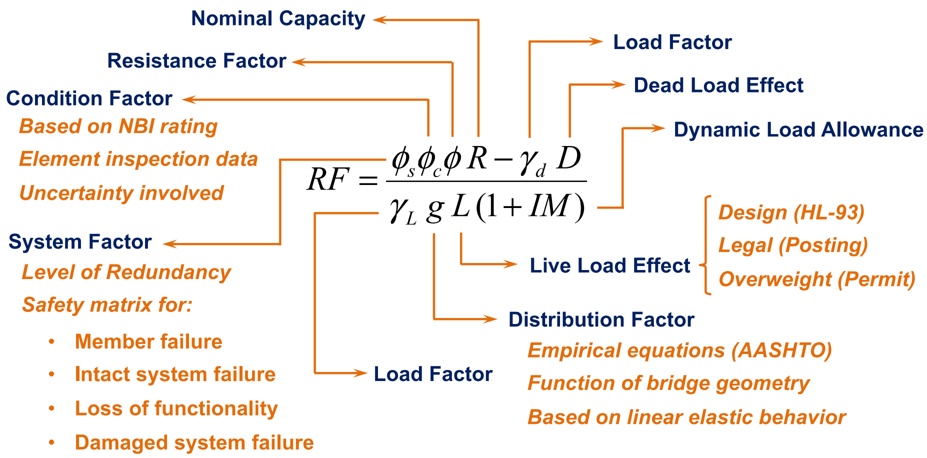


Figure 1 - Components of the Inputs for Load Rating Factor (RF)

A statistical analysis of the National Bridge Inventory (Federal Highway Administration (FHWA) 2015) showed that 21,303 in-service highway bridges in the United States are currently rated solely using engineering judgment. In the majority of these structures, detailed structural drawings or as-built plans do not exist or are insufficient for a routine structural analysis. Therefore, alternative behavior characterization methods are necessary. For these structures, the MBE provides limited guidance on the process for load rating, but this challenge remains one that exists within many states across the country. Within this population of structures, concrete slab bridges make up the largest percentage with over 4,500 (21.1%) structures, with an additional 2,345 (11.0%) bridges having T-beam design. In Virginia, 454 slab bridges and 22 T-beam bridges are rated only using engineering judgement due to lacking sufficient details for conventional analysis methods, according to the Virginia Department of Transportation (VDOT) Br|M (formerly PONTIS) database (FHWA 2015).

Conventional Methods for Load Rating

The MBE presents an analytical and an empirical approach for load rating and sets forth a standard procedure for each one. In addition to these methods for load rating, the MBE (Section 8.4) also provides limited guidance for nondestructive load testing techniques that can be used to better describe in-service bridge behavior (i.e. diagnostic, proof, weigh-in-motion, dynamic response, and vibration testing).

AASHTO Analytical Load Rating

The AASHTO analytical load rating method establishes the capacity of the members based on known design parameters and estimates the dead and live load force effects through structural analysis, which can be done either using simplified methods of analysis (such as empirical equations for load distribution factors) or via refined methods of analysis (such as the finite element method). Once the capacity and dead and live force effects are established, the analytical rating factor (RF_c) for each member can then be established using Eq. (1) given below.

$$RF_c = \frac{C\text{-}A_1 DL}{A_2(LL+IM)} \quad (1)$$

where:

C = member capacity
DL = dead load effect
LL = live load effect
A_1 = dead load factor
A_2 = live load factor
IM = impact factor or dynamic amplification

There are three analytical options available in the MBE, namely the allowable stress rating (ASR), the load factor rating (LFR) and the load and resistance factor rating (LRFR). LRFR is the most current method.

AASHTO Load Rating Through Diagnostic Load Testing

Experimental field testing is another approach that provides a realistic representation of the structural behavior of the system. This method is especially effective in cases of bridges with complicated load distribution behavior and deteriorated or damaged structures. Sections 6.1.4 and 8.3.1 of the MBE identify field testing as an option for cases where the lack of as-built information makes it difficult to establish the make-up of the members or their behavior. Despite the degree of accuracy provided, the application of field testing for load rating is not always feasible because of cost, time, testing requirements, disruptions for the traveling public, and safety. Statistics on the methods used for load rating of highway bridges in the U.S. show an estimated 0.6% of the bridges in the inventory are load-rated using field testing (FHWA 2015). The method proposed in Section 8.8.2 of the MBE serves as an extension of the AASHTO Analytical Load Rating method, whereby results of a diagnostic live load test can be used to update existing load ratings obtained by analytical or numerical methods (RF_c). In this approach, an adjustment factor (k) is used to represent the benefits from the diagnostic load test by comparing the measured test behavior with that predicted by models. This factor is then multiplied by the RF_c to produce an updated rating through diagnostic testing (RF_T) such that:

$$RF_T = RF_c \cdot k \quad (2)$$

The adjustment factor, k, is given by:

$$k = 1 + k_a k_b \tag{3}$$

where k_a is the benefit derived from the load test and is calculated as:

$$k_a = \frac{\varepsilon_c}{\varepsilon_T} - 1 \tag{4}$$

where:

ε_c = the theoretical strain in that member due to the actual test vehicle positioned at the location on the bridge when ε_T was recorded

ε_T = the largest strain in a given member that was measured during a load test

The term k_b accounts for the understanding of the test results and the ability to explain why the observed strains differ from theoretical values. Table 8.8.2.3.1-1 of the MBE (AASHTO 2015) provides values for k_b based on the weight of the test vehicle (unfactored test vehicle effect, T) relative to that of the load rating vehicle (unfactored gross rating load effect, W), as well as whether or not the test results can be extrapolated to overload situations. Note that by incorporating the results of a diagnostic load test, this method presents a more realistic evaluation of load ratings. However, as mentioned above, analytical calculations are still required. Therefore, this method cannot be used directly for bridges with missing or insufficient plans.

As noted within the AASHTO Analytical Load Rating description, finite element analyses (FEA) can be used to provide a refined method of analysis of load effects especially in cases where the analytical load rating method described above is inapplicable, insufficient or results in unsatisfactory ratings.

AASHTO Proof Load Testing

Proof load testing provides an alternative to analytically derived load ratings. As defined by the MBE Section 8.8.3 (AASHTO 2015), proof load testing "proves" the ability of the bridge to carry its full dead load plus some "magnified" live load. This serves as a lower bound on the true strength capacity and hence the load-rating capacity. While proof load testing is a viable option, it does impose a degree of risk for damage to the bridge during testing that must be considered. Proof testing is beyond the scope of this investigation and will not be further discussed; however, readers can review the guidance provided by AASHTO in the MBE regarding proof load testing.

PURPOSE AND SCOPE

The primary objective of this investigation is to evaluate rational engineering methods for load rating structures within the VDOT inventory that are difficult to rate as a result of either limited or missing as-built information. To accomplish this objective, the study was divided into two phases that built upon each other. The initial phase of the investigation focused on categorizing the VDOT inventory to determine which types of structures are likely to be missing information necessary for an analytical load rating. The initial phase also focused on the development of experimental and analytical approaches that could be evaluated on an existing bridge representative of the types of structures without plans or insufficient details. The next phase of the investigation focused on refinement of the proposed approaches and extension to a small population of bridges (three additional bridges) for further evaluation. Collectively, this study focused primarily on two structure types identified based on the characterization of the existing VDOT inventory, but the framework is generic and thus expected to provide similar results with other bridge types. However, the evaluation of load ratings of other bridge types is beyond the scope of this project.

METHODS

As described in the Purpose and Scope section, the investigation was divided into multiple phases. In this section, the objectives of this study were achieved through completion of the following tasks:

1. Conduct a literature review to identify methods used in the transportation community to rate bridges without plans (Task 1).

2. Characterize the existing VDOT inventory with respect to the structures without plans or sufficient details (Task 2).

3. Develop methods of rating for structures that lack information needed to perform an analytical load rating (Task 3).

Literature Review (Task 1)

The research team compiled a literature review to document the state-of-the-practice and state-of-the-art on load rating of bridges without sufficient details. The literature search included not only published articles in academic journals and proceedings but also research reports developed by different state DOTs. As the studies on the bridges without plans were limited, this review extended to articles about field testing-based load rating methods for bridges with plans, as these approaches were viewed as relevant to the development of the proposed new methods. A complete review and understanding of these studies helped the team to identify the needs for further research.

Characterization of VDOT Inventory of Bridges With Limited Information (Task 2)

The initial investigation focused primarily on developing an understanding of VDOT's inventory. Using the AASHTOWare Bridge Management (BrM) database provided by VDOT, the project team interrogated the available information to categorize characteristics and features of the structures within the inventory, such that the population of structures with either limited or missing as-built information could be readily distinguished within the database, since that information was not *specifically* set up as a data field in the database. With 209 data fields in the

BrM database, the primary goal was to determine which features could be used to infer that a particular structure was in the category of bridges with either limited or missing as-built information. The process for feature identification started with the exclusion of structures not deemed relevant to the investigation including culverts, non-highway bridges, bridges not open to traffic, and bridges containing obviously missing or incorrect data. The reduced population was then further filtered using the BrM database fields "IRTYPE," "ORTYPE," and "STRESS_METHOD" to identify those bridges that were either rated by "field evaluation and documented engineering judgment," or having "no rating analysis or evaluation performed", or "assumed." This refined population was considered to cover the majority of bridges with missing or insufficient details. For this objective, the research team explored the BrM database using the defining features and developed descriptive statistics on the VDOT bridge population with limited or missing as-built information.

Development of Alternative Methods for Load Rating Unknown Structures (Task 3)

The project team explored a series of experimental and numerical approaches on four bridges within VDOT's inventory. The described approaches were initially developed and evaluated on one bridge (Phase I), then refined and extended to an additional three bridges (Phase II). The methodologies presented in this section are summarized in their refined form with the field testing components described in a generalized form. All of the bridges evaluated in this study had sufficient plans and other information necessary to perform a traditional analytical load rating, but served as representative structures similar to those bridges without plans that were derived from the characterization portion of this study. With the consent of the project's technical review panel (TRP), the selections included structures in "good" and "fair" condition to allow for condition characteristics to be included in the assessment methods. The other qualification was the suitability for field testing (that is, proximity, accessibility, and low average daily traffic). This strategy allowed for a ground truth reference of the proposed methodologies, which were developed under the assumption that the necessary information for the conventional analyses was unavailable.

Unlike the conventional analytical load rating process, the assumption was made that the actual details of a "known" structure were unknown when conducting the alternate rating methods. For this objective, the selected structures were treated as structures with a degree of uncertainty in their final analyses, which was intended to mimic the class of structures with limited or missing as-built information. The following sections introduce the various methods considered in this study for load rating of bridges without plans. The summaries are intended to provide a basic overview of the process, but supplementary user guides are available to provide comprehensive illustrations and example problems. The user guides are not included in this report, but can be requested from VTRC with appropriate permissions. The developed methods are generalized into two categories, *Finite Element Model Updating Methods* and *Vibration Response Derived Methods*, with sub-variations within these two categories. The following sub-sections describe the developed methods followed by descriptions of the experimental approaches used to inform the methods.

Finite Element Model Updating (FEMU) Based Methods

Building upon the principles outlined in the MBE, analytical and experimental approaches can be combined to create an effective and realistic rating method that aims to leverage the strengths of each one of these approaches while reducing the operational difficulties of load testing. This method relies on the application of structural identification (St-ID) for finite element (FE) model updating based on live load testing, vibration testing, or both. The term St-ID is defined as the process of creating/updating a structural model based on experimental observations/data (Costa et al. 2016; Garcia-Palencia et al. 2015; Jang and Smyth 2017; Polanco et al. 2016; Sanayei et al. 2015). The St-ID framework aims to bridge the gap between model approximation and the real system behavior through improved simulations. One of the subcomponents of St-ID is finite element model updating (FEMU), which as noted requires the development of an initial model that can be updated based on experimental test data. An updated model is expected to reflect the measured data better than the initial model as a result of refinements to model uncertainties (e.g. boundary/loading conditions and constitutive properties).

In this investigation, initial models were developed in ABAQUS, a robust commercially available finite element software package (ABAQUS 2016); however, comparable FEA models could be developed in other packages. In this study, ABAQUS allowed for the development of an interface with MATLAB (MATLAB 2016a), which facilitated the integration of an automatic iterative parameter optimization algorithm. The optimization algorithm systematically searches a parameter space by revising input variables to achieve a solution match, in this work minimizing the difference between the model and experimental results by iterating on uncertain parameters. The optimization algorithm developed in this investigation incorporated the features of a genetic algorithm and a gradient-based scheme to iterate on the unknown parameters (Dizaji et al. 2016). A genetic algorithm (GA) is a global optimization method and provides an effective means to solve for unknowns whenever there are more than two unknown parameters in the problem. Therefore, using a GA, the chance of getting trapped in local minimums decreases dramatically. However, the proposed GA has a relatively high computational cost, which results in a long solution time. In order to reduce the computational cost, this method can be combined with a gradient-based algorithm. While other optimization algorithms could be used, this approach was selected because of the efficiency in localizing a global minimum amongst a large dataset (Dizaji et al. 2017).

In this investigation, boundary restraints were selected as key unknown parameters within the St-ID scheme as these values have time-dependent and condition-driven characteristics. In addition to identifying unknown boundary properties, the updating process allowed for identification of some of the key parameters that are needed to determine load rating, including the area of steel, A_s, the elastic modulus of concrete, E_c, and the concrete compressive strength, f'_c. A schematic of the overall model updating framework used is shown in Figure 2, and can be described as a process that minimizes the difference between the response of the bridge obtained in an FE model and those obtained from experimental testing.

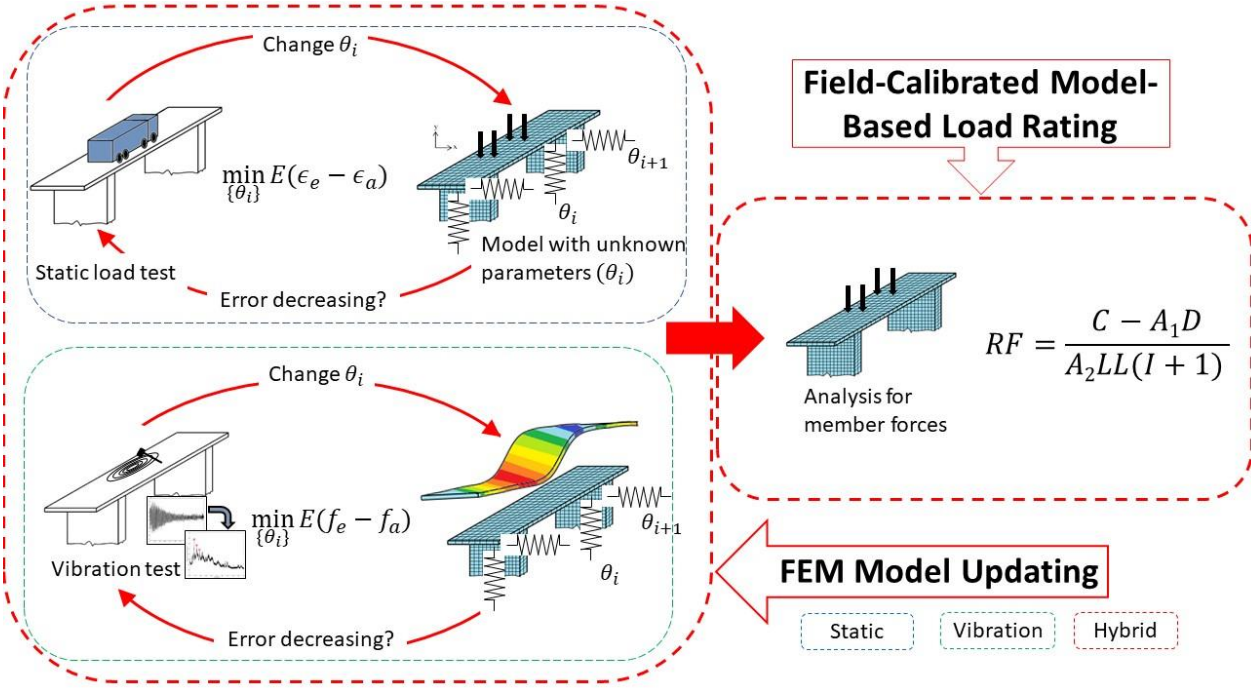


Figure 2 - Schematic of Load Rating Using FEMU Based on Field Test Data

In order to determine the unknown structural parameters, optimization was performed to minimize the difference between calculated data from a numerical model and a set of experimental data. This research developed an FE model with parameterized geometry and boundary conditions defined to allow the model to replicate the real conditions and well-known geometry of the test bridges as closely as possible once updated. The results from the FE model were compared with experimental data collected from sensors on a given bridge and the objective functions were evaluated for convergence (Ribeiro et al. 2012; Zapico et al. 2003). Note that sensors such as accelerometers, string potentiometers, tiltmeters and strain gauges were used for collecting experimental data in this study. The parameters were then iteratively updated in an optimization process until convergence. Three different finite element model updating scenarios were evaluated and are described in the following sub-sections:

1. Discrete Static Finite Element Model Updating (FEMU-S)

2. Discrete Dynamic Finite Element Model Updating (FEMU-D)

3. Discrete Hybrid Finite Element Model Updating (FEMU-H)

The sensor types used and their corresponding generalized objective function are summarized in Table 1 and further details are given below. However, it should be noted that in all of the FEMU methods, the optimization processes aimed to solve for the critical unknowns needed to perform the load rating, which for a concrete structure generally consisted of area, location, and distribution of the reinforcing steel, as well as the compressive strength and elastic modulus of the concrete.

Table 1 – General Form of Objective Functions for FEMU Methods

FEMU Type	Objective Function
FEMU-S	$F(E_c, A_s, K_{\theta x}, K_{\theta z}) = \sum_i \frac{\left|\varepsilon_i^{Measure} - \varepsilon_i^{FEM}\right|}{\varepsilon_i^{Measure}} + \sum_i \frac{\left|\delta_i^{Measure} - \delta_i^{FEM}\right|}{\delta_i^{Measure}} + \sum_i \frac{\left|\theta_i^{Measure} - \theta_i^{FEM}\right|}{\theta_i^{Measure}}$
FEMU-D	$F(E_c, A_s, K_{\theta x}, K_{\theta z}) = \sum_i \frac{\left|\omega_i^{Measure} - \omega_i^{FEM}\right|}{\omega_i^{Measure}}$
FEMU-H	$F(E_c, A_s, K_{\theta x}, K_{\theta z}) = \sum_i \frac{\left|\varepsilon_i^{Measure} - \varepsilon_i^{FEM}\right|}{\varepsilon_i^{Measure}} + \sum_i \frac{\left|\delta_i^{Measure} - \delta_i^{FEM}\right|}{\delta_i^{Measure}} + \sum_i \frac{\left|\theta_i^{Measure} - \theta_i^{FEM}\right|}{\theta_i^{Measure}} + \sum_i \frac{\left|\omega_i^{Measure} - \omega_i^{FEM}\right|}{\omega_i^{Measure}}$

In Table 1, E_c and A_s were selected as unknown parameters in the optimization scheme along with $K_{\theta x}$, $K_{\theta z}$ which are restraint stiffness of the rotational springs at the support locations. $\varepsilon_i^{Measure}$ is the longitudinal measured strain obtained from strain gage sensors installed on the bridge in the location shown with index *i* and ε_i^{FEM} is the longitudinal numerical strain obtained from the finite element model of the bridge in the location shown with index i. $\delta_i^{Measure}$ is the measured vertical displacement obtained from string potentiometer sensors installed on the bridge in the location shown with index *i* and δ_i^{FEM} is the numerical vertical displacement obtained from finite element model of the bridge in the location shown with index i. $\theta_i^{Measure}$ is the measured rotation obtained from tiltmeter sensors installed on the bridge in the location shown with index *i* and θ_i^{FEM} is the numerical rotation obtained from finite element model of the bridge in the location shown with index i. $\omega_i^{Measure}$ is the derived modal frequencies obtained from accelerometer sensors installed on the bridge in the location shown with index *i* and ω_i^{FEM} is the numerical modal frequencies obtained from finite element model of the bridge in the location shown with index *i*. Some of these objective function's parameters are shown in Figure 3 schematically.

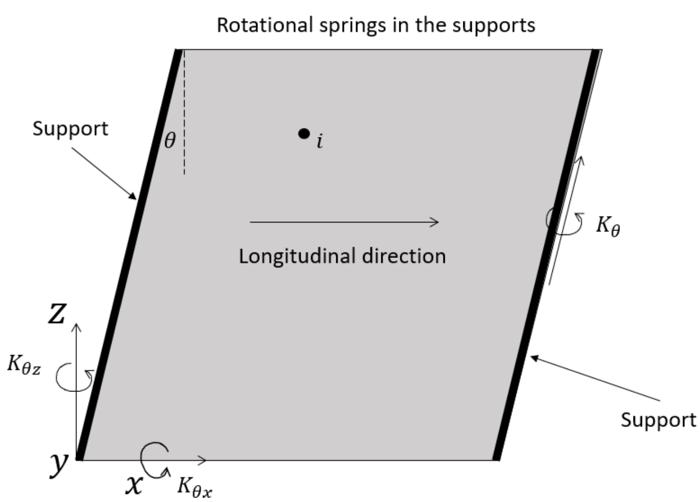


Figure 3 - Parameters of Objective Function Shown Schematically

Discrete Static Finite Element Model Updating (FEMU-S)

Only mechanical sensors were used in model updating. This type of measurement would be typical of a traditional live load test. Time series data was post-processed and the sensor data for the critical loading configuration (peak measured response) was used in the updating processes.

1. An approximate finite element model of a bridge was first developed. This model was based on readily observable geometric parameters, reasonable estimates of unknown material properties, and assumptions regarding internal structural parameters, such as cross-sectional area and location of the steel reinforcement.

2. The model was then updated based on measured mechanical static data. Measured static responses such as strain, deflection, and rotation were used as inputs to update the model of the bridge.

3. An inverse method based on an optimization process was then applied to find unknown structural properties of the structure such as stiffness and boundary conditions. The Young's Modulus (E_c) of the concrete and area of steel (A_s) of rebar were selected as unknown parameters in the optimization scheme along with restraint stiffness at the support locations.

4. Based on the identified concrete elastic modulus (E_c), area of steel of rebar and an assumption of dynamic amplification (*IM*), the capacity of the structure (*C*) was calculated

5. Finally, the rating factor (*RF*) was obtained using Equation 1.

Discrete Dynamic Finite Element Model Updating (FEMU-D):

The first three identified natural frequencies of the bridge (derived from vibration testing) were used in model updating. This type of measurement would be typical of traditional ambient vibration or an impact excitation test. This scenario was similar to the previous method, except that the finite element model was updated using an array of sensors used to record the vibration response of a bridge and extract the modal properties of the structure. In this method, an output-only modal identification technique called the Enhanced Frequency Domain Decomposition (EFDD) was used to obtain modal characteristics of bridge structures (i.e. first three natural frequencies). In this method, the single degree of freedom (SDOF) power density functions are transferred back into the time domain, and the natural frequencies are obtained by calculating the number of zero-crossings as a function of time. In addition, the damping ratio can be estimated from the logarithmic envelope of the corresponding SDOF correlation function using the logarithmic decrement method. The EFDD technique allows for the extraction of the natural frequency and damping characteristics of a particular mode by processing the collected time domain acceleration data in frequency domain. Additional details on the EFDD technique can be found in Brincker (2001). In this study, the results for the system identification techniques were obtained from the ARTeMIS Modal Pro software (ARTeMIS Modal Pro 2016). Similar to the

FEMU-S approach, the experimental data was used to update a model and estimate the rating factor:

1. An approximate finite element model of a bridge was first developed.

2. The model was then updated based on measured mechanical static data and dynamic data. Measured natural frequencies for the first three modes were used as inputs to update the model of the bridge.

3. An inverse method based on an optimization process was then applied to find unknown structural properties (E_c and A_s) of the structure such as stiffness and boundary conditions.

4. Based on the identified E_c and A_s and an assumption of dynamic amplification (*IM*), the capacity of the structure (*C*) was calculated

5. Finally, the rating factor (*RF*) was obtained using Equation 1.

Discrete Hybrid Finite Element Model Updating (FEMU-H):

Both mechanical and accelerometer sensors were used in model updating. This method designated as the hybrid approach, combines the FEMU-S and FEMU-D approaches to simultaneously include both global and local behavior characteristics of the bridge. Time series data was post-processed and the sensor data for the critical loading configuration (peak measured response) for the mechanical response along with the first three natural frequencies derived from the accelerometers using the EFDD were used in the updating processes. Similar to the FEMU-S and FEMU-D approaches, the experimental data was used to update a model and estimate the rating factor:

1. An approximate finite element model of a bridge was first developed.

2. The model was then updated based on measured dynamic data. Measured static responses such as strain, deflection, and rotation along with the natural frequencies for the first three modes were used as inputs to update the model of the bridge.

3. An inverse method based on an optimization process was then applied to find unknown structural properties (E_c and A_s) of the structure such as stiffness and boundary conditions.

4. Based on the identified E_c and A_s and an assumption of dynamic amplification (*IM*), the capacity of the structure (*C*) was calculated

5. Finally, the rating factor (*RF*) was obtained using Equation 1.

Vibration Response Derived Methods

As highlighted in the MBE (section 8.4) dynamic testing can be used to characterize a bridge's dynamic characteristics including frequencies of vibration, mode shapes, and damping characteristics. These dynamic characteristics can be related directly to the mechanical response of a structure through the equation of motion (Craig and Kurdila 2006), thus providing a pathway to link measured vibration response to structural behavior. The proposed methods rely on establishing a relationship between the dynamic characteristics of a bridge and flexural rigidity to determine the capacity of the bridge. These *Vibration Response Derived* methods leverage classical plate analysis and machine learning in the approach development, but ultimately only require limited dynamic measurements and geometric descriptions of the bridge to arrive at estimates of the load rating.

Vibration-based Simplified Method (VSM) for Load Rating

This method first approximates the RC slab and T-beam bridges as a plate-like or stiffened plate-like structures and employs the following analytical expression that relates the *i*-th angular natural frequency, ω_i, of a continuous plate-like structure to the flexural rigidity, D, of the plate:

$$\omega_i^2 = \lambda_i^2 \frac{D}{m}, \quad i = 1, 2, \ldots \tag{5}$$

where

λ_i = non-dimensional frequency parameter associated to the *i*-th vibration mode
m = mass per unit area of the plate and given as ρh
ρ = density of material
h = thickness of plate

The flexural rigidity (D) of RC slab and T-beam bridges without plans were estimated using Equation 5. To this end, λ_i and ω_i needed to be determined. The natural frequencies of the bridge (ω_i) were determined through vibration testing of the bridge with either ambient or impact excitation.

The non-dimensional frequency parameter λ_i for plate structures with simple boundary conditions can be computed using analytical equations or look-up tables that are readily available in the literature. However, these analytical expressions or tables are not suitable for skewed structures with edge stiffening, such as that described by a skewed slab bridge with rigid concrete parapets. In this work a machine learning approach was employed to estimate this parameter. A parametric analysis was performed using the finite element method for a bridge with different geometric characteristics and a rigid parapet, with natural frequency as the result from the finite element models. Using the model results (ω_i) and geometric characteristics of the model as input, an artificial neural network (ANN) was created to estimate the non-dimensional frequency parameter (λ_i) for structures with edge stiffening and possible skew angles. More details about the development of the ANN can be found in the User Guide. The developed ANN provides the parameter λ_i when the geometric properties and natural frequencies of a bridge obtained from vibration testing were fed to the ANN as inputs.

Once the ω_i (from vibration testing) and λ_i (using the developed ANN) were obtained, the flexural rigidity D of a bridge was determined from Equation 5. Noting that the flexural rigidity D of a plate-like structure with a unit width is given as (Timoshenko and Woinowsky-Krieger 2009)

$$D = \frac{EI_g}{(1-\nu^2)} \tag{6}$$

where

E = elastic modulus of the plate's material
I_g = moment of inertia of the cross-section
ν = Poisson's ratio of the plate's material

This result leads to calculating the slab or beam stiffness, EI_g, using the following equation:

$$EI_g = D\left(1-\nu^2\right) \tag{7}$$

The composite cross-section can be transformed into an equivalent cross-section with only E_c, and the moment of inertia of the transformed cross-section, I_t. Since E_cI_t must be equal to EI_g, the elastic modulus of concrete E_c can be determined from the equation below as:

$$E_c = \frac{EI_g}{I_t} \tag{8}$$

and using Equation (7), we obtain:

$$E_c = \frac{D\left(1-\nu^2\right)}{I_t} \tag{9}$$

After the elastic modulus of concrete was obtained, the compressive strength of concrete f_c was estimated by available relationships between the elastic modulus and ultimate compressive strength. Here, the following relationship was used to derive the ultimate compressive strength of concrete as (AASHTO 2014):

$$E_c = 33{,}000\rho_c{}^{1.5}\sqrt{f'_c} \tag{10}$$

where E_c should be provided in *ksi*, and the compressive strength of concrete represents the strength at the current age of bridge in *ksi*.

Next, the cross-sectional area of the internal reinforcing steel was estimated using strain data measured from a quasi-static live load testing coupled with a gradient-based optimization approach. The amplitude of a strain measurement in the RC slab or beam recorded during a quasi-static test depends on the cross-section geometry, area of steel reinforcement, elastic modulus of concrete, and the bending moment developed at the location of a sensor under the applied load. Since the cross-section and the elastic modulus of concrete were determined as described in previous steps, the strain was expressed as a function of two unknown parameters, namely, area of steel reinforcement and bending moment in the cross-section. Here, the area of steel reinforcement and the bending moment were determined by minimizing an objective function, described in detail in the User Guide, using a gradient-based optimization algorithm.

The yield strength of unknown reinforcing steel used in a concrete bridge was estimated by considering the era of bridge construction, following guidance in the MBE when structural details are unknown. These structural and material properties were then used to determine load effects and ultimately the bridge's capacity. Once the capacity was obtained, the *RF* was calculated. A more detailed description of the methodology is also available in (Bagheri et al. 2017; Bagheri et al. 2018).

The following discussion presents a summary of the steps that need to be followed for load rating of an RC slab and T-beam bridges without plans using the VSM load rating method.

1. First, the geometric characteristics of the bridge such as span length, width, and slab thickness, girder geometry are determined.

2. Vibration testing is conducted to collect acceleration data of the bridge and determine the modal properties of the bridge using the measured acceleration data.

3. In addition, the strain data at a selected point at the mid-span of the bridge is collected.

4. The non-dimensional frequency parameter λ_i is evaluated using the ANN.

5. The flexural rigidity of the bridge is determined from Equation (5).

6. The elastic modulus and compressive strength of concrete are estimated from Equations (9) and (10), respectively.

7. The yield strength is estimated by considering the era of bridge construction and the cross-sectional area of reinforcing steel is determined by using the measured strain data.

8. Based on the identified concrete elastic modulus (E_c), area of steel of rebar and an assumption of dynamic amplification (*IM*), the capacity of the structure (*C*) is calculated.

9. Finally, the rating factor (*RF*) was obtained using Equation 1.

Differential Mass Vibration Based Method for Load Rating

This method is an extension of the aforementioned *Vibration-based Simplified Method* and has potential for arriving at an estimate of the flexural stiffness/rigidity of a structure using only vibration data of a bridge under differential mass configurations. This method follows the same steps of the VSM method except for the fact that it leverages the concept of using an added mass during vibration testing to arrive at an estimate of flexural rigidity of the bridge. The method required the vibration testing to be conducted twice. The first test was conducted to determine the natural frequencies of the bridge. The second test was conducted while two axle dump trucks provided by VDOT with gross weights of 30,575 lb (Truck 1) and 29,600 lb (Truck 2) were located at each lane at the bridge's midspan, and the natural frequencies were measured again. The flexural rigidity of the bridge could then be estimated by relating the shift in natural frequencies of the bridge to the mass and stiffness of the bridge. A similar approach was presented elsewhere in the literature (Samali 2003) and provided a basis for improved estimates of bridge stiffness/rigidity.

Field Testing of Selected Bridges for Load Rating

Each of the proposed methods requires the collection of response data from the bridge of interest. The required response data includes both mechanical sensor measurements derived from live load testing and dynamic sensor measurements derived from either ambient vibration or impact excitation. To collect data, a series of bridges were selected from VDOTs inventory for field testing (live load and vibration). Results from this inventory assessment guided the bridge selection process for the load testing activities described in this sub-section. The bridges for the load testing program were selected with consideration of the typical characteristics of bridges within these classes, proximity to Charlottesville, accessibility, and traffic. The following sub-sections describe the general instrumentation plan and testing processes for the four bridges studied; however, each bridge had specific testing protocols that are not described in detail in this report as the methodology for using available test data remains the focus of the research. Data for each bridge can be provided upon request.

Instrumentation

All instrumentation and data acquisition equipment were from Bridge Diagnostics, Inc. (BDI), and were individual sensors physically connected to four-channel nodes, which in turn interfaced wirelessly with a base station/data acquisition unit. Representative images of the instrumentation are shown in Figure 4 for illustrative purposes. For each bridge, the instrumentation was separated into mechanical measurement sensors and vibration sensors, but it should be noted that all of the sensors were mounted concurrently, except in cases where sensors were reconfigured for staged measurements. In addition, all testing was conducted in collaboration with the Virginia Transportation Research Council (VTRC) and VDOT, with VDOT providing the necessary lane control and trucks used for testing.

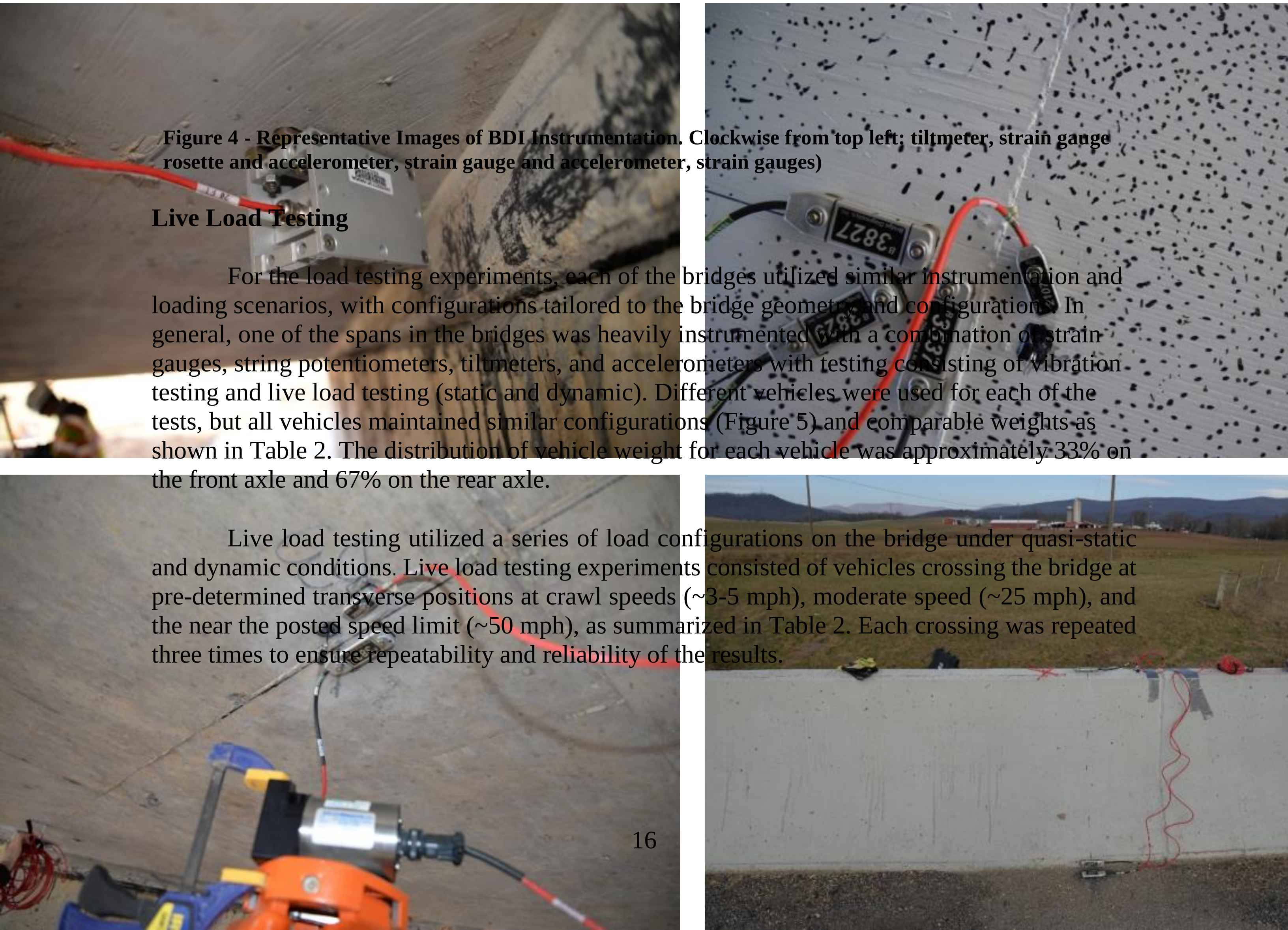


Figure 4 - Representative Images of BDI Instrumentation. Clockwise from top left; tiltmeter, strain gauge rosette and accelerometer, strain gauge and accelerometer, strain gauges)

Live Load Testing

For the load testing experiments, each of the bridges utilized similar instrumentation and loading scenarios, with configurations tailored to the bridge geometry and configurations. In general, one of the spans in the bridges was heavily instrumented with a combination of strain gauges, string potentiometers, tiltmeters, and accelerometers with testing consisting of vibration testing and live load testing (static and dynamic). Different vehicles were used for each of the tests, but all vehicles maintained similar configurations (Figure 5) and comparable weights as shown in Table 2. The distribution of vehicle weight for each vehicle was approximately 33% on the front axle and 67% on the rear axle.

Live load testing utilized a series of load configurations on the bridge under quasi-static and dynamic conditions. Live load testing experiments consisted of vehicles crossing the bridge at pre-determined transverse positions at crawl speeds (~3-5 mph), moderate speed (~25 mph), and the near the posted speed limit (~50 mph), as summarized in Table 2. Each crossing was repeated three times to ensure repeatability and reliability of the results.

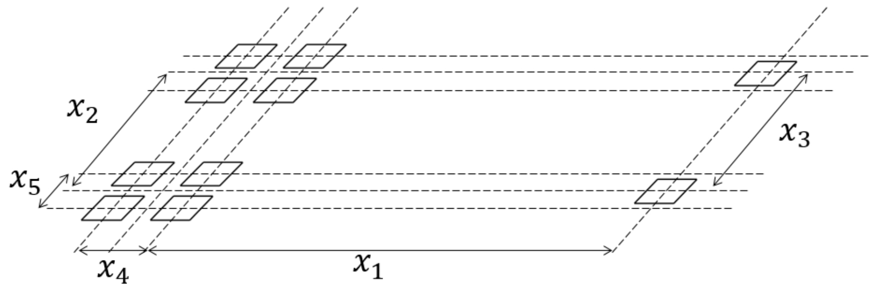


Figure 5 – Generalized Truck Configuration of Test Trucks. x_1= distance between front wheel axle and closer rear wheel axle; x_2 = distance between rear wheel tires; x_3 = distance between front wheel tires; x_4 = distance between rear wheel axles; x_5 = distance between rear tires on one side of the wheel.

Table 2 – Test Truck Details for Live Load Testing

	War Branch	**Smacks Creek**	**Flat Creek**	**Brattons Creek**
Truck 1				
Gross Weight (lb)	30,575	44,360	44,360	52,620
x_1 (ft)	12.91	13.5	13.5	13.5
x_2 (ft)	6.08	6	6	6
x_3 (ft)	6.91	6	6	6
x_4 (ft)	4	4	4	4
x_5 (ft)	1.12	1.12	1.12	1.12
Truck 2				
Gross Weight (lb)	29,600	45,380	45,380	52,680
x_1 (ft)	12.91	13.5	13.5	13.5
x_2 (ft)	6.08	6	6	6
x_3 (ft)	6.91	6	6	6
x_4 (ft)	4	4	4	4
x_5 (ft)	1.12	1.12	1.12	1.12

Note: x_1-x_5 are defined in Figure 5.

Vibration Testing

Vibration testing consisted of a series of experiments with excitation provided separately by ambient loading (wind and normal traffic) and an impact hammer. Figure 6 provides a demonstration of some of the components from the vibration testing. It should also be noted that the research team initially also explored the use of an electro-dynamic shaker (APS Electro-Seis shaker with a peak force of 100 lbf and operating frequency range up to 200 Hz); however, the results derived from the other two excitation methods proved to be more effective compared to those obtained from a shaker experiment due to the complexity of the shaker setup and operation. Thus, the shaker was not used in subsequent tests. Note that the primary goal of the vibration testing was to identify natural frequencies of the tested bridges; conducting either ambient testing or impact hammer testing provide an effective means to collect these data. In this project, both vibration testing methods were implemented to compare their results and provide recommendations for future implementation. For all of the vibration experiments, the tests were repeated twice; however, some of the early testing was executed in two phases in order to allow sensors to be relocated to the second half of the span once data had been collected from the first half of the span. This relocation was necessary due to the limited number of acceleration sensors available to the project team. However, for the latter tests, the spatial distribution of sensors was reduced in order to improve testing efficiency. In all tests, uniaxial accelerometers with a measuring range of ±5g were used to collect the vibration response. For the impact excitation, a large impulse sledge hammer with a force capacity range of ±5000 lbf was used. All data was collected with a sampling frequency of 500 Hz.

(a) **(b)**

Figure 6 – Representation of Excitation Methods Used During Vibration Testing: (a) impact hammer excitation; (b) ambient excitation

RESULTS

Literature Review of Existing Strategies (Task 1)

Limited studies have been conducted on load rating of bridges with limited or missing as-built information, especially of reinforced concrete bridges where the complexity and number of unknown structural parameters increased (Alipour et al. 2016). The literature review presented in this section highlights relevant works related to load rating of bridges with limited or missing as-built information, estimating load carrying capacity, load rating through static testing, and load rating through dynamic testing. These areas represent topics that inform the investigation and highlight the state of practice.

Load Rating of Bridges With Limited or Missing As-Built Information

Shenton et al. (2007) and Huang and Shenton (2010) investigated a method for load rating of RC bridges without as-built information. The method used strain or displacement data to estimate the unknown area of reinforcing steel in a bridge. This estimated reinforcing steel area was then used to determine the bridge’s capacity based on a sectional analysis procedure. To utilize the proposed approach, strain sensors needed to be mounted on the top and bottom surface of a deck to quantify the internal stain distribution of the bridge’s cross-section. This approach also required knowledge of the relevant mechanical properties for the selected materials.

Subedi (2016) used non-destructive technologies including a concrete rebound (Schmidt) hammer and covermeter to estimate concrete strength and rebar details, respectively, and compared the results with existing plans for a flat slab bridge. A finite element model of the structure was then built and the bridge model was loaded with the state legal truck. The load was increased such that the model reached AASHTO's maximum serviceability deflection. The ratio of the corresponding allowable load to the original design load was then calculated designed as a rating factor. It should be noted that the method did not include the effects of field factors or bridge condition in load rating, and the use of deflection as the limit state does not correspond with the actual allowable capacity for load rating calculation.

Aguilar et al. (2015) presented a four-step load rating procedure for prestressed concrete bridges without plans using proof load test results. Their proposed procedure included estimating the number and eccentricity of strands using Magnel diagrams and typical details at the time of construction. Material properties were estimated based on age using the MBE. A rebar detection system was used to detect location and size of the reinforcement and concrete cover as a verification of the estimates. A diagnostic load test was then conducted to determine the critical transverse truck path and to estimate load effects under the trucks. Finally, a proof load test was performed with a target proof load based on the MBE and the results were used to determine final load ratings.

Taylor et al. (2011) presented a method to load rate the bridges with small to medium simple span bridges in Larimer County, CO that are currently load rated solely based on visual inspections. The objective of the project was to load rate these bridges using structural analysis with very little -to -no information available related to their design. Due to the limitations of the plan, a basic structural analysis was performed using a program developed for Colorado Department of Transportation with rating-conservative assumptions in order to determine the capacities of the bridges.

Briones (2018) proposed a general procedure for load rating bridges without plans. The procedure had four critical parts that included bridge characterization, bridge database, field survey and inspection, and bridge load rating. The bridge characterization focused on the identification of critical bridge information needed to conduct the load rating and evaluation. The bridge database provided guidelines and recommendations for obtaining the unknown information discerned from the bridge characterization. A field survey was used to supplement the unknown bridge information through collecting field measurements. The bridge load rating procedure was divided into three bridge evaluation options. The first option was a simplified structural analysis of the bridge. The second option was a refined structural analysis, e.g., FEA, and the third option was load testing. Recommendations and guidelines for each type of bridge evaluation were presented in this study. The load rating of bridges without plans was successfully completed by following the developed general procedure for two case study bridges without plans.

Determining Load Carrying Capacity

As noted, the challenge of load rating structures without sufficient design details is not exclusive to Virginia. The literature review did not produce significant references related specifically to this challenge, but a number of owners/agencies, have established programs for determining the load-carrying capacity of multiple bridges in a timely fashion. Selected studies pertinent to this investigation are summarized in this section.

With 2,440 T-beam bridges in its inventory as of 2001, the Pennsylvania Department of Transportation also had the largest number of structurally deficient and functionally obsolete T-beam structures amongst all of the state transportation agencies (Catbas 2001). Instead of considering each individual bridge, Drexel University researchers endeavored to establish load capacities by managing bridges as “fleets” of similar structures and establishing the critical parameters for load capacity and failure mode despite the many different aspects that characterize any given structure. The researchers conducted a statistical analysis to determine the relationship between the bridges that had both extensive modeling and field evaluation and the rest of the T-beam population. To determine the load rating, the research team generated three-dimensional (3D) finite element (FE) models of a statistically representative sample of T-beam structures in Pennsylvania. The researchers suggested that when properly calibrated and validated by field data, FE models can better simulate bridge behavior compared to idealized simple beam or grillage models because the simpler models do not adequately account for load distribution and secondary elements that can increase the load capacity. The FE modeling did require a certain level of expertise and a sufficient amount of field test data for calibration; otherwise, the results would be expected to have a lower degree of confidence compared to the idealized methods. The models used for specific bridges needed to be generalized as compared to the actual structural plans; thus, the additional strength provided by secondary elements was ignored and the supports were assumed to be pin-roller supports. Additional testing included collecting concrete core samples and impact modal results from impact-hammer and falling weight deflectometers (FWD). Using the data, the researchers determined the natural frequency of a given bridge and its mode shapes, critical concrete and steel strains, and maximum deflections for various truck loads and positions. These analyses were then used to update the FE models, with restraint conditions shown to have the greatest impact on the model predictions. Final results from the study revealed that the ~~actual~~ computed load capacity using the calibrated FE model was at least twice the load rating calculated using AASHTO’s guidelines, primarily due to modified load distribution factors associated with the refined analysis methods used. The conclusions from the research indicated that the load rating of Pennsylvania’s T-beam bridges could be increased anywhere from 10% to 55%, depending on the geometry of the bridge.

Researchers from the University of Delaware (Chajes 2000) collaborated with engineers from the Delaware Department of Transportation to develop a program through diagnostic field testing and in-service monitoring to evaluate posted structures — those bridges for which permit load vehicles were restricted from crossing. The in-service monitoring system was analogous to a weigh-in-motion system that recorded peak live load strains during a given loading event over a period of several weeks or months. These measurements provided a statistical measure of the level of stress observed in a specific bridge, as well as the average daily number of loading events that occurred. Although the weights of the vehicles on the bridge were unknown and the amount of instrumentation was limited, the research team used the data from the monitoring system to evaluate the load-carrying capacity. However, their inventory load rating was based on the observation that very few vehicles resulted in live-load stresses that were greater than a certain stress level during an 11-day monitoring period.

West Virginia had over 600 short-to-medium span bridges that were a part of its Coal Resource Transportation System (Zhou 2012). Many of these structures were reinforced concrete slabs, filled arches, or T-beam designs constructed in the 1920’s for which plans were not available. A majority of the bridges were posted and were deemed structurally deficient. Researchers tested one such reinforced concrete T-beam bridge that had three simply-supported spans totaling 142 ft and had an 18° skew. At the time of testing, the deck exhibited substantial cracking at the piers and the abutments, while the beams had several flexural and shear cracks. For the substructure, there was some major spalling at the piers, pier caps, and beam seats. Researchers used field measurements and material samples, including reinforcement and concrete cores, to generate an FE model prior to the load test. For the load test, one span was extensively instrumented. For the live load, the test used a proof loading of 90 tons per lane, as directed by the *MBE*, partly for the purposes of evaluating a forced vibration method called multi-reference impact testing (MRIT). The MRIT results were used to develop modal data for calculating the modal flexibility and displacement profile. To automate the FE model updating process, researchers developed a coupling between the FE analysis package with MATLAB. MRIT results were interpolated between measurement locations to enable the truck locations and modal flexibility to be correlated with measured displacements, as the nodes within the FE model did not match the location of the truck tires. For this approach, a degree of expertise was needed to perform such analyses and the research team noted that this procedure may not be feasible for any decision that requires greater than an 80% degree of confidence in the modeled response. Additionally, the researchers noted that they would not have been able to recommend removing the posting for the bridge in question had they not carried out a proof-level load test using extensive instrumentation. On the other hand, the authors noted that these recommendations would also not have been possible without the finite element analysis component. Thus, their findings pointed to the need for integration of the experimental, heuristic, and modeling knowledge base, as well as knowledge regarding risk, life cycle costs, and sustainability.

Researchers contracting for the Ohio Department of Transportation developed a simplified aid in rating reinforced concrete slabs after testing 20 bridges with spans ranging from 10 ft to 22 ft, widths between 22 ft and 42 ft, skew angles ranging from 0° to 44°, and slab thicknesses that varied from 9 in to 27 in (Eitel 2002). Data from these tests established a database from which a proper load-based rating methodology could be designated and confirmed. The researcher determined that a governing property for structural behavior of short-span reinforced concrete slab bridges was the effective width of the slab. Another primary factor was the span-to-depth ratio of the slab, which largely determined the live load flexibility coefficient. In the investigation, the research team assumed that the reinforcement ratio was set at 0.01 and the compressive strength of the concrete was fixed at 3 ksi, while the tensile strength of the reinforcement was 40 ksi. Using the finite element method, the researchers developed a series of graphs that would enable an engineer to select both the live load flexibility coefficient and the live load versus dead load effective width ratio when performing the load rating of a structure of this type.

Load Rating Through Static Testing

Field tests results of old bridges show that there is a considerable reserve capacity in terms of strength in most of the bridges that is not justified by the rating procedure within the standards of AASHTO which classify them as structural deficient. Azizinamini et al. (1994) outlined an experiment of aged reinforced concrete bridges both at service and ultimate levels in order to rate them more realistically. To accomplish these objectives, six concrete slab bridges were tested under the selected weights of truck loads so that the bridge responses would be confined to the elastic regime (service load tests). Also, a five-span reinforced concrete bridge built in 1938 was tested destructively, which was performed by applying loads that simulated two trucks side by side on the structure. Experimental test results show that the reinforced slab bridges have much higher strengths than indicated by AASHTO rating procedures.

Chajes et al. (1997) conducted an experimental load rating of a posted, three-span, slab and steel-girder-and-slab bridge. Each span of the bridge consisted of a cross section of nine non-composite steel girders, with the outer girders spaced 1.37 m apart and the interior girders spaced 1.52 m on center. They conducted a load diagnostic test and found that the girders act compositely with the concrete deck and a high restraint observed at the supports. Along with the diagnostic test, a predetermined load was placed at several different locations along the bridge and the bridge response was measured. The measured response was then used to develop a numerical model of the bridge. This numerical model was employed to determine the maximum allowable load by applying the load incrementally until a target load was attained or a predetermined limited state was exceeded. The results indicated that the bridge's load carrying capacity may be substantially higher than the current load levels indicated and suggested that the posting levels on the bridge may be unnecessary.

Cai and Shahawy (2004) conducted a load test on six prestressed concrete bridges with different geometric characteristics to evaluate analytical methodologies for load rating, which were shown to be unreliable (Cai et al. 1999). The main objective was to compare the results from the measurements obtained from their study with AASHTO codes specifications and with those ratings predicted using finite element analysis. The comparison showed a notable difference between the analytical and experimental due to the effects of several factors. However, to examine these effects, the authors included some field factors in their finite element models which in turn had a larger effect on the maximum strain than on the load distribution factor. Parametric studies on the effects of the components of the bridges were also carried out in these studies to assess how the distribution and maximum strain were affected.

Turer and Shahrooz (2011) presented an investigation of the use of 2D grid models for field-calibrated model based load rating of concrete deck on steel stringer bridges. The authors began with a review of the concept and the calculation of load rating and then discussed three different levels of analytical modeling for bridge load rating; namely the 1D line-based, the 2D grid-based and the 3D finite element models. The main hypothesis in this work was that 2D grid models are an efficient tool for modeling concrete deck on steel stringer bridges which will then be used for model calibration against bridge tests and the calibrated models can be finally used for load rating. The 2D grid model used in this paper employed linear elastic beam elements configured as a grid simulating the entire superstructure and the deck. As to their model calibration, the researchers used a code written in Matlab based on an automatic updating algorithm which performs structural analysis and objective function optimization. As to the response variable used for updating, they used both modal data (frequencies, modal assurance criteria (MAC) and order of modes) and static deformations (BGCI-Bridge Girder Condition Index). Therefore, their objective (or error) function was a sum of normalized strain error, frequency error, MAC error and mode order errors. The optimization was performed in a step-by-step and staged manner in which similar parameters were grouped and treated as one parameter so as to find an initial approximate solution and then the group was divided into subgroups of parameters and the process was repeated until convergence. The proposed method was then applied on an actual three-span four-lane concrete deck on a steel stringer bridge built in 1968. Load ratings were calculated using the allowable stress rating (ASR) and the load factor rating (LFR). The authors concluded that the updating scheme had been efficient and successful, that 2D grid models provided results close to 3D models and that transverse members were the critical and controlling members in the system's load rating.

Load Rating Through Dynamic Testing

Several researchers have concentrated on the dynamic response of the structures to estimate their stiffness and load bearing capacity. Islam et al. (2014) developed a method for load rating of pre-stressed box beam (PSBB) based on the dynamic response collected via wireless sensors networks (WSNs). Two single-span bridges were selected for this study: one of which was 85 ft, long and 36 ft wide and was used to collect the data used in the development of the proposed load rating method; and the other one of which was 90 ft. long and 44 ft. wide and was used for validation. Two WSNs were set out on the PSBB to collect data at a sampling rate of 100 Hz at 2g scale, and three trucks were run (12 runs) with three variable loads and at four different speeds for collecting real time dynamic response of the bridge in its current condition. Each set of WSN included four small programmable object technology wireless accelerometer sensors and one base station connected via serial bus cable to a laptop. Finite element (FE) simulations of 3D bridge models under vehicular loads were performed to get the dynamic response of the bridge at its initial state and then the model was validated by field testing and numerical analysis. Fast Fourier Transform and pick-picking algorithms were used to get the maximum peak amplitudes and their corresponding frequencies. Using the SDOF method and the load displacement relationship, the bending stiffness of the bridge was calculated to estimate its load-bearing capacity, which is the same as the actual rating of the structure. The results obtained from the FE were used within a developed software application that can instantaneously determine the load rating of the bridge from the collected dynamic response.

Siswobusuno et al. (2004) proposed a load rating technique based on modal testing and ambient traffic measurements on a single span bridge that has a concrete deck with steel girders. The instrumentation was composed of a sledge hammer of 20 lb applied to the deck at specific spatial points and a single piezoelectric accelerometer fixed underneath the middle of the outermost steel girder to extract the mode shapes and vibration frequencies. A grid of 42 nodes was specified on the first bridge and 54 for the second bridge. Each node was excited with the sledge hammer five times and data was collected at a sampling rate between 500 to 1000 Hz. The signals were collected using a 12-channel data acquisition system (DAQ) along with a 4-channel ICP Sensor Signal Conditioner to enhance the signals. The bending frequencies were determined from the response functions computed from the collected time domain signals. The bridge's first bending frequency was used to back calculate the stiffness and load capacity of the bridge. The bridge design capacity, which was obtained by subtracting the change in load capacity from the maximum load, was then validated using a static load test. The static test consisted of a 2-axle truck placed in 9 different positions, different weight increments applied to the bridge top and 9 dial gages below the deck to measure the deflection. The results were satisfactory, showing that the dynamic test results were close to the static test results and may be employed for bridge load rating.

Samali et al. (2007) presented a novel dynamic based method by which the in-service stiffness of the bridge is estimated. This method involved the attachment of a few uniaxial accelerometers underneath the bridge girders. The vibration measurement of the bridge superstructure was collected considering two cases: (1) the original bridge, and (2) the bridge with an added mass at the mid-span. Two sets of bending frequencies were measured for the bridge: “as is” and when loaded by extra weight. Upon the application of additional loading to the bridge, the bending frequency of the bridge decreased. From the resulting frequency shift due to added weight, the flexural stiffness of the bridge was calculated. From the obtained flexural stiffness, load carrying capacity of the bridge was computed. The reliability and simplicity of the proposed methodology had been demonstrated by testing over 200 bridge spans covering a wide range of single and multi-span timber bridges.

Application of finite element tools for condition assessment of arch bridges was studied by Boothby and Atamturktur (2007) with detailed instructions in relation to geometric and solid models as well as meshing and implying boundary conditions. The physical parameters of FE models were adjusted during the calibration process with reasonable assumptions for accuracy of the 3D FEt models of stone arch bridges by Fanning et al. (2001).

Similarly, Caglayan et al. (2012) investigated a three-span arch bridge located in a region prone to earthquakes. The researchers generated a finite model of the bridge using a commercial software. Accelerations data tests that were conducted on the bridge were used to refine the model by changing the structural parameters of the bridge. The obtained final model was used for condition assessment.

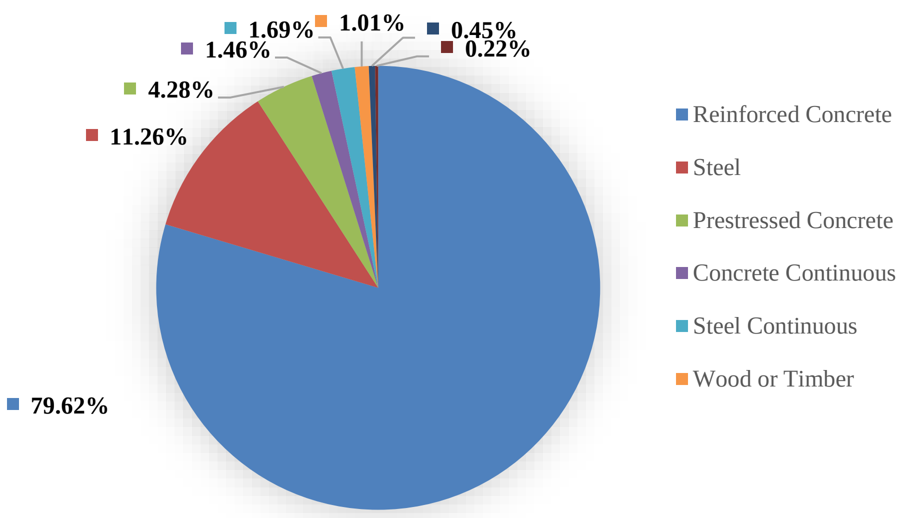
Wang et al. (2005) proposed a condition assessment methodology that consists of generating an FE model, calibrating that model to match experimental data, and using the results after calibration to rate the condition of the bridge or investigate unique loadings or retrofit schemes. During the calibration process, different parameters were adjusted using two conditions of loadings that are static and dynamic. The selection of the parameters was made by referring to the work of Turer (2000). The final calibrated model that can mimic the behavior of the real structure was used for load rating.

Inventory Classification (Task 2)

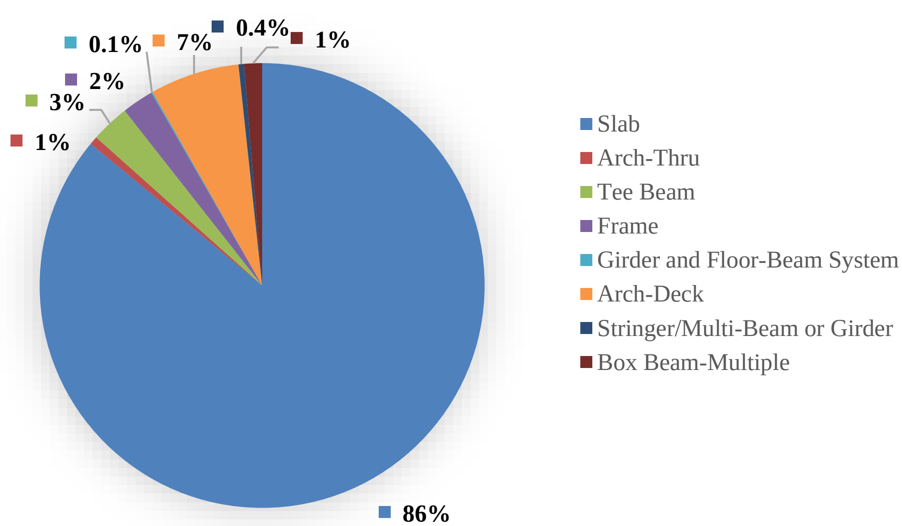
Characterization of Bridges Without Plans

As previously noted, the BrM database provided by VDOT was used for classification of the VDOT inventory and evaluation of the target bridge population with missing or insufficient plans. The database contains approximately 22,900 bridges; with the exclusion of culverts (8,091), non-highway bridges such as railroad or pedestrian (1,026), and bridges that are closed or not yet open to traffic (887), the population was reduced to 12,925 bridges. It was determined that approximately 7% of these structures (933 of the population of 12,925) within the database did not have plans.

Within this pool of 933 structures, the vast majority were reinforced concrete structures (Figure 7a), with the primary design configurations being slab, arch-deck, and T-beam bridges (Figure 7b). In consultation with the project's TRP, the structure types selected for further study were slab and T-beam structures. Combined, these two classes of structures are 63% of the population of bridges without plans (compared to all slab and T-beam structures being less than 30% of the entire VDOT inventory.



(a)



(b)

Figure 7 – Distribution of VDOT Bridges Without Plans by (a) Primary Structural Material, and (b) Bridges Built With Reinforced Concrete Material

Table 3 shows the relative similarity of the basic characteristics of the population of bridges without plans with those of the entire population of each bridge type. Generally speaking, bridges without plans are typically older, have slightly lower condition ratings, lower importance (based on average daily traffic [ADT]), and a lower likelihood of having been reconstructed.

Table 3 - Average Characteristics of VDOT Bridge Inventory

Bridge Type	RC Slab Bridges		RC T-beam Bridges	
Variable	**All (3,026 bridges)**	**Without Plans (571 bridges)**	**All (826 bridges)**	**Without Plans (22 bridges)**
Span Length (ft)	22.2	14.4	45.4	36.9
Age	65.5	79.6	69.0	86.3
ADT	2,325	1,915	6,259	2,718
Deck Condition Rating*	6.33	6.12	5.89	5.56
Superstructure Condition*	6.33	6.12	5.71	5.17
Substructure Condition*	6.24	6.06	5.81	5.17
Number of lanes*	2.0	2.0	2.1	1.9
Number of spans*	1.4	1.1	3.0	1.9
Skew Angle	14.1	11.5	12.3	8.7
Percent posted	5.9	10.9	4.8	38.9
Percent Reconstructed	31.7	24.3	41.8	16.7

* These are discrete values between 0 and 9 but are averaged as numbers for comparison and illustration purposes.

With this task, the research team also began an initial study on the use of inference-based methodologies to formulate a rapid screening tool for VDOT's inventory based on existing rating and posting classifications. This method was an attempt to formulate and systematize existing judgment-based load ratings by leveraging the emerging data-mining techniques to find hidden relationships between certain characteristics of bridges and the load rating/postings for those bridges. Similar to when an engineer uses engineering judgment and experience to infer a rating for a structure based on a comparison with similar structures with plans, the method involved building a set of models based exclusively on the dataset of reinforced concrete slab bridges solely in Virginia and, separately, across the nation. Using regression techniques, the models identified highly recurrent trends in the both the BrM and National Bridge Inventory (NBI) databases between bridges with plans (that also have certain design live loads, condition, traffic and so on) and their analytically-calculated load ratings. These trends were then applied to bridges without plans in order to estimate their load ratings. Initial findings of this inference modeling approach proved successful as a potential screening tool, but were only developed for slab bridges as a trial. This supplementary study was beyond the scope of the investigation, but could be studied more extensively in the future and also be extended to T-beam bridges. Further details on this preliminary study are presented in the Appendix.

Evaluation of Alternative Methods for Load Rating Unknown Structures (Task 3)

To evaluate the performance of the proposed methods for load rating of bridges with unknown or insufficient details, the methods were evaluated on a small population of bridges within VDOTs inventory. This population included four bridges, two slab bridges and two T-beam bridges. The bridges selected were in good and fair condition, one in each condition state for each bridge category. Plans were available for each bridge and provided a basis for comparison of the results derived from each method. The following sections include a summary of the selected bridges, an overview of the instrumentation, testing, and measured data processing, and an evaluation of method results.

Field Testing of Selected Bridges

Description of Selected Bridges

Based on the criteria set in the *Methods* section, the following bridges were selected for evaluation (additional details on the selected bridges are available from VDOT):

War Branch Bridge: The War Branch Bridge (Federal ID No. 15935, VA Structure No. 00826014, Figure 8a) carries Route 613 over War Branch in Rockingham County. The superstructure is comprised of two 32-ft long, simply-supported reinforced concrete slabs that are 21 inches thick and have a 45° skew. Designed using VDOT's standard plan CS22½ through 32½, the deck has 12-inch diameter voids oriented in the direction of traffic, spaced 18 inches apart. Built in 1976, the 2014 inspection report (date of inspection: 06/4/2014) described the bridge to be in "good" condition, with a deck/superstructure condition rating of 7. The load rating using LRFR method conducted in 2016 listed the inventory load rating at 1.86 and the operating rating at 2.44 for HL-93 design load, while the 2014 average daily traffic count was 777 vehicles per day, with 6 percent of those vehicles being trucks.

Smacks Creek Bridge: The Smacks Creek Bridge (Federal ID No. 01260, VA Structure No. 0046178, Figure 8b) carries Route 628 over Smacks Creek in Amelia County. The superstructure is comprised of two 32-ft long, simply-supported reinforced concrete slabs that are 21 inches thick and have a 15° skew. Designed using VDOT's standard plan CS22½ through 32½, the deck has 12-inch diameter voids oriented in the direction of traffic, spaced 18 inches apart. Built in 1965, the most recent inspection, dated 08/12/2015, described the overall condition of the bridge to be in "fair" condition, with a deck/superstructure condition rating of 5. The concrete slabs had cracking, delamination, efflorescence, rust staining, pop-outs and scaling. The pourable joint sealer was brittle and deteriorated. The substructure had ~~typical~~ cracking and delamination with efflorescence and minor spalling. The 2011 load rating using the LRFR method listed the inventory load rating at 1.05 and the operating rating at 1.36 for design load HL-93.

Flat Creek Bridge: The Flat Creek Bridge (Federal ID No. 01262, VA Structure No. 0046060, Figure 8c) carries Route 632 over Flat Creek in Amelia County. There are five simple spans, each 42.5 ft long, for a total length of 212 ft. Each span is 24 ft wide and consists of three longitudinal T-beams. Each exterior T-beam has a vertical stem with a width of 14 in and a depth of 32 in. Each interior T-beam has a vertical rectangular stem with a width of 16 in and a depth of 32 in, and a wide top flange of 7.5 in thick. The wide top flange is the transversely reinforced deck slab. There is zero skew. Built in 1950, the most recent inspection (Inspection date: 12/15/2015) described overall condition of the bridge to be in "good" condition, with a deck/superstructure condition rating of 7. The structure had isolated areas of minor spalling and scale. The 2011 load rating using the LRFR method listed the inventory load rating at 0.84 and the operating rating at 1.09 for design load HL-93.

Brattons Creek Bridge: The Brattons Creek Bridge (Federal ID No. 15707, VA Structure No. 0816182, (Figure 8d) carries Route 780 over Brattons Creek in Rockbridge County. The structure is comprised of three simple spans of the same length, 32 ft, with a total length of 98 ft - 2in, a width of 23 ft - 8 in, and consists of three longitudinal T-beams. Each exterior T-beam has a vertical stem with a width (this is horizontal length) of 14 in and a depth (this is vertical length) of 24 in. The interior T-beam has a vertical rectangular stem with a width of 16 in and a depth of 24 in, and a wide top flange of 8 in thick. The top flange is the transversely reinforced deck slab and the riding surface for the traffic. There is no skew. Built in 1953, the inspection dated 04/28/2015 described the overall condition of the bridge to be in “fair” condition, while the deck condition rating was 6. The report noted spalls, cracks and delamination in the superstructure. The substructure exhibited scaling, cracking and signs of channel drift. The load rating using the LRFR method conducted in 2011 listed the inventory load rating at 0.77 and the operating rating at 1.02 for design load HL-93.

(a) (b) (c) (d)

Figure 8 – Target Bridges Used for Evaluating Load Rating Methodologies: (a) War Branch Bridge; (b) Smacks Creek Bridge; (c) Flat Creek Bridge; and (d) Brattons Creek Bridge

Live Load Testing

Each of the bridges tested for live load testing utilized similar instrumentation, but the configurations were established prior to testing, as shown in Figure 9 through Figure 12. In this report, field testing results alone are not described in detail as the focus of the investigation centers on leveraging these field test results to arrive at load ratings of the selected structures.

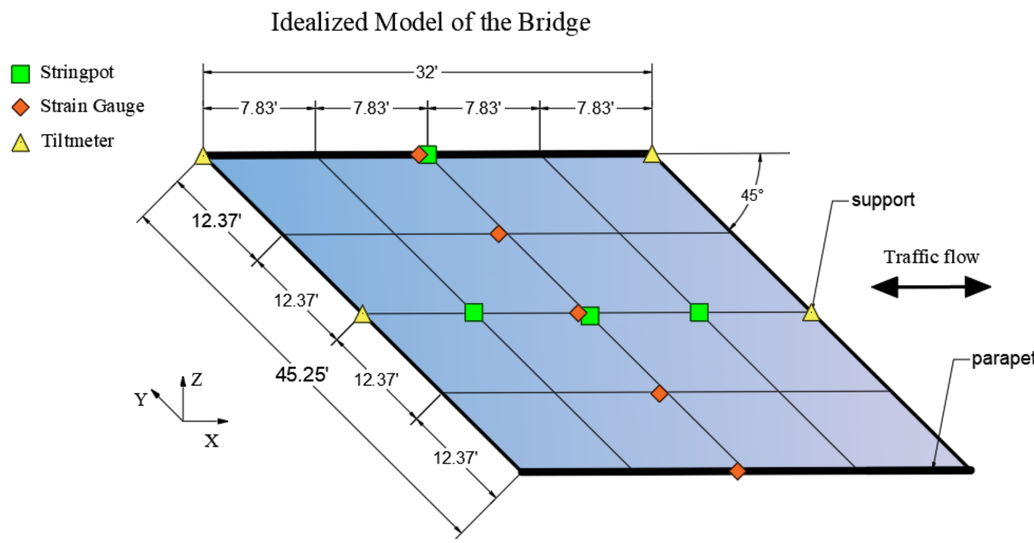


Figure 9 – Instrumentation Configuration for War Branch Bridge for Live Load Testing

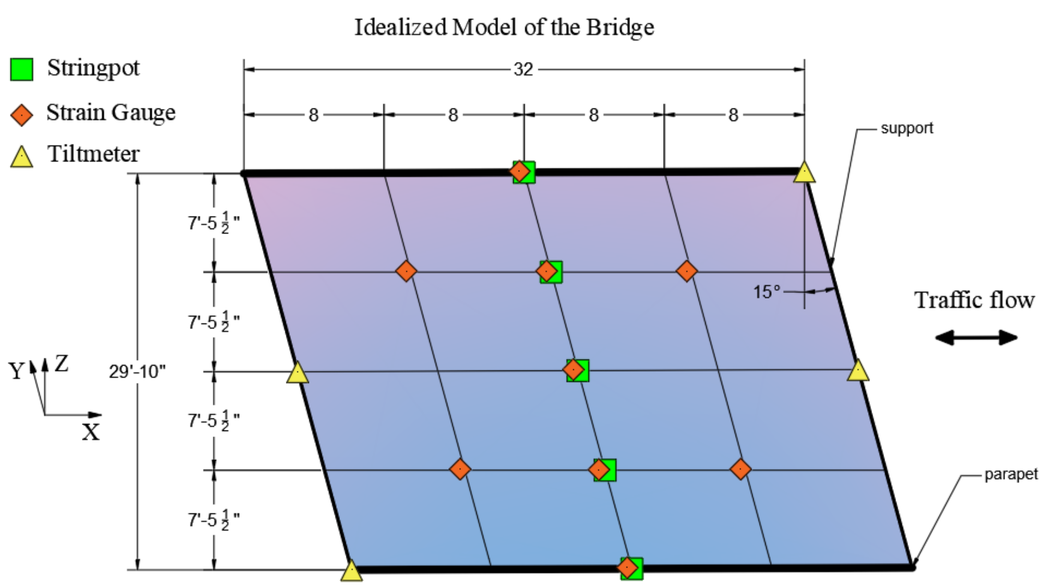


Figure 10 – Instrumentation Configuration for Smacks Creek Branch Bridge for Live Load Testing

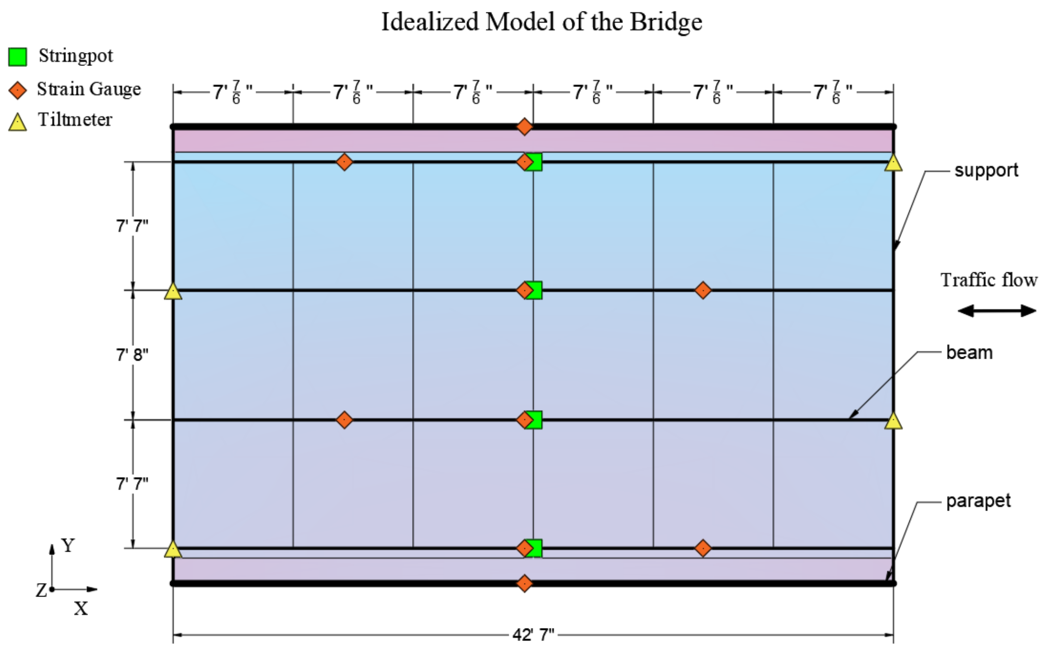


Figure 11 – Instrumentation Configuration for Flat Creek Bridge for Live Load Testing

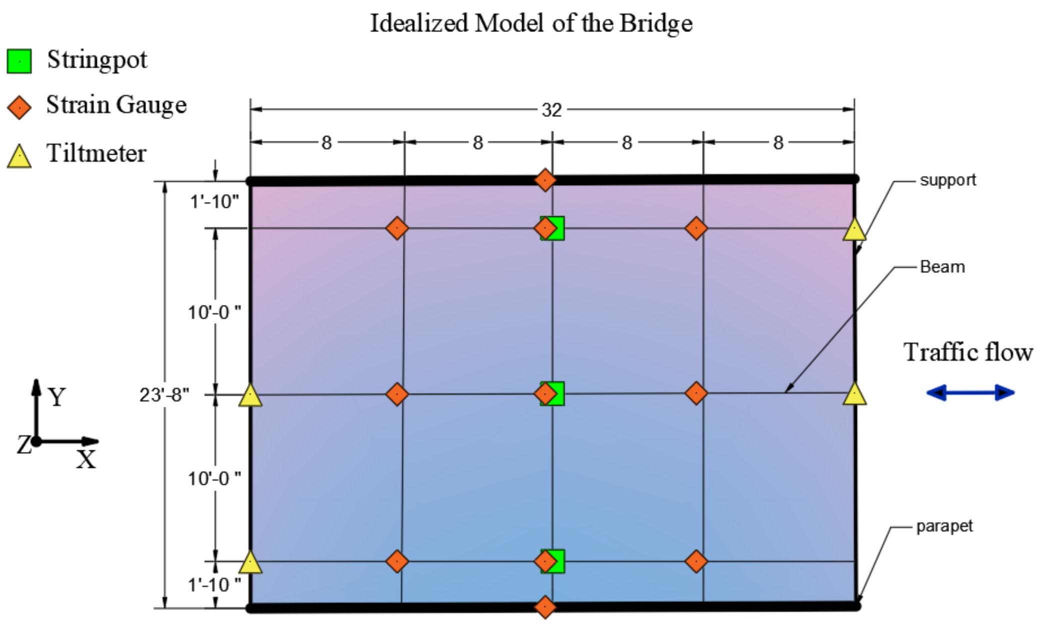
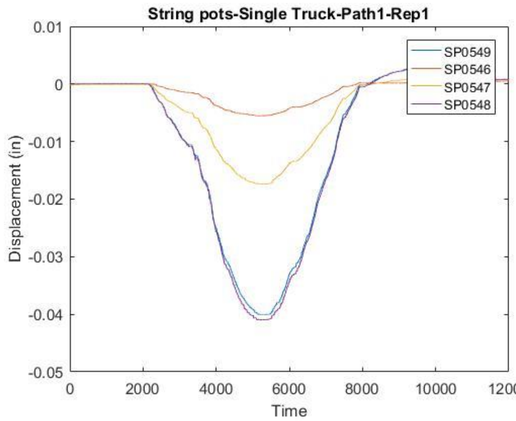
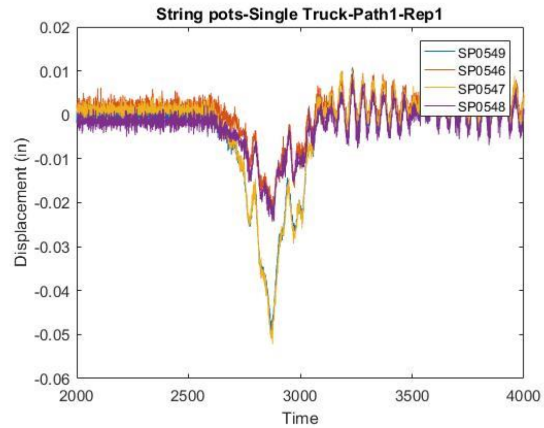


Figure 12 – Instrumentation Configuration for Brattons Creek Bridge for Live Load Testing

The live load testing consisted primarily of a known truck traversing the bridges at various transverse positions at controlled speeds. However, the detailed measurements from each loading configuration are not included in the body of this report. Representative time history plots of the raw field data collected from string pots during quasi-static and dynamic live load testing are shown in Figure 13. Results demonstrated a response that can be described as force effect and load path-guided, with the maximum response observed at a specific sensor location when the loading was driving the measured deformation.



(a)



(b)

Figure 13 - Representative Sensor Measurements (Flat Creek-T Beam Bridge 01262): (a) quasi-static live load test; (b) dynamic live load test

Vibration Testing

Vibration testing conducted on each selected bridge is briefly described below. This testing included both ambient excitation as well as impact hammer excitation.

War Branch Bridge: The dynamic bridge assessment procedure involved the attachment of nine accelerometers underneath the selected span at 16 measurement points in two set-ups as shown in Figure 14. In the first set up, the sensors 1 to 9 were attached to the bridge, while the sensors 10 to 18 were present on the bridge during the second set-up. Note that two sensors were attached to common measurement points in each set-up and were used as reference. The response of the bridge to ambient excitations and an impact excitation were measured. Ambient vibrations were generated by the traffic, wind and people walking across the bridge, and recorded for a total of 15 minutes. Two impact locations as shown in Figure 14 were chosen in order to excite the vertical/bending modes of the bridge using the modal impact hammer. These chosen points were excited by the impact hammer for five repetitions and for a duration of 15 seconds.

Smacks Creek Bridge: The dynamic bridge assessment procedure involved the attachment of accelerometers underneath one span of the bridge at 9 measurement points in one set-up, as shown in Figure 15. The response of the bridge to ambient excitations or an impact excitation generated by a modal impact hammer was measured. Ambient vibrations were generated by the passing traffic, wind and walking people and recorded for a total of 15 minutes. Three impact locations were chosen and excited by the impact hammer for five repetitions and for a duration of 15 seconds.

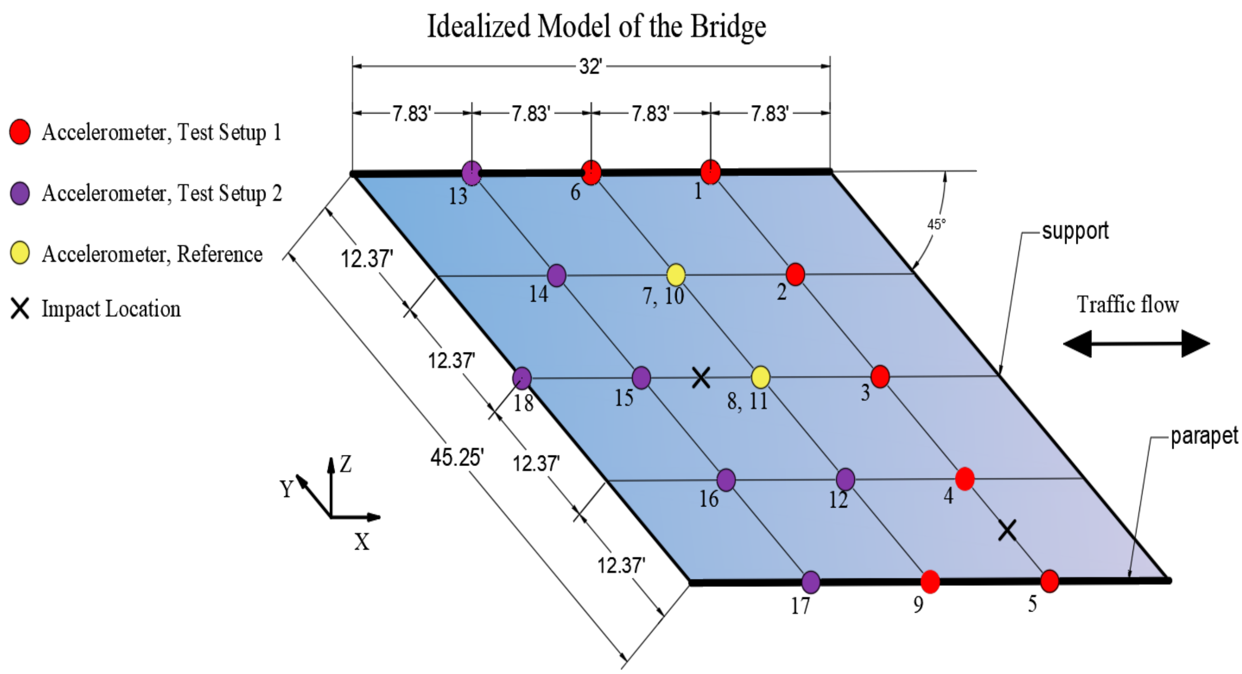


Figure 14 – Instrumentation Configuration for War Branch Bridge for Vibration Testing

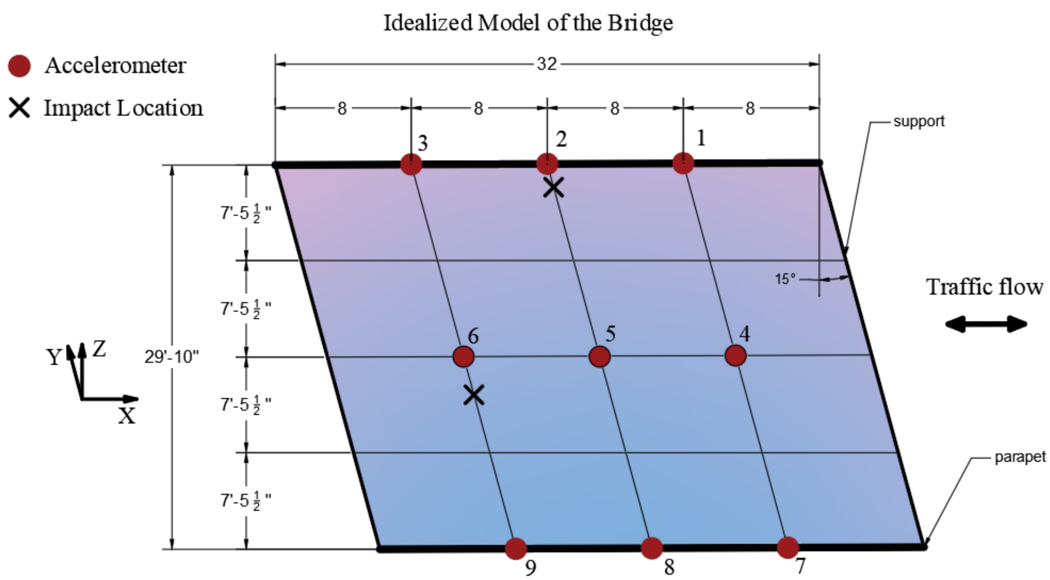


Figure 15 – Instrumentation Configuration for Smacks Creek Bridge for Vibration Testing

Flat Creek Bridge: The dynamic bridge assessment procedure involved the attachment of accelerometers underneath one span of the bridge for a total of 19 measurement points in two set-ups, as shown in Figure 16. In the first set up, the sensors 1 to 10 were attached to the bridge, while the sensors 11 to 20 were present on the bridge during the second set-up. One sensor was present in a common measurement point in both set-ups and was used as reference point. The connection of the accelerometers to the girders of the deck was performed by means of metallic plates bonded to the surface of the concrete. Note that two common sensors were used as reference in each set-up. Ambient vibrations were generated by the passing traffic, wind and walking people and recorded for a total of 15 minutes.

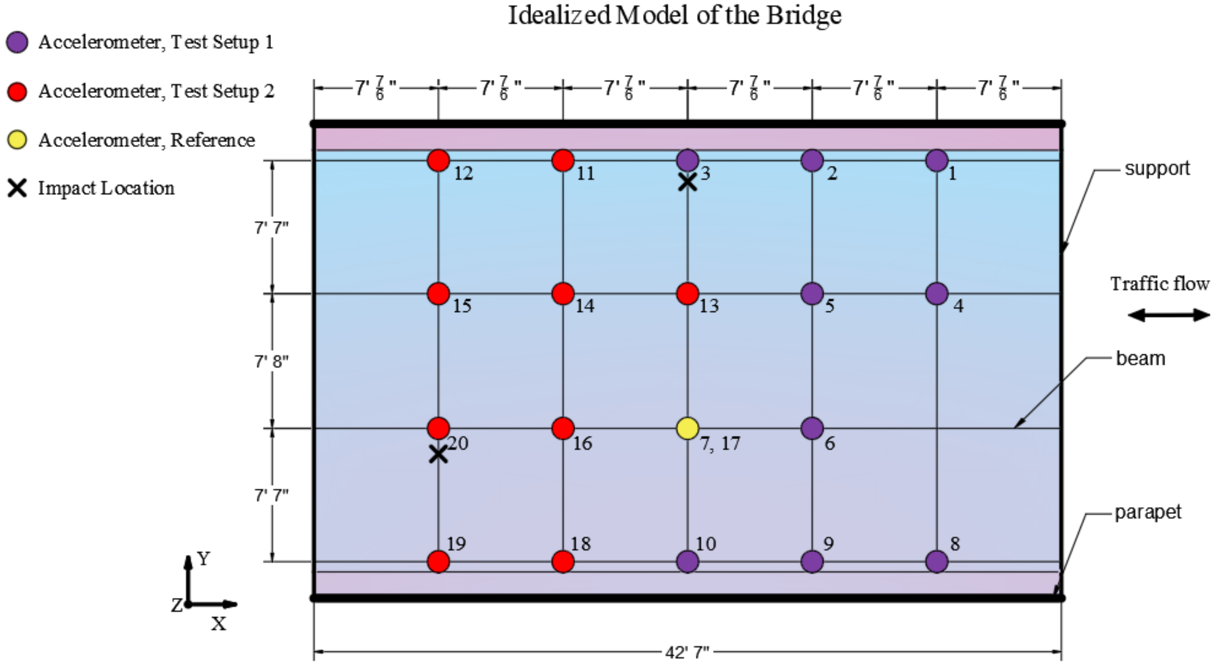


Figure 16 – Instrumentation Configuration for Flat Creek Bridge for Vibration Testing

Brattons Creek Bridge: The dynamic bridge assessment procedure involved the attachment of accelerometers underneath the span of the bridge at 9 measurement points in a single set-up, as shown in Figure 17. The connection of the accelerometers to the girders of the deck was performed by means of metallic plates bonded to the surface of the concrete. Note that two common sensors were used as reference in each set-up. Ambient vibrations were generated by the passing traffic, wind and walking people and recorded for a total of 15 minutes.

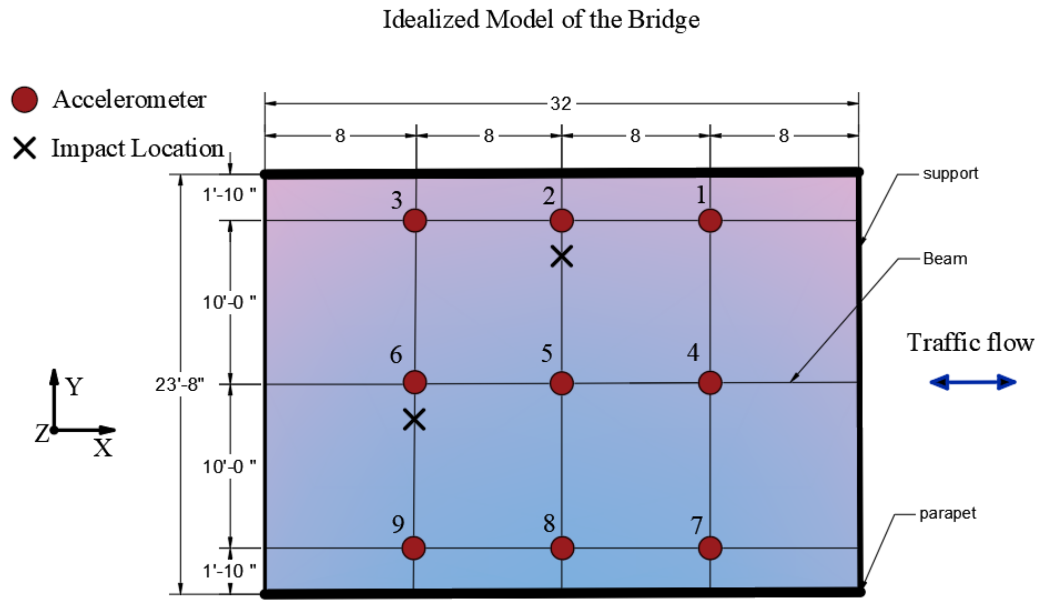


Figure 17 – Instrumentation Configuration for Brattons Creek Bridge for Vibration Testing

Figure 18 illustrates representative time series plots of acceleration data collected during the ambient vibration and impact hammer experiments. It can be seen that peak acceleration responses were observed as a result of either large magnitude ambient loadings (vehicle passing) or impact from the hammer strike. Table 4 and Table 5 provide summaries of the modal properties identified for each of the structures using the previously described EFDD approach for both the ambient and impact excitation methods.

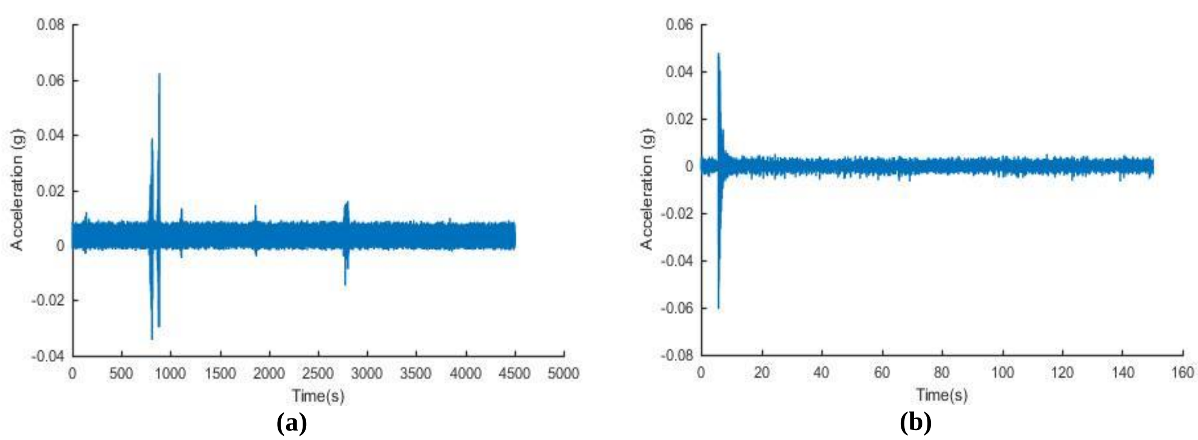


Figure 18 - Representative Sensor Measurements (Smacks Creek-T Beam Bridge 1262): (a) ambient vibration; and (b) impact hammer vibration

Table 4 – Summary of Identified Modal Properties From Ambient Vibration Excitation

	Modal Frequency (Hz)				Damping Ratio %			
Mode	**War Branch**	**Smacks Creek**	**Flat Creek**	**Brattons Creek**	**War Branch**	**Smacks Creek**	**Flat Creek**	**Brattons Creek**
1	25.78	14.11	10.74	14.24	2.92	5.029	6.061	1.97
2	32.22	20.007	13.77	17.53	1.64	3.569	0	0.76
3	38.41	32.70	18.54	25.06	2.69	4.44	2.36	0.54

Table 5 - Summary of Identified Modal Properties from Impact Hammer Excitation

	Modal Frequency (Hz)				Damping Ratio %			
Mode	**War Branch**	**Smacks Creek**	**Flat Creek**	**Brattons Creek**	**War Branch**	**Smacks Creek**	**Flat Creek**	**Brattons Creek**
1	26.35	14.08	11.09	14.15	4.17	2.76	7.85	1.94
2	32.72	20.03	13.94	17.32	4.16	2.32	1.76	1.52
3	39.06	32.63	18.40	22.99	2.71	2.64	3.67	1.38

Load Rating Results From Different Methodologies

AASHTO LRFR Load Rating

As a baseline to which the results of all the other methods could be compared, the conventional analytical load rating method described in the AASHTO Manual for Bridge Evaluation (MBE) was performed (AASHTO 2015). In particular, the LRFR method, which is the most current method and corresponds to the AASHTO LRFD Bridge Design Specifications (AASHTO 2015), was selected as the baseline in this investigation.

In LRFR of the slab bridges, structural analysis was done via the equivalent strip width method prescribed in the AASHTO Specifications, Section 4.6.2.3, accounting for the effects of skew. The HL-93 live load model consisting of the 36-ton design truck or 25-ton tandem and equivalent lane load was considered for live load analysis in the single-lane and multiple-lane loading scenarios. The Strength I limit state and the corresponding load factors were obtained from the Table 6A.4.2.2-1 of the MBE, and the 33% impact factor was used. LRFR was also carried out for the T-beam bridges according to the guidelines of the MBE and AASHTO Specifications.

All the structural and mechanical properties of the bridges according to the plan of the bridges are shown in Table 6, where:

ρ = Density of concrete
E_c = Elasticity modulus of concrete
f_y = Yield strength of reinforcing steel
f_c' = Compressive strength of concrete
M_n = Nominal bending capacity
A_s = Area of reinforcing steel

For the slab bridges, A_s is calculated for a foot-wide section and for T -beam bridges, A_s is calculated for one beam, while Table 6 shows the values for a single bar within the cross-section. For instance, in the War Branch Bridge cross-section within every one foot there are two rebar with area of steel 1.27 in^2; therefore, the entire area of steel within this one foot strip would be 2.54 in^2.

Table 7 presents a summary of the results for all of the bridges evaluated using the AASHTO LRFR method. Note that the rating factors provided in the first row of the table were calculated by the research team, where the factors at the second row were obtained from the VDOT database and provided here as a point of comparison.

Table 6 - Properties of the Bridges Based on Plans

Properties	War Branch	Smacks Creek	Flat Creek	Brattons Creek
$\rho\ (pcf)$	150	150	150	150
$E_c\ (ksi)$	3,322	3,322	3,322	3,322
$A_s(in^2)$	1.27	1.27	1.27	1.27
$f_y(ksi)$	40	40	40	40
$f_c^{'}(ksi)$	3.00	3.00	3.00	3.00
$M_n(kip-ft/ft)$	140.15	140.15	1,251.27	683.09

Table 7 – Summary of AASHTO Inventory Load Rating Methods for Tested Bridges

Methods	War Branch	Smacks Creek	Flat Creek	Brattons Creek
AASHTO LRFR (RF_c)	1.37	1.07	0.88	0.71
VDOT Database	1.39	1.05	0.84	0.77

AASHTO Load Rating Through Diagnostic Load Testing

From Table 8.8.2.3.1-1 in the MBE, *K*, the adjustment factor resulting from the comparison of measured test behavior with the analytical model was calculated for the War Branch, Smacks Creek, Flat Creek, and Brattons Creek bridges. Based on the experimental data collected during the field testing, the diagnostic testing allowed for the original load rating factors, RF_c, to be adjusted to a higher values (RF_T). Table 8 provides a summary of the ratings derived from the AASHTO diagnostic testing method. The results demonstrated the potential improvement in load rating that can be derived from a better understanding on the as-built response and performance versus design approximation.

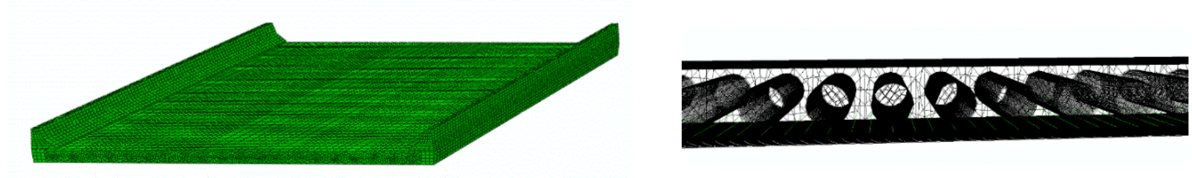
Table 8 – Parameters Using in Load Rating Through Diagnostic Load Testing

	ε_c	ε_T	T (k-ft)	W (k-ft)	K_a	T/W	K_b	K	RF_c	RF_T
War Branch	83	22	143.0	236.5	2.70	0.61	0.8	3.16	1.38	4.37
Smacks Creek	101	70	227.3	289.7	0.44	0.77	1.0	1.44	1.06	1.53
Flat Creek	189	106	247.0	343.4	0.78	0.72	1.0	1.78	0.85	1.52
Brattons Creek	206	116	227.8	245.3	0.77	0.93	1.0	1.78	0.71	1.26

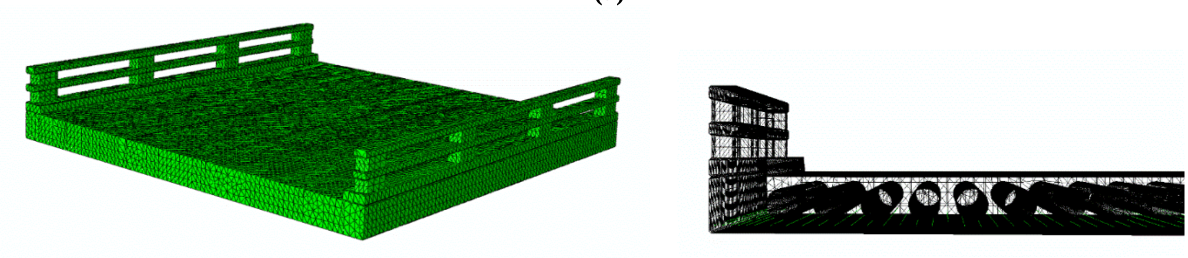
Finite Element Model Updating based Load Rating

An illustration of each of the finite element models is shown in Figure 19, with representations of the model detail and internal features included in the model development. Results for the estimated parameters, calculated capacity, and derived load rating factors are summarized in Tables 9 through 12 for each of the bridges evaluated. For the FEMU-S and

FEMU-H methods, the results are presented for three different loading configurations (i.e. Paths 1 through 3) to illustrate the robustness of the approach.



(a)



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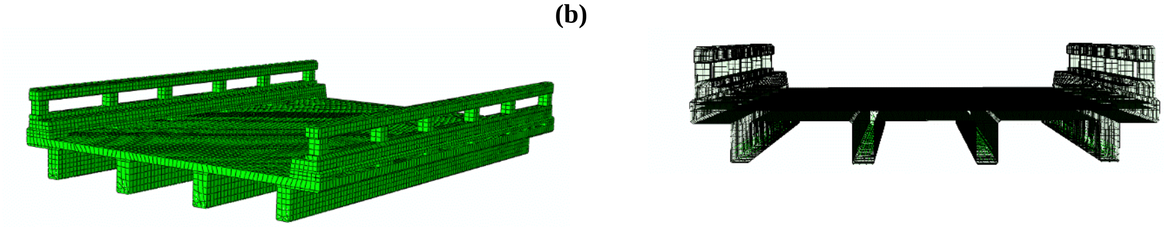

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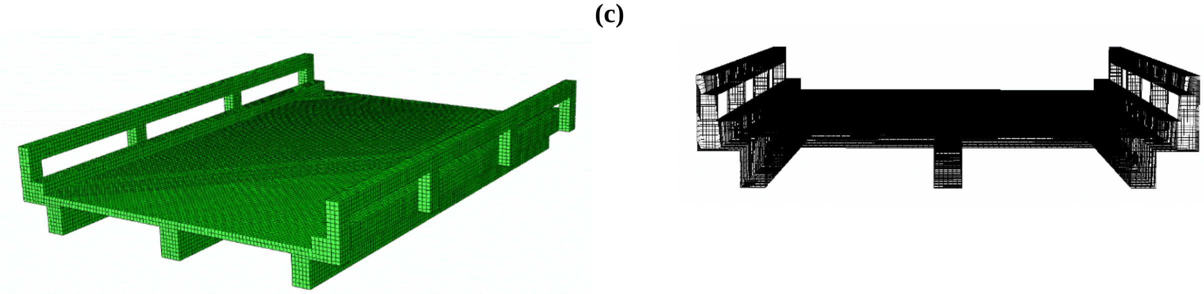

(d)

Figure 19 – Preliminary Solid Finite Element Model and Wireframe Representations to Illustrate Internal Geometry Assumptions: (a) War Branch Bridge; (b) Smacks Creek Bridge; (c) Flat Creek Bridge; and (d) Brattons Creek Bridge

Table 9 –Estimated Parameters and Load Ratings Based on FEMU Approaches (War Branch)

Parameter / Result	FEMU-S			FEMU-D	FEMU-H		
	Path 1	Path 2	Path 3	N/A	Path 1	Path 2	Path 3
$\boldsymbol{E_c\ (ksi)}$	4,502	4,510	4,322	5,003	4,288	3,744	4,115
$\boldsymbol{A_s(in^2)}$	1.30	1.30	1.30	1.40	1.20	1.20	1.20
$\boldsymbol{f_c'(ksi)}$	4.68	4.87	4.68	5.79	4.25	3.24	3.91
$\boldsymbol{M_n(kip-ft)}$	150.90	150.94	150.18	163.81	139.14	136.38	138.38
$\boldsymbol{RF_{moment}\ (Inventory)}$	1.52	1.99	1.51	1.69	1.36	1.33	1.36
$\boldsymbol{RF_{moment}\ (Operating)}$	2.27	2.26	2.25	2.51	2.04	1.98	2.02
$\boldsymbol{RF_{VDOT}\ (Inventory)}$				1.39			

Table 10 – Estimated Parameters and Load Ratings Based on FEMU Approaches (Smacks Creek)

Parameter / Result	FEMU-S			FEMU-D	FEMU-H		
	Path 1	Path 2	Path 3	N/A	Path 1	Path 2	Path 3
$\boldsymbol{E_c\ (ksi)}$	3,084	3,200	3,115	4,335	3,988	3,899	4,122
$\boldsymbol{A_s(in^2)}$	1.21	1.21	0.94	0.96	1.10	1.11	1.22
$\boldsymbol{f_c'(ksi)}$	2.21	2.37	2.24	4.34	3.67	3.51	3.93
$\boldsymbol{M_n(kip-ft)}$	131.83	133.07	105.61	112.85	127.07	130.52	140.56
$\boldsymbol{RF_{moment}\ (Inventory)}$	0.98	0.99	0.70	0.78	0.92	0.92	1.06
$\boldsymbol{RF_{moment}\ (Operating)}$	1.46	1.48	1.07	1.18	1.39	1.39	1.58
$\boldsymbol{RF_{VDOT}\ (Inventory)}$				1.05			

Table 11 – Estimated Parameters and Load Ratings Based on FEMU Approaches (Flat Creek)

Parameter / Result	FEMU-S			FEMU-D	FEMU-H		
	Path 1	Path 2	Path 3	N/A	Path 1	Path 2	Path 3
$\boldsymbol{E_c\ (ksi)}$	3,301	3,341	3,479	4,100	3,114	3,254	3,098
$\boldsymbol{A_s(in^2)}$	0.80	1.00	0.91	1.00	1.10	1.10	1.20
$\boldsymbol{f_c'(ksi)}$	2.52	2.58	2.79	3.88	2.24	2.45	2.22
$\boldsymbol{M_n(kip-ft)}$	798.26	992.95	897.79	1,062.03	1,084.32	1,087.49	1,178.69
$\boldsymbol{RF_{moment}\ (Inventory)}$	0.43	0.63	0.53	0.59	0.72	0.73	0.82
$\boldsymbol{RF_{moment}\ (Operating)}$	0.67	0.96	0.82	0.77	1.09	1.10	1.24
$\boldsymbol{RF_{VDOT}\ (Inventory)}$				0.84			

Table 12 – Estimated Parameters and Load Ratings Based on FEMU Approaches (Brattons Creek)

Parameter / Result	FEMU-S			FEMU-D	FEMU-H		
	Path 1	Path 2	Path 3	N/A	Path 1	Path 2	Path 3
$\boldsymbol{E_c\ (ksi)}$	3,711	4,113	4098	3,666	3,601	3,887	3,741
$\boldsymbol{A_s(in^2)}$	1.68	1.63	1.51	1.40	1.23	1.20	1.25
$\boldsymbol{f_c'(ksi)}$	3.18	3.91	2.79	3.10	2.99	3.49	3.23
$\boldsymbol{M_n(kip-ft)}$	903.87	882.36	820.54	754.95	666.50	650.88	698.30
$\boldsymbol{RF_{moment}\ (Inventory)}$	1.03	1.00	0.91	0.81	0.68	0.66	0.59
$\boldsymbol{RF_{moment}\ (Operating)}$	1.63	1.58	1.44	1.30	1.10	1.07	0.76
$\boldsymbol{RF_{VDOT}\ (Inventory)}$				0.77			

Vibration-based Simplified Method for Load Rating

The VSM utilized the measured vibration response to arrive at the load ratings of the tested bridges. As shown in Table 4 and Table 5, the measured frequencies for both the ambient excitation and impact excitation are similar and would be expected to yield similar estimates of the load rating factors. A summary of results for the estimated parameters and derived load rating factors using the ambient vibration results is provided in Table 13 for each of the bridges evaluated.

Table 13 – Estimated Parameters and Load Ratings based on VSM Approach

Bridge	War Branch	Smacks Creek	Flat Creek	Brattons Creek
$f_1^*(Hz)$	25.78	14.11	10.74	14.24
$E_c\ (ksi)$	5576	3,811	5,266	4,571
$A_s(in^2/ft)$	1.28	1.05	1.31	1.23
$f_y(ksi)$	40	40	40	40
$f_c'(ksi)$	7.97	3.72	6.83	5.35
$M_n\ (kip-ft)$	140.55	109.86	1,308.56	706.26
$RF_{moment}\ (Inventory)$	1.61	0.88	0.79	0.63
$RF_{moment}\ (Operating)$	2.09	1.14	1.02	0.82
$RF_{VDOT}\ (Inventory)$	1.39	1.05	0.84	0.77

* f_1 = First natural frequency of the bridge.

Differential Mass Vibration Based Method for Load Rating

This method was explored only during the War Branch Bridge testing. There was no significant difference in identified natural frequencies of the bridge when the vibration testing was conducted on the bridge with and without added mass. This could be due to the location of the added masses; however, the research team did not explore this method further in testing of other bridges.

DISCUSSION

Table 14 presents a summary of the various load rating methods explored in this study. The comparison uses the Inventory load rating factor for comparison, but the Operating load rating factor would give a similar result. For the FEMU-S and FEMU-H methods, the table presents the results from each loading configurations (i.e. truck paths) as well as the average result obtained from these tests.

Table 14 –Inventory Load Rating Results From Different Analyses

Rating Method		War Branch	Smacks Creek	Flat Creek	Brattons Creek
AASHTO LRFR (RF_c)		1.38	1.06	0.85	0.71
AASHTO Diagnostic (RF_T)		1.38	1.06	0.85	1.02
FEMU-S	Path 1	1.52	0.98	0.43	1.03
	Path 2	1.99	0.99	0.63	1.00
	Path 3	1.51	0.70	0.53	0.91
	Avg.*	1.67	0.89	0.53	0.98
FEMU-D		1.69	0.78	0.59	0.81
FEMU-H	Path 1	1.36	0.92	0.72	0.68
	Path 2	1.33	0.92	0.73	0.66
	Path 3	1.36	1.06	0.82	0.59
	Avg.*	1.35	0.97	0.76	0.64
VSM		1.61	0.88	0.79	0.63

* Average of three truck paths.

In evaluating the results, the proposed methodologies are able to provide reasonable estimates of the rating factors relative to those derived using the baseline AASHTO LRFR method. In this case, "reasonable" is defined as rational estimates on the same order of magnitude; in some cases, these estimates are similar to the AASHTO LRFR estimate. Ideally the proposed physics-based load rating methods would be able to yield conservative approximations of load rating ($RF_{method} < RF_{AASHTO}$) that may not be achieved with subjective rating practices. However, for the tested structures, this outcome was not observed consistently for each load rating method evaluated.

As shown in Table 15, the results obtained from FEMU-S method, which relies on static measurement for model updating, produced the largest rating differences compared to the RF obtained from the AASHTO LRFR method. For two of the tested bridges, the method overestimated the rating factor while for other two bridges it underestimated the rating factor. In addition, for a given bridge, the results obtained from this method showed relatively high loading configuration (truck path) dependence.

Table 15 – Comparison of Percent Difference* Between Method Load Ratings and AASHTO Load Ratings

Rating Method		War Branch	Smacks Creek	Flat Creek	Brattons Creek
FEMU-S	Path 1	10%	-8%	-49%	45%
	Path 2	44%	-7%	-26%	41%
	Path 3	9%	-34%	-38%	28%
	Avg.**	21%	-16%	-38%	38%
FEMU-D		22%	-26%	-31%	14%
FEMU-H	Path 1	-1%	-13%	-15%	-4%
	Path 2	-4%	-13%	-14%	-7%
	Path 3	-1%	0%	-4%	-17%
	Avg.**	-2%	-9%	-11%	-9%
VSM		16%	-16%	-7%	-11%

* Percent difference defined as $(RF_{method} - RF_C)/RF_C * 100\%$.

** Average of three runs.

The results derived from the FEMU-H method, which includes both static and dynamic measurements in model updating, produced the most reasonable estimates of rating factors with a percent difference ranging from 0% to -17%, with negative differences indicating lower estimates of RF than the AASHTO standard. Based on the results, the FEMU-H load rating method tends to consistently estimate load ratings less than those derived from the AASHTO LRFR method, suggesting an overall degree of conservativeness; however, it should be emphasized that only four bridges were evaluated and it is not clear that this trend would hold for all bridges. In addition, there are no significant differences in the load rating estimates of the method when different loading configurations were used in the testing, indicating the robustness of the method in terms of testing configuration.

The results obtained from the FEMU-D method, which used only dynamic measurements for the model updating, produced better results than FEMU-S method but not as good and consistent as those obtained from the FEMU-H method.

The VSM load rating approach produced results with a percent difference ranging from 16% to -16%. The method overestimated the rating factor by 16% for one bridge while it yielded conservative but close estimates to the AASHTO LRFR method for the other three structures. It should also be noted that the VSM load rating method had been developed as an approach that emphasizes limited testing and modeling, and thus provides a mechanism for easier application.

Note that both the VSM and FEMU-H methods requires the identification of natural frequencies of the bridge structure. In this study, vibration testing with both ambient excitations, which does not require any traffic control, and impact hammer excitation, which requires at least partial traffic closure, were considered. As shown in Tables 4 and 5, the natural frequencies obtained from ambient vibration testing and impact hammer testing were very close to each other for all four tested bridges. This indicates that ambient vibration testing where the bridge is excited by passing traffic can be reliably used as the preferred vibrating testing method as it has minimal effects on the operational condition of the bridge.

Sensitivity Analyses of Proposed Load Rating Methods

This investigation proposed two core methodologies for determining the load ratings of bridges without plans or lacking sufficient details to perform a traditional load rating. The methodologies were evaluated on four different bridges representative of these types of structures; however, the selection of an optimal strategy amongst the methods is a complex challenge that includes selection of optimal sensor type and configuration and error assessment, which could not be resolved in this limited investigation. Based on the results, each of the methods has pros and cons, but can all be used to provide a rational estimate of load ratings for bridges without plans or lacking sufficient details. Using the developed methods, a limited sensitivity analysis was performed to evaluate the influence of sensor selection and placement. This sensitivity analysis was not deemed exhaustive; a more appropriate approach would include a formal design of experiments study to consider multiple iterations. However, the proposed sensitivity studies were developed to align with traditional live load and vibration testing strategies that are typically used in bridge evaluation. These live load and vibration tests could be performed by VDOT/VTRC, university partners, or industry consultants and are deemed as standard practice in the bridge community. In this work, the sensitivity analysis aimed to determine how changes in input parameters ultimately influence or impact the derived output, the rating factor. The goal of this sensitivity analysis was to evaluate the impact of measurement scenarios on the estimates of the unknown parameters, or ultimately, to guide the type of measurements that should be collected in future implementations of these approaches.

Sensitivity Analysis of Model Updating-based Methods

In traditional bridge testing, there are numerous types of sensors that can be used to describe the static/dynamic behaviors of the in-service structure. The general assumption in load and modal testing of bridges is that the more measured data collected from the structure, the better the numerical model will be in tune with the structure. However, installing more sensors can be difficult, especially in complex structures, but also uneconomical in terms of time and money. Selecting an appropriate sensor configuration becomes very important to achieving a satisfactory model updating result since the sensors are directly related to the change of the static/dynamic properties predicted by FE models. Therefore, selection of an optimal number/type of sensors to sufficiently characterize the static/dynamic behavior of a bridge remains a topic of study; however, in this work the sensor count and distribution was not deemed excessive.

In this work, the original sensor selection and placement criteria was based on previous experiences in live load and modal testing on other bridges. The primary goal in the selection process was characterization of expected behaviors from the tested structures, such as load sharing, composite action, boundary restraint, operational mode shapes, and damping effects. This resulted in a relatively dense sensor placement during testing; however, the expectation was that these measurements could be reduced for the purposes of implementation. To evaluate the impacts of the reduction in sensor usage, two scenarios were evaluated to assess the performance of the FEMU-H method in estimating the unknown parameters of interest. The reductions focused on selecting sensor measurement combinations that were likely to be used in traditional live load or modal testing, rather than the more extensive distributed sensing approach used in the method development. To achieve this outcome, the following optimization processes were evaluated:

- *Scenario 1: Using midspan deflection and accelerometer sensors.* This type of data is typically collected during a traditional live load test. It is used to describe global load sharing amongst components of the bridge along with the global vibration characteristics.

- *Scenario 2: Using midspan longitudinal strain and accelerometer sensors.* This type of data can be collected during a traditional live load test and has been used to describe both load sharing behavior and localized member deformation along with the global vibration characteristics.

Results of Sensitivity Analysis of Model Updating-based Methods

For the scenarios described, the model updating processes was carried out and the unknown parameters were estimated. A summary of the results is provided in Table 16-Table 19 along with the FEMU-H results using all sensors previously described (bold values indicate largest errors). The results do not clearly indicate that any of the sensing configurations are more appropriate than the others, but do illustrate that comparable results can be achieved using a more limited sensor suite. The results suggest that the updating process is not constrained by the sensor configuration. However, a general principle within a St-ID framework is the assurance that the measurements used in a model updating strategy are inclusive of response characteristics that are activated during testing (e.g. deflection measurements at midspan during a live load test, vertical accelerations for flexural impact test, or support rotation measurements for a midspan loading).

Table 16 - Summary of Sensitivity Analysis Results for War Branch Bridge

Scenario	E_c	E_c/E_{c0}	A_s	A_s/A_{s0}	$E_{c\text{-error}}$ (%)	$A_{s\text{-error}}$ (%)
Path 1						
1	3,988	1.10	1.0	0.79	10	**21**
2	3,111	0.86	1.4	1.10	**14**	10
FEMU-H	4,288	1.18	1.2	0.94	29	6
Path 2						
1	3668	1.01	1.2	0.94	1	6
2	3899	1.08	1.1	0.86	8	14
FEMU-H	3,744	1.04	1.2	0.94	4	6
Path 3						
1	3974	1.10	1.2	0.94	10	6
2	3110	0.86	1.4	1.10	14	10
FEMU-H	4,115	1.14	1.2	0.94	14	6

Sensitivity Analysis of Vibration-based Simplified Method

Number of Sensors Required for Vibration Testing

In the vibration-based simplified method, the estimation of capacity strongly depends on the estimated frequency of the first longitudinal mode. The frequency estimation can be done by a simple Fast Fourier transform of one of the sensors, but the challenge in this case is the uncertainty of whether or not the estimated frequency corresponds to the first longitudinal mode. Therefore, the first sensitivity study for this methodology focused on determining the number of sensors necessary to reliably identify the modal parameters of the bridges and the other output parameters of the method. The War Branch Bridge was chosen for the sensitivity study, as the process for the other bridges was expected to yield similar outcomes. Using the Enhanced Frequency Domain Decomposition method, the natural frequency of the bridge was estimated using full deployment of accelerometers (a total of 16 sensors) distributed across the bridge, and separately, using only measurements from three accelerometers distributed along the centerline of the bridge (that is, in the direction of traffic).

Table 17 - Summary of Sensitivity Analysis Results for Smacks Creek Bridge

Scenario	E_c	E_c/E_{c0}	A_s	A_s/A_{s0}	E_{c}-error (%)	A_{s}-error (%)
Path 1						
1	3744	1.04	1.1	0.86	4	14
2	3111	0.86	1.0	0.78	14	22
FEMU-H	3,988	1.10	1.1	0.86	10	14
Path 2						
1	3655	1.01	1.0	0.78	1	22
2	2988	0.83	1.1	0.86	**17**	14
FEMU-H	3,899	1.08	1.1	0.86	8	14
Path 3						
1	4001	1.10	1.2	0.94	10	6
2	3311	0.92	0.9	0.71	8	**29**
FEMU-H	4,122	1.14	1.2	0.94	14	6

Table 18 - Summary of Sensitivity Analysis Results for Flat Creek Bridge

Scenario	E_c	E_c/E_{c0}	A_s	A_s/A_{s0}	E_{c}-error (%)	A_{s}-error (%)
Path 1						
1	3001	0.83	1.1	1.1	17	10
2	2500	0.69	0.7	0.7	**31**	30
FEMU-H	3,114	0.86	1.10	1.10	14	10
Path 2						
1	3111	0.86	1.1	1.1	14	10
2	3001	0.83	0.9	0.9	17	10
FEMU-H	3,254	0.90	1.10	1.10	10	10
Path 3						
1	3111	0.86	1.2	1.2	14	20
2	2877	0.80	0.6	0.6	20	**40**
FEMU-H	3,098	0.86	1.20	1.20	14	20

Table 19 - Summary of Sensitivity Analysis Results for Brattons Creek Bridge

Scenario	E_c	E_c/E_{c0}	A_s	A_s/A_{s0}	E_{c}-error (%)	A_{s}-error (%)
Path 1						
1	3554	0.98	7.40	0.97	2	3
2	3000	0.83	8.10	1.06	**17**	6
FEMU-H	3,601	1.00	7.40	0.97	0	3
Path 2						
1	3666	1.02	7.20	0.94	2	6
2	3110	0.86	9.70	1.25	14	**25**
FEMU-H	3,887	1.08	7.20	0.94	8	6
Path 3						
1	3744	1.04	7.50	0.98	4	2
2	3777	1.05	8.80	1.14	5	14
FEMU-H	3,741	1.04	7.50	0.98	4	2

Comparisons of a number of parameters using the two different accelerometer arrangements are shown in Table 20. The first three natural frequencies of the bridge (*f_1*, *f_2*, and *f_3*) obtained from two testing scenarios were very close, i.e. within 3% difference. Therefore, the other parameters such as the area of reinforcing steel or capacity of the bridge were almost the same for both instrumentation cases. The only challenge with the reduction of sensors is the uncertainty associated with the correspondence between the identified natural frequencies and the flexural modes; however, the reduction in sensor placement and processing represents a significant improvement in efficiency with respect to field deployment. Therefore, a limited sensor configuration is suitable for identifying the first three fundamental frequencies in short span structures. This outcome also has an impact on the FEMU methodology, as this limited sensor placement reduces instrumentation needed on the FEMU-D and FEMU-H updating approaches.

Table 20 - Sensitivity Analysis for the Simplified Vibration-Based Method – Smacks Creek Bridge

Parameter	**Full Deployment (16 sensors)**	**Middle Longitudinal Sensors (3 sensors)**	**Percentage Difference (%)**
$f_1(Hz)$	14.11	13.70	2.97
$f_2(Hz)$	20.01	19.81	0.99
$f_3(Hz)$	32.71	32.28	1.31
D (lb.in)	3.123 x 10^9	2.942 x 10^9	5.78
$E_c\ (ksi)$	3811	3590	5.78
$A_s(in^2/ft)$	1.05	0.975	7.1
$M_n\ (\ kip - ft)$	109.86.22	101.26	7.82

Selection of Strain Measurement Location

Independent of the vibration sensor configuration, the VSM approach also relies on the collection of a strain measurement to estimate the area of steel reinforcement used in the bridge cross-section. Only one strain measurement is sufficient for this purpose. The method uses the maximum strain measured during the test at a given location, to estimate the area of steel reinforcement and bending moment through an optimization algorithm. To provide guidance on the selection of strain measurement location and explore the robustness of the optimization technique used, the area of the steel reinforcement was estimated by using strain measurements obtained from different sensors and different tests.

Table 21 lists the value of the measured strain for two of the tested bridges (War Branch and Flat Creek Bridges) with the obtained reinforcement area and the bending moment at the location of sensor due to the live load. Two of the sensors deployed in the transverse middle line of the bridge were used. It can be seen that the identified reinforcing steel from the data of different sensors in two tests is identical and bending moment is the only changing value when different strain measurements are used. This indicates that the objective function is stable enough to predict the value of steel reinforcement and any strain measurement at the midspan of the bridge can be used in the estimation of cross-sectional area of steel reinforcement.

Table 21 - Estimated Reinforcement Area and Bending Moment

Bridge	**War Branch**				**Flat Creek**			
Load Test	Path 1		Path 2		Path 1		Path 2	
Sensor	1	2	1	2	1	2	1	2
Strain ($\mu\varepsilon$)	13.2	3.8	2.8	4.6	117	111	116	83
A_s (in^2/ft)	1.23	1.23	1.23	1.23	1.31	1.31	1.31	1.31
M (*ft-kip*)	20.9	5.8	4.3	7.1	431	406	425	304

Problem Resolution

As described in the previous sections, the research focused on developing rational methodologies for determining the load ratings of bridges with insufficient details or unknown plans. Absent full-scale destruction of the four test bridges in this study, the only rational basis for assessing the performance of the alternative methodologies is through comparison with the design approximation-driven analytical approach, that is, the AASHTO LRFR Load Rating. For bridges without plans or insufficient details these physics-based approaches represent a significant improvement in methodology and provides a foundation for rational decision making that could not be previously achieved without either proof testing or some form of non-destructive evaluation. Both groups of methods rely on the collection of field test data, either mechanical, vibrational, or both, for successful application; however, the effectiveness with respect to sensor configuration and deployment requires further study to develop an optimized solution. For the finite element model updating approaches, it is evident that the updating process is enhanced with the inclusion of measurements that capture both the local and global characteristics of the bridge under consideration. For the vibration-based approach, the global response measurement can be captured using a limited suite of sensors, but the approach exhibits some variability in the estimates of load rating. This variability is likely attributed to the multiple approximations and simplifications used in the approach in addition to the primarily global response measurement, but this is somewhat offset by the simplicity in deployments. The outcomes of the study highlighted two general approaches that are suitable for estimating load ratings in the absence of sufficient details. Also included in the report are initial guidance on the process used for each method and synthesis of the rationale for instrumentation needed to execute these approaches. Finally, the project team has supplied VDOT with comprehensive user guides for each of the methods developed such that they could be used internally or even by external consultants.

CONCLUSIONS

- *The VDOT inventory without plans or insufficient details is primarily composed of slab and T-beam bridges.* These bridges are generally classified as short span concrete structures with traditional reinforcement. The majority of these bridges are two-lane simple span structures with fair condition ratings.

- *Finite element model updating and simplified vibration-based assessments are two effective and rational engineering strategies for establishing a starting point for load rating a reinforced concrete bridge that has certain parameters that cannot be determined without* *some form of non-destructive or destructive evaluation.* The finite element model updating approach minimizes the difference between the initial model and experimental data derived from either live load testing, vibration testing, or the combination. The finite element model updating method requires more time to execute and computational modeling expertise, but results in direct estimates of unknown parameters needed for load rating. Compared to the finite element model updating approach, the vibration-based simplified method is easier to implement, but leads to results with somewhat higher percentage differences in rating factor compared to AASHTO calculations.

- *Once unknown parameters (areas of steel and elastic modulus of concrete) have been estimated using either the finite element model or simplified vibration-based approaches, the load rating of the bridge can be calculated using standard approaches.*

RECOMMENDATIONS

1. *VTRC should support a more in-depth study evaluation of a larger number and more diverse classification of bridges without plans that are in in the VDOT inventory.* Albeit with a limited pool of structures for validation, in their current form, the proposed approaches provide a toolset that can be used VDOT's Structure and Bridge Division as an option for rating structures that have limited or insufficient design details that are required to conduct conventional analytical load ratings. While the approaches explored in this study encompass the vast majority of VDOTs structures that do not have sufficient as-built details, there are still other bridge types that warrant evaluation. This additional study should utilize finite element model updating and simplified vibration-based methods in evaluating a selection of bridges that are of concern to VDOT engineers, with the goal of engineers gaining greater confidence in these methods. The study should incorporate the time and overall costs in comparison to other conventional methods, such as load tests, non-destructive evaluations, and destructive testing.

IMPLEMENTATION AND BENEFITS

Implementation

Regarding Recommendation 1, VTRC will propose to its Bridge Research Advisory Committee the need for a more in-depth study for the purposes of gaining greater confidence in the viability of the proposed methods compared to current practices.

Benefits

The benefits of this investigation are derived from the formulation of a series of methods that can be used by VDOT engineers and VDOT consultants responsible for performing load ratings of bridges without plans or insufficient details required to rate a bridge using the AASHTO analytical approach. The approaches developed will be of direct benefit to VDOT and

Virginia as they will provide VDOT with a method to rationally assess bridges that were rated using engineering judgment or other subjective means; these rational assessments will allow for ratings to be updated and postings to be removed or continued, if warranted, based on better information.

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APPENDIX

Data Driven Preliminary Screening Tool

Complementary to this evaluation of the BrM database, the project team explored inference-based methodologies to formulate a rapid screening tool for VDOT's inventory based on existing rating and posting classifications. This method is an innovative attempt to formulate and systematize existing judgment-based load ratings by leveraging the emerging data-mining techniques (such as the results from Objective 1 of this study) to find hidden relationships between certain characteristics of bridges and the load rating/postings for those bridges. Similar to when an engineer uses engineering judgment and experience to infer a rating for a structure based on a comparison with similar structures with plans, this method involves building a set of regression models based exclusively on the dataset of reinforced concrete slab bridges solely in Virginia and, separately, across the nation. Using regression techniques, the models identified highly recurrent trends in the both the BrM and National Bridge Inventory (NBI) databases between bridges with plans (that also have certain design live loads, condition, traffic and so on) and their analytically-calculated load ratings. These trends were then applied to bridges without plans in order to estimate their load ratings. The schematic of the method is illustrated in Figure A1. This inference modeling approach in this study was only developed for slab bridges, but could also be extended to T-beam bridges in the future.

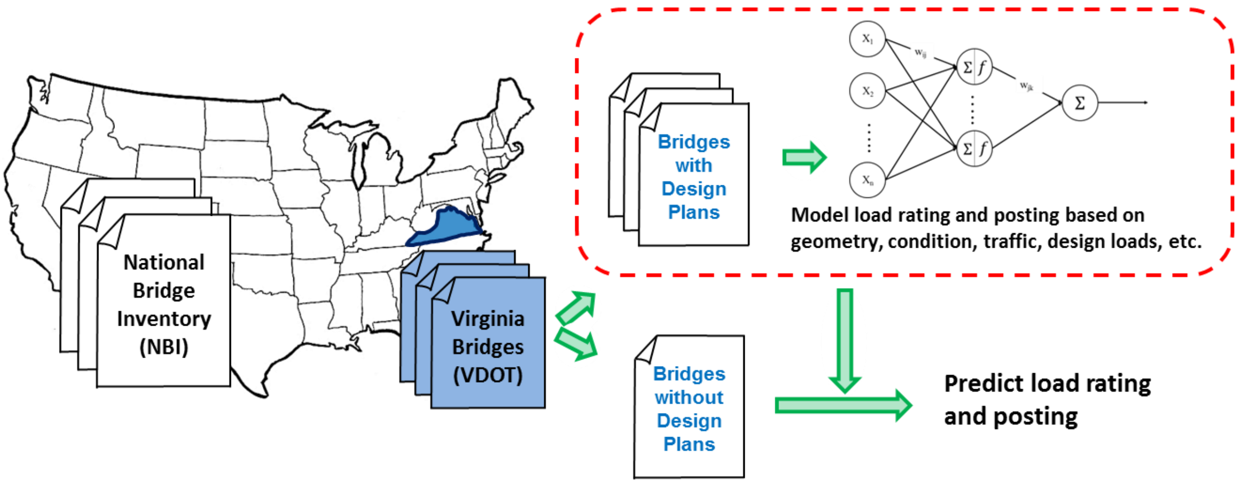

Figure A1 - Flowchart for the Data-Driven Load Rating Method

A set of models was built exclusively on the dataset of RC slab bridges with plans in Virginia. These models took bridge age, span length, condition ratings, skew and ADT as inputs, and predicted load rating based on all other bridges in the database. Several regression models were constructed for this purpose and based on inputs from the databases, these models generated load ratings that were either the same as, or more conservative than, the conventionally-calculated load ratings more than 90% of the time. While the accuracy of the models in estimating the numerical value of load rating was relatively satisfactory, it was concluded that a more valuable approach would be to try to leverage the same statistical and data mining techniques as a preliminary screening tool to inform posting decisions. This screening tool would suggest whether or not a bridge with certain characteristics, such as age, length, conditions ratings, ADT, etc., was likely to require posting or not based on the statistics of the database and recurrent patterns. This yields a systematic approach to the practice of judgement-based ratings where an engineer decides whether or not to post an unknown bridge, based on experience and evidence from service life, site conditions, and inspection reports.

Therefore, the next set of models was focused on predicting the posting label of a bridge, that is, whether or not a bridge should be posted. Additionally, these models were built on the entire NBI dataset of RC slab bridges in the country (about 50,000 bridges), which include Virginia bridges as a subgroup to increase the number of observations used in pattern extraction. Two widely used data mining techniques, namely Decision Tree and Random Forest models, were constructed and trained using the NBI dataset. A sample decision tree is depicted in Figure A2 and illustrates the patterns observed between bridge characteristics and whether or not the bridge had a relatively higher statistical probability of being posted. Note that climate zone and economic index are additional predictors that were added to the dataset to better categorize bridges belonging to different parts of the country subject to different climatic and budgetary conditions. Such a decision tree outlines the process of deciding on the vulnerability of specific bridges without plans, similar to the thought process of a rating engineer exercising judgement-based rating. This model can be thought of as a preliminary statistical screening tool. In a separate study, decision tree models for other bridge types (RC T-beam, steel multi-girder, pre-stressed multi-girder and box beam, and wood multi-girder bridges) demonstrated comparable level of accuracy (Alipour et al. 2017); however, these results have not been included in this work.

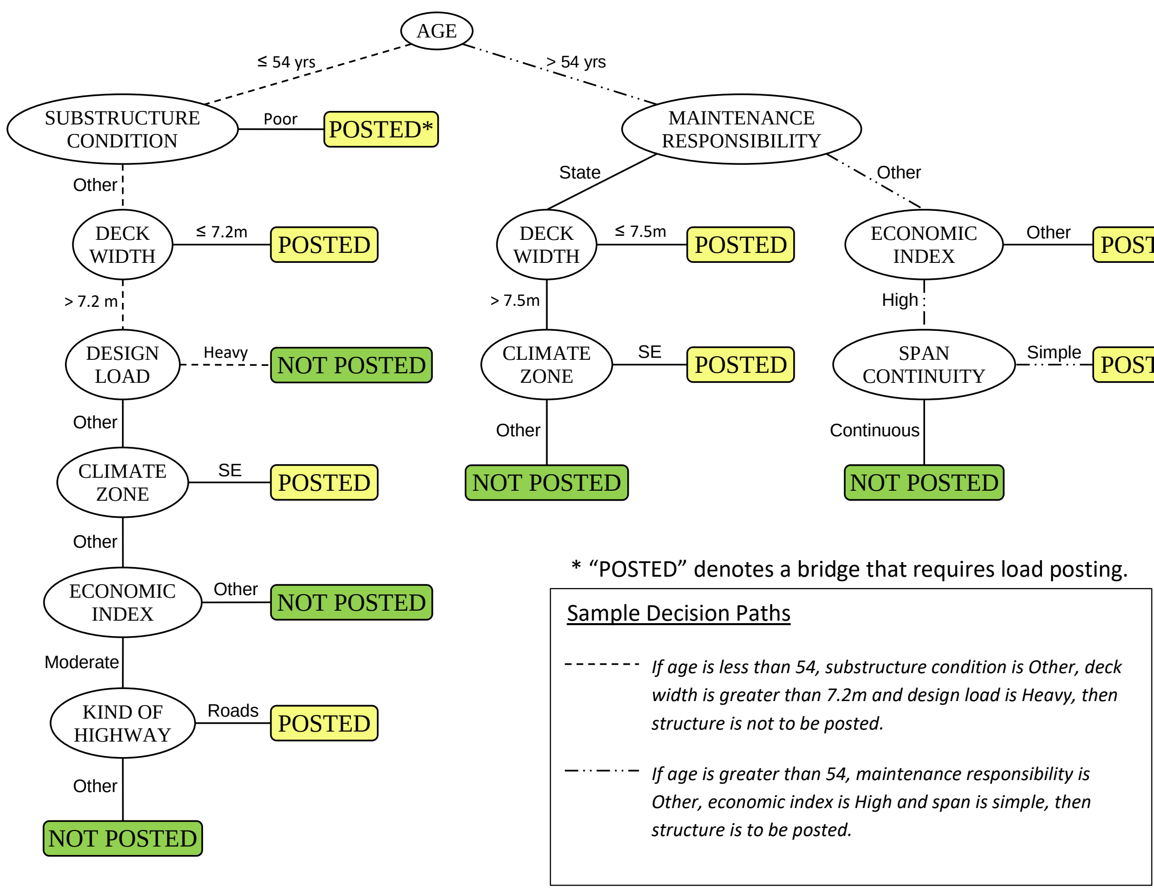

Figure A2 - A Sample Decision Tree

The top five models were able to predict between 77% and 90% of posted bridges, between 84% and 93% of unposted bridges, and between 85% and 92% on the entire population. Note that these best performing models have different strengths, whereby no one model always resulted in the most accurate prediction. Nevertheless, these five models were markedly more accurate than auxiliary tables and flowcharts used by other state DOTs to rate bridges without plans, where the guidance was primarily based on condition ratings alone (ITD 2014; KYTC 2015; TXDOT 2013; WSDOT 2015).

To describe the actual application of these data mining methods in practice, the flowchart in Figure A3 illustrates the process of applying the data-driven models for the screening tasks. To aid in the allocation of resources for improving safety and traffic flow, these models can be used to highlight bridges which would likely benefit from a more thorough evaluation. Unposted bridges, which the model predicts to be posted, may have hidden load carrying deficiencies. Conversely, posted bridges predicted to be unposted may have sufficient reserve capacity to safely carry unrestricted traffic. Such bridges may have been subject to overly conservative analysis and can be candidates for further evaluation and possible load posting removal, thereby increasing the flow of commercial and emergency vehicles.

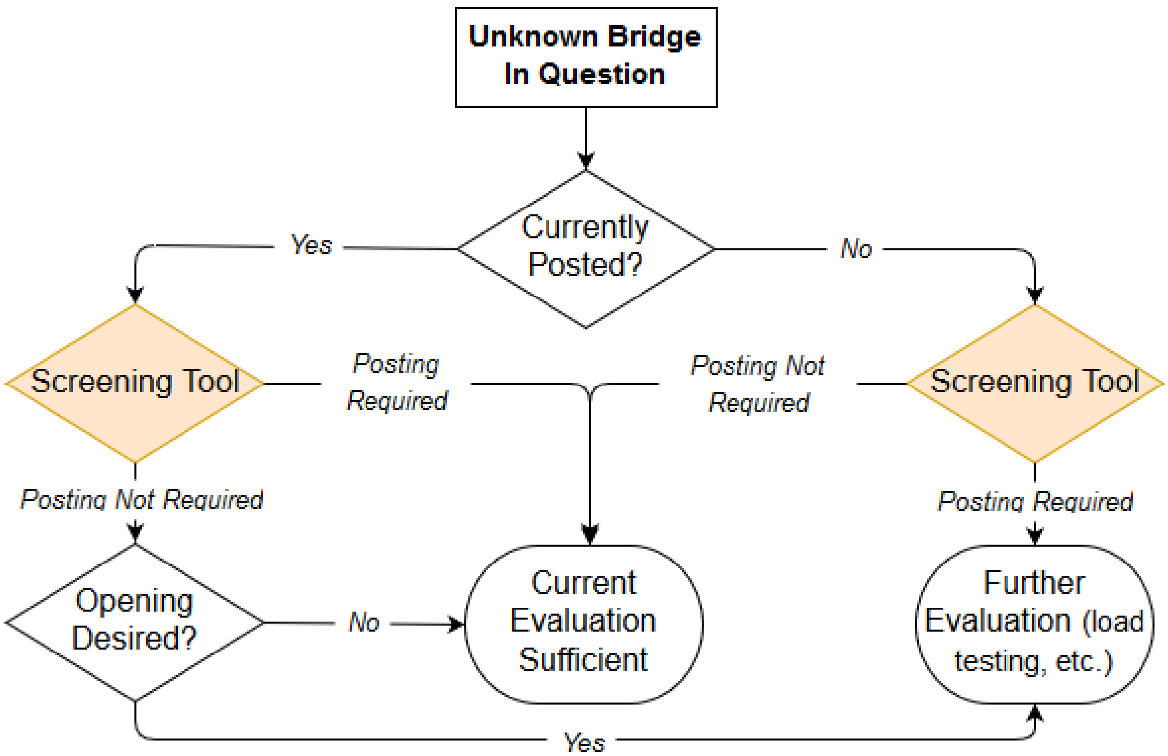

Figure A3 - Application Flowchart for Using the Data-Driven Screening Tool

Finally, a prototype software application was designed to use a number of bridge characteristics as inputs for calculating the probability of the need for posting. The computations are based on the above-mentioned data-driven models and use data from BrM and/or NBI. Figure A4 depicts a screenshot of this software prototype, where the user inputs the bridge data in the left window, and the output reports the load posting predictions on the right. These preliminary tools will be provided to VDOT for future reference, but are not currently publicly available.

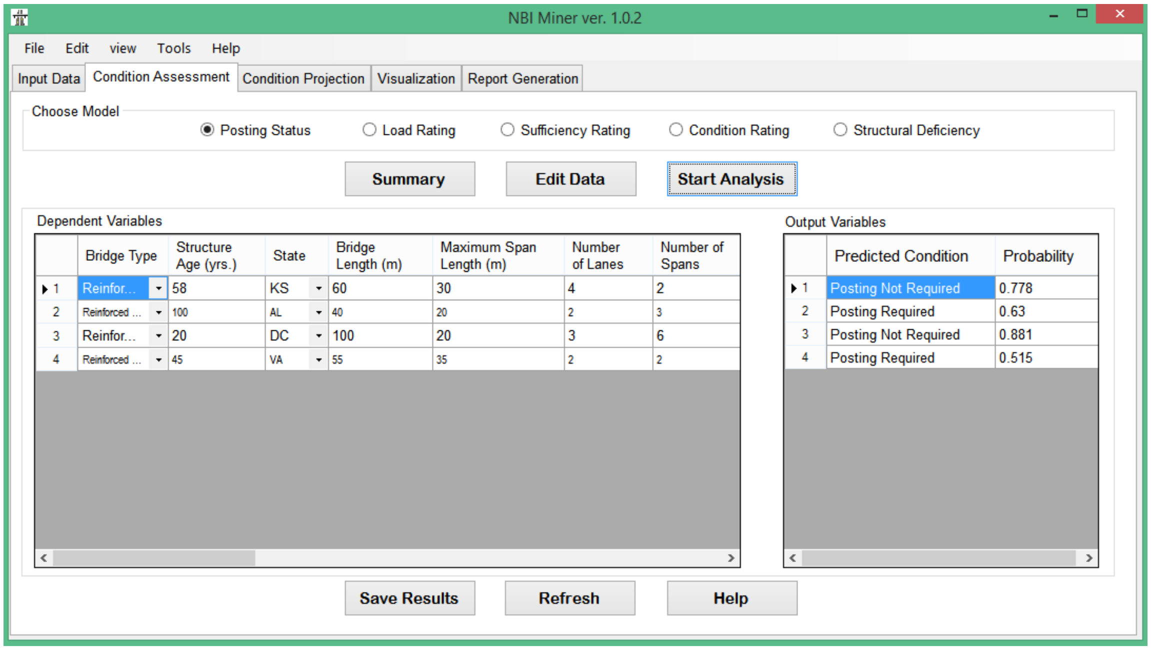

Figure A4 - Screenshot of the Software Application Under Development

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