BRIDGE SAFETY
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ABSTRACT

Bridge weigh-in-motion system (B-WIM) testing is a popular technology in bridge applications. The B-WIM system can track extensive information about loading conditions to which bridges are subjected, and engineers can evaluate the responses of bridges and assess their performance relative to the safety index and serviceability.

FAD (Free-of-Axle-Detector) or NOR (Nothing-On-Road) B-WIM system works well, but only if the system detects axle locations. In the USA, there are challenges for some beam-and-slab bridges. In the first manuscript, we describe a study with alternative strategies for sensor types and sensor installation locations for beam-and-slab bridges. The sensor layouts are identified and two new sensors are investigated.

Most of the commercially available B-WIM systems are based on an algorithm developed by Moses (1979). The performance of this method is acceptable for estimating gross vehicle weight (GVW), but it can be unsatisfactory for estimating single axle loads. In order to improve the accuracy to an acceptable level, two algorithms are proposed. The second and third manuscripts present the measurement of axle weights and GVWs of moving heavy vehicles based on these algorithms. As determined in a case study of a bridge on US-78, both algorithms significantly improved the accuracy of measurements of axle weights in comparison with the commercial B-WIM system.
Existing bridges may be functionally obsolete or have deficient structures based on older design codes or features. These bridges are not unsafe for normal vehicle traffic, but they can be vulnerable to specific traffic conditions. We propose, in manuscript 4, use of a simulation model based on B-WIM experimental data derived during extreme events. The results provide an improved understanding of the possible deficiencies of this bridge, and an appropriate retrofit is suggested.

Finally, the dynamic amplification factor (DAF) is a significant parameter for design new of bridges and for evaluation of existing bridges. AASHTO guidelines provided very conservative values. So, improved methods for determination of DAF values need to be developed to evaluate the safety of existing bridges. This manuscript presents a simulation method to evaluate the DAF of existing bridges by use of the B-WIM data. The accurate results are obtained based on site-specific data.

Keywords: weigh-in-motion, WIM, bridge safety evaluation, dynamic amplification factor, DAF, brake, emergency, simulation model.
DEDICATION

To my wife
Shaohua
And
Daniel and Kaylee
My connection to the future
ACKNOWLEDGEMENTS

There are many people without whom this dissertation would not have been completed.

My deepest gratitude is to my advisor, Dr. Nasim Uddin, for his constant support and guidance, which kept me motivated throughout my Ph.D. curriculum. I have been amazingly fortunate to have an advisor who gave me the freedom to explore on my own and, at the same time, guidance to recover when my steps faltered. Dr. Uddin has provided a great deal of effort to help me form ideas, given research advice, revised the manuscripts, and refined and improved the quality of my research results. His patience and support guided me through difficult situations and helped me accomplish what I have today.

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<td>ALDOT</td>
<td>Alabama Department of Transportation</td>
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<td>ASCE</td>
<td>American Society of Civil Engineers</td>
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<td>B-WIM</td>
<td>Bridge weigh-in-motion</td>
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<td>DAF</td>
<td>Dynamic amplification factor</td>
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<td>DLA</td>
<td>Dynamic load allowance</td>
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<td>FAD</td>
<td>Free of axle detector</td>
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<td>FE</td>
<td>Finite element</td>
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<td>GVW</td>
<td>Gross vehicle weight</td>
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<td>NDT</td>
<td>Nondestructive test</td>
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<tr>
<td>NOR</td>
<td>Nothing on road</td>
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<td>OHOBC</td>
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INTRODUCTION

Much of the bridge infrastructure in North America and Europe was constructed in the 1950’s and 1960’s and has since deteriorated significantly. Moreover, the Safe and Efficient Transportation Act of 2010, by increasing maximum truck weights from 80 to 97 kips, allows states to authorize heavy trucks to operate on the Interstate system (Congress Bills, 2010). Of the more than 3 trillion vehicle miles of travel over bridges each year, 223 billion miles come from trucks. Substandard load ratings and posting of bridges due to heavy truck weights in combination with deterioration problems have led to over 40% of US bridges being classified as structurally deficient or functionally obsolete (AASHTO, 2008). The situation is expected to become worse, and, in the near future, bridges will need more maintenance, major rehabilitation, or replacement. The last reported estimate of the cost to fix the deficiencies of these bridges is over $51 billion (Roberts and Shepard, 2000). So, improvements of current structural safety evaluation are needed. The examination, retrofit, and maintenance of deficient bridges provide challenges to state and local agencies as well as to industry and academia (Adored, 2007).

Structural safety assessments should be essential in any bridge management process (Ţnidarič and Moses, 1997). Many seriously deteriorated bridges can still safely carry normal traffic, and most of these bridges operate in a safe, functional manner. However, these bridges are still classified as structurally deficient or functionally obsolete, because the load models in current structural design codes are usually established for general applications with conservative assumptions, although the live
loads such as traffic volumes and wind speeds may vary considerably from site to site. The use of actual traffic stresses due to truck loads measured during structural field investigations may have a great potential in repair and rehabilitation cost saving by taking advantage of the differences between the actual stress conditions and those specified in the live load models. A first step in the evaluation of bridges is to collect and analyze information about vehicles that pass over the bridge, including types, axle weight, gross weight, space of axles, and speed.

Monitoring of the flow of heavy vehicles must be expedited to improve traffic operations and thus slow the deterioration of the highway system. In combination with the state-of-the-art nondestructive test (NDT) technologies, which provide a cost-effective inspection system to establish the capacity of existing bridges (Washer, 1998; Tarmer, 2001), it is possible to evaluate existing bridges with updated bridge assessment methods rather than standard approaches.

Bridge weigh-in-motion system (B-WIM) is a rapidly developing technology with applications in many fields. Information about bridge behaviors, vehicle weights, and road profiles is essential for detection of bridge damage and for prediction of bridge response under extreme conditions. By applying B-WIM testing, vehicle weights and the serviceability of bridges can be determined, and, based on the B-WIM experimental information, the service life of bridges can be extended, and replacement, repair, or strengthening can be postponed or avoided (Frangopol, 2007, 2008).

The B-WIM systems include a bridge instrumented with sensors, a data acquisition system, and an algorithm to extract relevant information about the vehicles transiting the bridge. The current uses of B-WIM data are in the enforcement of load
regulations to provide accurate estimations of static loads; to provide useful traffic information and the general layout of the traffic distribution; to survey the numbers of vehicles, along with estimations of their gross weight, speed, and axle spacing; and to analyze the dynamic characteristics of traffic.

Currently, the B-WIM technique provides traffic information with an acceptable accuracy (< 15%) for moving weights. Realistic traffic loading and the live load safety factor can be achieved. Both of these factors influence bridge safety (Zag, 2005). The new application of B-WIM systems overcomes this limitation with better performance, or at least with the same accuracy and function (O’Brien, 2008). Meanwhile, the most useful advantage of the B-WIM system is that vehicles can be weighed while moving on highways without having to deviate from their routes. Because of their effectiveness and accuracy, B-WIM systems have gained popularity in recent years. They are now employed in the USA, Europe, and other countries (Yiannis and Antoniou, 2005). Performance results have demonstrated that, when properly employed, existing B-WIM system technologies have the potential of becoming a solid support tool for measuring the size of commercial trucks and for enforcement of weight limits. The simulated WIM (SiWIM) system applied in this research is a B-WIM system that fits the general description of B-WIM systems. The trademarked SiWIM system was developed jointly by the Slovenian National Building and Civil Engineering Institute (Zavod Zagradbenisto Slovenije, ZAG) and a private technology manufacturing firm, Cestel (ZAG, 2005).

This dissertation advances B-WIM research by proposing a simulation of a B-WIM system, integrated with bridge safety.
Organization of Content

The simulation of bridge behavior based on B-WIM experimental data and related applications based on this model are presented in this report in the form of five interconnected manuscripts. Each manuscript is consistent with the objects defined for the entire study.

Manuscript 1 is comprised of the FAD sensor layout based on finite elemental analysis. The bending sensor, compression sensor, and shear sensor are analyzed to provide the best locations for each sensor and the best combination of different sensors. This manuscript has been submitted to *ASCE, Journal of Bridge Engineering*.

Manuscript 2 includes a simulation model to verify the behavior of the bridge frame and to match the actual strain from B-WIM. In this study, the following effects are considered: boundary conditions, semi-rigid connections, bridge elastic stiffness, bridge vibration, dynamic loads, and time delay. Also, this simulation model can be a platform for the following: improvement of accuracy for vehicle weight in the B-WIM system, calculation of the bridge dynamic application factor (DAF), and bridge safety evaluations when vehicles brake during extreme events. This manuscript has been submitted to *Engineering Structures (Elsevier)*.

Manuscript 3 includes two alternative algorithms to improve the current algorithm in the commercial B-WIM system. The first alternative algorithm is to investigate the adjusted moment based on end moment of the member, and is applied to the influence line developed from the proposed simulation model for vehicle weight calibration. The second alternative algorithm is to filter the moment based on the proposed simulation model and then apply it to the corresponding influence line for vehicle weight calibration.
For both algorithms, results show substantial improvements in accuracy. This manuscript has been submitted to *Engineering Structures (Elsevier).*

Manuscript 4 describes an investigation of bridge behavior and predictions of bridge safety during extreme events based on a simulation model. The results indicate that the site-specific bridge has a potential safety issue about the bridge capacity, and, based on the research results, a retrofit is recommended. This manuscript has been submitted to *Baltic Journal of Road and Bridge Engineering.*

Manuscript 5 consists of determination of DAF values based on the simulation model. This is accomplished by determining the static moment based on the simulation model, and then determining the dynamic moment based on the B-WIM testing data. The ratio of the dynamic moment to the static moment is defined as DAF. Also, a method is used to simulate a group of vehicles trafficking on the highway, and DAF is determined from this simulation. This manuscript has been submitted to *ASCE, Journal of Bridge Engineering.*
Literature Review

History of the B-WIM system

The first B-WIM system was developed by Moses (1979). Detection strips to estimate the axle position at each scan of the B-WIM system obtains the strain signal, which can be decomposed into the contribution of each axle. The influence line of a given quantity is a function that returns the value of that quantity for a given load position. By using axle detection strips an accurate estimation of the axle position at every time can be obtained and therefore the strain signal can be decomposed into each of the axles’ contribution. Many factors should be considered in selecting appropriate sites for the installation of effective WIM system, such as the road geometry, pavement characteristics and deformation, etc.

The technologies of sensor types in WIM system include piezoelectric sensors, capacitive mats, bending plates scale, double bending plate scale, single load cell systems (deep pit load cell), optical WIM, and Bridge-WIM systems (McCall and Vodrazka, 1997; Yiannis and Antoniou, 2005).

Bushman and Pratt (1998) analyzed three basic types of WIM sensor technology (piezoelectric, bending plate and single load cell) in terms of accuracy and cost. Taylor and Bergen did some research on WIM system between different level of accuracy and different system and maintenance costs (Taylor and Bergan, 1993; Yiannis and Antoniou, 2005).

Measures should be taken to ensure that the vehicles approaching the weighing system are not subjected to acceleration or deceleration so as to record the constant speed of the passing vehicles (McCall and Vodrazka, 1997). The facilities, which are necessary to install and operate the WIM system, should contain specific temperature requirements,
environment (i.e. water and salt exposure), the traffic conditions, mechanical resistance, electronics, and facilities (Cost, 1999).

B-WIM system installed on existing bridges becomes a portable measuring platform. It also supplies information about the dynamic impact factor, lateral distribution factor and strain records which can be further used for the assessment of bridges (Zag, 2005).

The replaces traditional ones with axle detector technology named like “NOR (Nothing on the road)” or “FAD (Free of axle detector)” (Zag, 2005). This technology was first tested on orthotropic deck bridges (Dempsey et al., 1998); then it was applied in real-time on short slab bridges (Žnidarič and Baumgartner, 1998). O’Brien and Žnidarič (2001) demonstrate the effectiveness of NOR B-WIM system by instrumenting orthotropic bridge decks.

The main advantages of a portable B-WIM system includes: (1) Monitoring truck weight and vehicle size without interfering with the traffic flow; (2) Avoiding damage in the pavement or interfering with the traffic; (3) The transducers can be reused, (4) Can increase the safety, durability and reduce the cost of the installation; and (5) Enhancing data quantity and quality (Zag, 2005; Zhao et al., 2008a, 2008b). B-WIM accuracy mainly depends on (1) Algorithms of B-WIM system, (2) Road roughness.

If the B-WIM system was instrumented bridge with strain gages mounted on the longitudinal girders to weigh gross weight and axle loads as well, and FAD sensors right under the bridge slab to detect the axle of the vehicles, this system was called as SiWIM system. The SiWIM system uses different numerical techniques to solve the system of equations, but the basic principles of B-WIM algorithm remain as developed by Moses in
SiWIM system use actual data from transducers instead of theoretical analysis to define the influence line (IL) of the bridge. The IL are calculated based on the strain records acquired at the site to represent bridge behavior. The more accurate influence line we obtain, the more reasonable estimate of weight can be calculated (Zag, 2005). For this FAD BWIM system (SiWIM 2007), the IL could be calculated just based on the signals without knowing any material, and section information of the bridge. The algorithm is based on the comparison of measured and modeled bridge response (bending moments) of a span due to a passing vehicle. During passing of a vehicle, the strain of the sensors is measured continuously. In the SiWIM system, the strain is recorded at a high rate of sampling (512 samplings per second). SiWIM provides not only the same traffic data as the pavement B-WIM systems, but also some additional structural parameters that can be used for optimized bridge assessment.

The main part of the SiWIM system includes (Figure 1): (1) sensors (weighing transducers and axle detectors) to acquire the signals; (2) cabinet to keep the processor of the system (electronics in the casing and cabling, computer and software); (3) antenna, PDA and WiFi system to represent the core of a SiWIM system to communicate with each other through the TCP/IP (network) protocol; (4) camera system to recognize and capture pictures of vehicles; and (5) solar panels to provide power supply. SiWIM system can be controlled remotely through a mobile phone line (Zhao, 2010).

Algorithm of the B-WIM System

The first B-WIM, developed by Moses (1979), was improved by Moses and Ghosn (1983). In 1984, Peters developed the AXWAY (Peters, 1984), which calculates
the gross weight of a vehicle under the assumption that the gross weight will be proportional to the area under the strain time-history, and CULWAY systems (Peters et al., 1986) for weighing trucks and calibrating truck weights by using peak strains from continuous strain data (Peters, 1986; Matui, et al, 1989). The systems first measure the strain at the center of the culvert or bridge when the vehicle is approaching. In 1989, Snyder (1992) developed the first commercial B-WIM system based on the Moses algorithm. Dempsey (Dempsey et al., 1995), O’Brien (O’Brien et al., 1999a), Žnidarič (Žnidarič and Baumgartner, 1998), and Ojio (Ojio et al., 2000) contributed to the development of the B-WIM system for practice testing. Dempsey et al. (1998) tested an orthotropic deck bridge by applying this B-WIM system, and Žnidarič, et al. conducted testing for short-slab bridges (Žnidarič and Baumgartner, 1998; Žnidarič et al., 2002). Quilligan et al. (2002) developed a two-dimensional algorithm for orthotropic steel decks. In addition, González et al. (1998) and Leming et al. (2003) introduced a dynamic application. These B-WIM systems are based on the least-squares errors method of the Moses algorithm. Other approaches have been made to solve this problem (O’Connor and Chan, 1988a, 1988b; González, 2001; González and O’Brien, 2002). Xiao et al. (2006) proposed mounting the longitudinal ribs of an orthotropic box girder bridge to obtain axle weights with sufficient accuracy. Further, the moving force identification (MFI) theory for predicting axle weights has been developed (O’Connor and Chan, 1988a, 1988b; Chan et al., 1999, 2000, 2001a, 2001b; Chan and Ashebo, 2006; Law and Zhu, 2000; Law et al, 1997, 1999; Yu and Chan, 2003a, 2003b; Zhu and Law, 1999, 2001b, 2003a).

The dynamic contact force is used to estimate dynamic amplification factors. Moving force identification, an ill-posed problem, presents serious difficulties, but major
advances for this approach have been made, especially with the introduction of regularization schemes. Nevertheless, the series of equations of Moses’ algorithm are ill-conditioned because a small change in the constant coefficients results in a large change in the solution. To improve the accuracy of the equations in the B-WIM system, Rowley (2007, 2008) combined the Tikhonov regularization method with the original least squares method for determining vehicle weights. O’Brien et al. (2009) verified that application of the Tikhonov regularization method significantly improved the accuracy for calibration of vehicle weights. A regularized solution following from Moses’ algorithm was proposed, and a regularization method was introduced to control the conditioning of the problem. A method for obtaining the optimal regularization parameter was also proposed and tested in a numerical experiment. It was found that the regularized solution reduced the error in the presence of noise, and that the accuracy of the regularized solution was improved by using a factor of 4 for smooth surfaces and a factor 10 for rough surfaces.

Also, Kim et al. (2009) used two artificial neural networks as a pattern recognition technique to estimate vehicle characteristics from the time signals are recorded for a simply supported concrete bridge and a cable-stayed bridge.

Yamaguchi et al. (2009) applied B-WIM to a curved bridge with skew. The axle loads were obtained by a modification of Moses’ algorithm using local strain influence lines. The algorithm presents the gross weight with errors <10%. Deng and Cai (2010) introduced a method that separates the total response of a bridge into an inertial component due to the momentum of the bridge and an interaction component due to the vehicles moving on the bridge (damping forces are ignored). The study concludes that lack of consideration of the inertial effects (usually disregarded in other MFI applications)
introduced unacceptable errors to load estimations, while the algorithm proposed remained accurate independent of vehicle speed, road surface condition, and moderate levels of noise. However, Wang and Qu (2011) proposed the concept of the dynamic influence line to calculate axle loads, even when the effects of bridge dynamics are relatively large. The dynamic influence line corresponds to the measured response when a unit load crosses the bridge at a certain speed. In this, the dynamics of the bridge are included, so it has a free-vibration part after the vehicle has left the bridge. The dynamic influence line cannot be obtained from measurements. Therefore, a sufficiently accurate model (confidence level $\pi_c = 95\%$) of the modes of the structure is required for obtaining the influence line.

With use of B-WIM systems for single orthotropic deck bridges, multi-span bridges, and continuous bridges, more applications have been developed. Thus, for evaluations of bridge integrity, a higher accuracy of detection of vehicle axle weights and related bridge strains is needed.

Current practices and research on the applications of B-WIM are limited, however, to short, single-span, and orthotropic bridges. In this dissertation, we propose an innovative simulation model and two alternative algorithms for calculating vehicle weights for multi-span bridges using B-WIM experimental data.

Project Objectives

Current practices and research for the application of B-WIM are limited to short, single-span, and/or orthotropic bridges, mostly in European countries. In the USA, the typical bridge is slab-on-girder with multi-spans. For these, commercial B-WIM systems
have limited utility. In the present study, we aimed to improve the reliability and effectiveness of the B-WIM and to evaluate the safety of existing bridges based on B-WIM-monitored structural responses and conditions of bridge components, and thereby to extend bridge life.

The overall goal of this research was to improve performance of the current commercial B-WIM system and to develop new applications based on the information provided by the B-WIM system. The following research was addressed:

**Vehicle Axle Detection Strategy for Bridge Weigh-In-Motion Systems**

In recent years, the innovative FAD (Free-of-Axle-Detector) or NOR (Nothing-On-Road) B-WIM has been deployed as an axle detection method. It involves placement of sensors under the bridge rather than the traditional detection of axles above the road surface. This axle detection method is incorporated into the B-WIM system. The FAD B-WIM system works well if axle locations can be identified in the strain signal, but there are challenges for some bridges, including the typical slab-on-girder bridges in the USA. When the wheels of vehicles are located close to the girders, the girders distribute the vehicle loads directly to the foundation or support, so that the strains of the slab are often so faint that axle detecting sensors miss the peak at transverse position for axle detection. An alternative strategy is proposed to improve the reliability of B-WIM systems by use of finite element analysis (FEA). In field testing of this system, there were fewer instances of missing the detection of axles.
Field Verification of a Filtered Measured Moment Strain Approach to the Bridge Weigh-in-Motion Algorithm, Part I: Simulation model

A multi-span bridge has the characteristics of a continuous bridge, and the joint connections between each span have the capacity of moment transfer. In order to verify the behavior of a bridge frame and to measure strains by B-WIM, the following effects were considered: 1) boundary conditions; 2) semi-rigid connections; 3) bridge elastic stiffness; 4) bridge vibration; 5) dynamic loads; and 6) time delay. The model was developed to simulate these effects and to verify the assumption by using the B-WIM moment. This simulation model can be a platform for the following: 1) accuracy improvement for weight calibration by the B-WIM system; 2) calculation of the bridge dynamic application factor (DAF), and 3) bridge safety evaluations when vehicles brake during extreme situations.

Field Verification of a Filtered Measured Moment Strain Approach to the Bridge Weigh-in-Motion Algorithm. Part II: Bridge Weigh-in-Motion Algorithm

The performance of a B-WIM system is generally acceptable for estimating GVWs, but it can be unsatisfactory for estimating single axle loads. To improve the current algorithm in the commercial B-WIM system, two alternative algorithms are proposed for vehicle weight calibrations. The first is to investigate the adjusted moment based on the end moment of the member, and it is applied to the influence line developed from the proposed simulation model for vehicle weight calibration. The second involves filtering the moment based on the proposed simulation model and then applying it to the corresponding influence line for vehicle weight calibration. Both algorithms have results
that show a substantial improvement in accuracy. Currently, both algorithms focus on the improvement of accuracy of weight calibrations for single run. Multiple runs will be investigated by algorithms to be proposed in the future.

Field Calibrated Simulation Model to Perform Bridge Safety Analysis against Extreme Events

A bridge may be functionally obsolete or have a deficient structure because of older design codes or features. Such bridges are not unsafe for normal vehicle traffic, but they can be vulnerable to unusual traffic conditions, such as heavy trucks braking in emergency situations or during a disaster. Continued ignorance of deficiencies or lack of sufficient maintenance can cause a bridge to be subjected to hazards and damage, disrupting its reliable and efficient function. A traditional visual inspection cannot effectively explore potential problems.

To evaluate bridge safety, we propose a simulation approach when a five-axle semi-trailer brakes on the bridge during an extreme event. Further, an appropriate retrofit is recommended.

Determination of Dynamic Amplification Factor using site-specific B-WIM Data for a Specific Bridge

The dynamic amplification factor (DAF) is a significant parameter for new bridge design and for verification of existing bridges. The AASHTO code provides a high, conservative value relative to results of field testing. Since, by the Safe and Efficient Transportation Act of 2010, the maximum truck weight was increased from 80 kips to 97
kips, the development of a more accurate DAF for bridge design and for evaluation of existing bridges is required. We propose a simulation method to evaluate the DAF for existing bridges using B-WIM data. This report presents the experimental findings and describes determination of the DAF value using a simulation model.
(1) FAD sensors; (2) Spider; (3) Weighing sensors; (4) Cabinet & panel; (5) Batteries housing; (6) Solar panels; (7) Solar panel installation; (8) Antenna; (9) Camera; (10) PDA

Figure 1. Components of FAD BWIM system
VEHICLE AXLE DETECTION STRATEGY FOR BRIDGE WEIGH-IN-MOTION SYSTEMS

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Abstract

An innovative bridge weigh-in-motion (B-WIM) system deploys an axle detection system involving sensors under the bridge to replace the traditional axle detection above the road surface; this is called FAD (Free-of-Axle-Detector) or NOR (Nothing-On-Road) B-WIM [1]. This technology requires that the transducers be mounted at the bottom of the bridge slab to assess information related to moving vehicle axles. The FAD B-WIM system works well, but only if the system detects axle locations. In the USA, there are challenges for some beam-and-slab bridges, in which the deep girders directly transfer the vehicle loads to the foundations or supports. If vehicle wheels are too close to the girders, the strain of the slab often has an unusually small variation so that axle detection sensors miss the required peak at transverse position. The present report describes a study with alternative strategies for sensor types and sensor installation locations for beam-and-slab bridges using finite element (FE) analysis.

Research Highlights

- We model the sensor types and position arrangement for moving loads.
- We propose sensor types and installation positions for the B-WIM system.
- We report new combination applications of sensors for B-WIM testing.
- The newly proposed approach has improved effectiveness for detection of vehicle axles.

Keywords: Bridge, Weigh-in-Motion, WIM, B-WIM, Axle Detection, FAD, NOR, FE.
Introduction

The conventional bridge weigh-in-motion (B-WIM) algorithm is credited to Moses (1979) [2]. Static axle weights are calculated by minimizing the sum of squares of differences between measured bridge strains and corresponding theoretical strains. The B-WIM system is a popular and portable measuring platform for existing bridges, which are equipped with strain sensors that record transducer readings triggered by moving vehicles. The vehicle speed, axle spacing, axle loads, and gross weight can be measured with acceptable accuracy if there is proper calibration of the B-WIM system. The traditional B-WIM system includes devices for longitudinal strain (bending transducers) and devices for recording information about the vehicle speed and axle spacing. Two axle detectors at each lane are used to measure the times of occurrence, axle spacing, velocity, and vehicle class.

A new B-WIM system called NOR (nothing on the road) or FAD (free of axle detector) has been recently developed. This technology requires that transducers be mounted underneath the bridge slab to recognize the signals from traffic vehicles. The first application, in real-time, was tested on an orthotropic deck with short-span bridges [3] [4]. O’Brien and Žnidarič (2001) [5] verified the effectiveness of the NOR B-WIM system. For the FAD BWIM system, an advantage is that it eliminates all actions on the pavement and consequently saves costs of installation and decreases inconveniences to the road users. This technology has been popular, and there are now more than 1000 B-WIM sites worldwide.

Axle detection is an essential part of the B-WIM system, as it provides vehicle silhouettes, such as axle spacing, velocity, and vehicle type. The FAD B-WIM system is
effective only if axle locations can be detected. This technology works well for bridges, except for those that are beam-and-slab. In 2008, a field test of a B-WIM system was accomplished in Alabama. Field test results for bridges at I-59 and I-459 showed that many vehicles were undetected by the B-WIM system during initial calibration and the following in-service check. Table 1 presents a summary of failed cases.

Orthotropic deck bridges made from stiffened steel exhibit comparatively pronounced strain responses for each axle passage. For the beam-and-slab bridges, the peaks in the slab strains are sensitive to the transverse position of the wheels. In deeper concrete bridges, the strain amplitudes for FAD sensors fail to reveal the effect of individual axles as clearly as that in orthotropic deck bridges, because the beams dispersed the loads to the support [6]. Chatterjee et al. [6] applied a wavelet analytical technique to detect axle locations from measured signals. This method improved the detection of axles, but the results were limited for practical implementation in the current FAD B-WIM system. In order to detect axle locations effectively, the FAD sensor system needed to be enhanced.

**Approach**

*Bridge Weigh-in-Motion*

In previous B-WIM systems, axle detection involved installation of tape switches or pneumatic hoses to the road surface. In recent years, these original systems were replaced with the NOR (Nothing-On-Road) or FAD (Free-of-Axle-Detector) B-WIM system, with sensors mounted to the bottom slab. In the current FAD system, strain transducers detect slab bending when vehicles pass overhead. The current strategy of sensor location is keeping near points of the quarter span for a simply supported span. As a vehicle passes
overhead, a peak strain can be detected by the data acquisition system with a scan rate of 512 Hz. Under excellent road surface conditions, this approach works well for bridges with short and slender spans. A serious problem arises, however, for beam-and-slab bridges (Table 1). The peaks of strains are not sharp due to interactions with other strains or to the fact that most loads are distributed by beams (girders) so that the system misses the peak response.

*Finite Element (FE) Model*

By using finite elemental (FE) analysis, we developed a simulation model for the different sensor types to determine the feasibility of the strain sensor and related sensor locations as an axle detection device. In order to perform the FAD application effectively, the FE model includes the bridge geometry, wheels moving load, and types of strain sensors. In this study, we modeled a simply supported bridge with similar properties of the bridge at US I-459 (Figures 1 and 2). The simulated bridge has a span of 12.19 m (40 feet) with 10 beam girders spaced at 2.13 m (7 feet). In order to simplify the model, rectangular girders were adopted, and the slope of the slab was ignored. This FE model was set up with a girder depth of 0.914 m (3 feet) and a slab thickness of 203 mm (8 inches). An element mesh size of 305 mm x 305 mm (1 ft x 1 ft) for each slab and girder was used. We simulated two concentric loads of 44.5 KN (10 kip) and spacing at 1.22 m (4 feet) (Figure 3), moving from bridge start to end. These configurations were selected based on wheelbase parameters of semi-trailer trucks. In order to model traffic conditions, seven wheel traffic routes were simulated at each line of transverse location from “0” to “7” between girders “F” and “G”. Three types of sensors were simulated as
follows (Figure 4): 1. A local vertical compression strain sensor at the top of girder web (S2 at Girder “F” and S8 at Girder “G”); 2. A global shear sensor at middle of the girder web (S1 at Girder “F” and S9 at Girder “G”); 3. A local bending strain sensor under the slab (sensor S3, S4, S5, S6 and S7). All of these sensors were located at bridge longitudinal location of 4.88 m (16 feet).

The objective of the study was to determine the feasibility and optimal location of strain sensors, and thus to determine the best location for different sensor types in the NOR B-WIM system. With FE modelling, the simulation results for the possible locations of each sensor related to the vehicle load locations are presented in the following results.

**Results**

*Sensor Effective Influence Area*

The FE simulation was accomplished to understand how well the sensors work in the following cases: 1. Two-wheel loads (Figure 3) moving along the line load location “0” (directly above the Girder F), and 2. Two-wheel loads moving along the line load location “4” (at mid-span of slab). Figure 5 shows the FAD sensor location of the bridge at I-459. Figure 6 illustrates the maximum stress contours, which indicate that the higher stresses varied sharply only near the load area between Girder “E” and Girder “G”, while the lighter stress changes were at other areas. Thus, the FAD sensor could cover only its related traffic lanes, and it did not work well for the other traffic lanes, an observation confirmed by field-testing. The results of the testing strain signals (Figure 7) indicated that only the sensors for lane 3 had the identified peaks when wheel loads were on lane 3.
Based on the above results, one sensor cannot cover more than one lane, and each lane should have its own FAD sensors.

*Compression Sensor*

In this FE model, a local compression strain sensor (S2 or S8) was simulated, where its function was used to detect the compression peak stress when wheels pass over the bridge. The compression sensor was installed at the top of the girder web oriented along the vertical direction. If the wheel loads were far away from the sensor location, the local stress in the sensor varied slightly, while it had high peak stress when loads acted directly on the sensor location. Seven line loads were simulated, from location “0” to “7”, to verify the sensor effectiveness with different areas and to determine the effective coverage of sensors. Figure 8 presents the FE simulation results for different load cases. Figures 8(a), 8(b), and 8(c) show that FAD sensor S2 had a clearly identified peak at line load locations “0”, “1” and “2”, and sensor S8 had a clearly identified peak at line load locations “5”, “6” and “7” (Figures 8(f), 8(g), and 8(h)). Figures 8(d) and 8(e) indicate that the peak of sensor S2 (S8) is unacceptable for field practice. From this analysis, the compression sensor S2 (S8) can cover effective areas from location “0” to “3” (“5” to “7”), which is about 1/4 L girder spacing (≈ 2/7 L, L is the spacing of girder).

*Girder Shear Sensor*

The purpose of shear strain sensor was to test the global shear stress and verify effectiveness of the identified shear stress for axle detection. When a wheel is moving along the bridge, it has the shear peak change immediately before and after the sensor
location. In this FE simulation, the shear sensor S1 (S9) was simulated at the mid-depth of the girder web. Seven simulated line loads, from location “0” to “7”, were applied to verify the sensor accuracy at different areas. Figure 9 shows the peak of shear stress at positive (negative) coordinates. Figure 10 summarizes the absolute values of shear stress. As determined by figure, if the error due to the mesh size of the finite elements is ignored, only Figure 10(a) and 10(h) have acceptable peak locations (the simulation load is located at 5.03 m (16.5 feet) and 6.55 m (21.5 feet)) when wheel loads act directly above the girder, while the other six results have unacceptable errors, which may be caused by multiple loads interacting with each other in such a way that the maximum shear stress location shifts when loads distribute from the slab to the girder. Therefore, this shear sensor can be a back-up instrument in the event other sensors fail.

Transverse Bending Sensor

The transverse bending sensor is a common device in current FAD B-WIM systems. It is mounted at the bottom of the slab to detect the transverse bending strain when vehicles pass overhead. In the FE simulation, we simulated five sensors at different locations to test the sensor coverage and effectiveness. The locations of sensors (S3 to S7) are illustrated in Figure 4. Seven load lines, from location “0” to “7”, were simulated. Figure 11(a) shows that only sensor S3 had a peak when the wheel load was directly above the girder (load location “0”); the other four sensors had no identified peak. This phenomenon was also observed in the field B-WIM testing of the bridge US I-459; only 51 out of 128 runs (Table 1) could be identified since the FAD sensor failed to detect the axle information. As indicated in Figures 11(c), 11(d), 11(e) and 11(f), sensors S4, S5,
and S6 recorded sharp peaks when wheels acted at load locations “2”, “3”, “4”, and “5”. The results in Figure 11 suggest that each bending sensor can cover about 1/2 L load area (L is the spacing of girder), i.e., for sensor S5, the coverage is from location “2” to location “5”. Based on comparisons of the results, the bending sensor effectively covers about 1/2 L area, and the best location arrangement of bending sensors is at transverse span from 1/4 L to 3/4 L.

Summary

Sensor Types and Location Arrangement

From analysis of the FE results, three types of sensors can be applied for the FAD B-WIM system: 1) slab bending sensors, 2) girder compression sensors, and 3) girder shear sensors. Each sensor has its effective coverage shown in Figure 12:

1. A bending sensor mounted under the slab can be installed at a range from 1/4 L to 3/4 L, covering about 1/2 L load area (L is the spacing of the girder). For simplicity, in practice, the sensor can be installed at the middle of span, but in order to improve the accuracy and enhance the sharpness of the peak, the sensor can be shifted close to the location of vehicle wheel load if the traffic lanes are confirmed.

2. The girder shear sensor can be installed at middle depth of the girder. Since errors exist, and the shear sharp peak location varies with multi-wheel loads, this type of device can be a back-up instrument.
3. The compression sensor has excellent accuracy for axle peak detection. It can be installed at depth of 1/3 H girder. Figure 11 indicates that the sensor can cover about 1/4 L load area.

4. To detect axle information effectively, the compression sensor should be installed close to the wheel-load span (see Figure 12 for sensor S’c at load P’).

Priority List of Sensor Selection

When only a bending sensor under the slab was applied, the FAD system had an extremely high failure ratio (Table 1). Based on analysis of the FE results, a combination of two types of sensors is required in the FAD system to identify the axle information and maintain the effectiveness of the B-WIM system. The best choice is application of both a bending sensor (Sb) and a compression sensor (Sc) (Figure 12). However, in complex field conditions, if there are skew bridges or if traffic lanes are unclear under the bridge, shear sensors are a good back-up selection to combine with the other two sensors.

Therefore, as an alternative to current practice, two or more sensors can be selected to improve axle detection. If two or more sensor types are selected, different types of sensors in the B-WIM system may detect axle information at the same time. The following guidance determines the priorities in the system:

1. The bending sensor has excellent sensitive strain for the traffic load, and it has superior accuracy and reliability. Its signal should be the first selection from the system.
2. The compression sensor has accuracy when the wheel load is directly above/around bridge girders. If the bending sensor has no identified peak, the compression sensor is the second choice.

3. When above two types of sensors fail at the same time, the shear sensor can be an appropriate backup.

Case Study

The bridge on highway I-459 is located on Sulphur Springs Road in Hoover, Alabama, USA. In 2008, ALDOT conducted B-WIM experiments for initial calibration testing and in-service testing. Ten weighing sensors were mounted on the soffit of girders (one for each girder) on the third span. Eight FAD sensors were mounted directly under the bridge slab for vehicle axle and speed detection. An additional six sensors were installed to determine if vehicle axle information could be detected by the sensors on the diaphragm. The bridge information and sensor layout are shown in Figures 1 and 6, respectively. Of 128 runs, 90% were arranged at lanes 2 and 3 to follow the “real” traffic conditions of the highway. From the initial calibration, a substantial number of runs could not be identified by the system, and some of the identified runs did not provide reliable data to evaluate wheel axles due to the corruption from multiple vehicles and other factors. For an in-service check during the B-WIM experiments, 24 vehicles, preselected from traffic flow and weighed at a weighing station, were subjected to B-WIM testing. Only 15 of these vehicles were detected by the B-WIM system, which failed to identify the vehicle information for others. Table 1 shows the summary of the failed cases.
As seen in the table, fail ratios as high as 60% were observed due to the undetected information resulting from weak signals of the FAD B-WIM system, which was not acceptable for the commercial B-WIM system. Based on analysis of the sensor layouts from this case study, the positions of the FAD sensors were an issue affecting the quality of the signal for axle detection. Figure 6 shows that the locations of the FAD sensors were too close to the girders, and they were far away from the effective coverage of each sensor for typical beam-and-slab bridges. We suggest that the reasons for the high fail ratio of testing were disarrangement of the sensor locations and lack of appropriate sensor types for this beam-and-slab bridge.

Based on the results of the FE analysis for the FAD sensor types and position optimization, the existing layout for the sensors (Figure 6) should be modified, and new types of sensors should be applied to compensate for the coverage of the existing sensors. The adjustments are:

1. Figure 13 shows the modified layout and additional types of sensors. The existing sensors, in particular sensor numbers 3, 2, 17, 23, 10, 11, 9, and 14, need to be shifted from the original position (0.66 m (2.17 feet) from girder) to the central span (1.07 m (3.5 feet) from girder); new compression sensors (A1-A4) need to be installed at the top web of the girder; and/or shear sensors (S1 ~ S4) need to be installed at the mid-web of the girder.

2. At lane 1, the wheel “L-1” location is directly above sensor number 3 (2). If the wheels shift along with the transverse direction during normal traffic, the wheel positions are still within the effective coverage of this sensor. However, if the wheel “L-1” is too close to girder G2, the compression sensor (A1) or shear sensor (S1)
would effectively cover the wheel load. Therefore, with the combination of the bending sensors (numbers 3 and 2), compression sensor (A1), and/or shear sensor (S1), vehicle axle information at Lane 1 could be detected effectively.

3. At Lane 2, the wheel “L-2” is located at L/4 spacing of girder. If the wheel shifts a distance to the right transverse side, sensors 17 and 23 can effectively detect the wheel axle, and if it shifts a distance to the left transverse side near the girder G4, sensor A2 or S2 can effectively cover the wheel locations. The application with combinations of bending sensors and compression sensors also works well to detect the wheel axle information for lane 2.

4. All sensors located under the diaphragm were deleted because no signals were detected.

5. At lanes 3 and 4, the sensors are mirror images of those for lanes 1 and 2; therefore, the analysis is same as for lanes 1 and 2.

With the above modifications, the vehicle axle information will be detected effectively, and this will improve the success ratios for B-WIM operation and enhance reliability of the system.

Conclusions

For the FAD B-WIM system, the combined application of slab bending sensors and girder compression sensors is the best selection for beam-slab bridges. Also, the girder shear sensor is a good back-up choice in field practice. The effective coverage of each type of sensor is demonstrated with the identified peak for axle detection, and optimized location of the axle detector is recommended. With the combination application of multi-
sensors, the priority list is suggested for selection of sensors and determination of vehicle axle in the B-WIM system.

Acknowledgement

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References


Table 1. Summary of failed cases for B-WIM testing.

<table>
<thead>
<tr>
<th>Bridge</th>
<th>Total Runs</th>
<th>Identified Runs</th>
<th>Effective Runs</th>
<th>Fail Ratio (%)</th>
<th>Note</th>
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<td>3</td>
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</tr>
<tr>
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<td>128</td>
<td>51</td>
<td>27</td>
<td>60.16</td>
<td>In-initial testing</td>
</tr>
<tr>
<td>US I-459</td>
<td>24</td>
<td>15</td>
<td>-</td>
<td>38.00</td>
<td>In-service testing</td>
</tr>
</tbody>
</table>
Figure 1. Axle detection at bridge on US I-459.

Figure 2. Bridge FE modeling.
Figure 3. Multiple wheel loads in the FE model.

Figure 4. Location of sensors in the FE model.
Figure 5. Original sensor positions for bridge on US I-459.
Figure 6. Slab stress contour for different load locations.
(a) Load along with Location “0”, (b) Load along with Location “5”

Figure 7. Strain of FAD axle detection for Lane 3.
Figure 8. Compression stress for different load locations.
(a) Load at location 0, (b) Load at location 1, (c) Load at location 2, (d) Load at location 3, (e) Load at location 4, (f) Load at location 5, (g) Load at location 6, (h) Load at location 7.
Figure 9. Girder shear stress for different load locations.

(a) Load at location 0, (b) Load at location 1, (c) Load at location 2, (d) Load at location 3, (e) Load at location 4, (f) Load at location 5, (g) Load at location 6, (h) Load at location 7.
Figure 10. Girder absolute shear stress for different load locations.

(a) Load at location 0, (b) Load at location 1, (c) Load at location 2, (d) Load at location 3, (e) Load at location 4, (f) Load at location 5, (g) Load at location 6, (h) Load at location 7.
Figure 11. Slab transverse local bending stress for different load locations.

(a) Load at location 0, (b) Load at location 1, (c) Load at location 2, (d) Load at location 3, (e) Load at location 4, (f) Load at location 5, (g) Load at location 6, (h) Load at location 7.
Figure 12. Sensor arrangement.
Figure 13. Adjusted and added sensor positions for bridge on US I-459.
FIELD VERIFICATION OF A FILTERED MEASURED MOMENT STRAIN APPROACH TO THE BRIDGE WEIGH-IN-MOTION (B-WIM) ALGORITHM, PART I: SIMULATION MODEL FOR EFFECTS OF B-WIM MOMENT

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42
Abstract

Most of the commercially available bridge weigh-in-Motion (B-WIM) systems are based on an algorithm developed by Moses (1979). The performance of this method is acceptable for estimating gross vehicle weight (GVW), but can be unsatisfactory for estimating single-axle loads. Various factors reduce the reliability of the B-WIM system, and any direct measurement of dynamic increment is sensitive to the estimated accuracy of static axle weights. Therefore, studies on the dynamic increment of load effect need to be conducted to improve the accuracy of B-WIM systems. For multi-span bridges, the corresponding characterizations work similarly with characterizations of frame structures, since the connection joints have the capacity of rotation. In the present investigation, to reflect the behavior of a bridge and determine the actual strain resources from a B-WIM system, we investigated alternative algorithms to calibrate the vehicle weight. In Part I, we explore a simulation method to isolate the true superstructure strain for a multi-span bridge based on site-specific B-WIM data; and, in Part II, we propose a filtered, measured moment strain approach as the B-WIM algorithm.

Research Highlights

- We develop a simulation model to simulate the effects of moving loads.
- We verify the moment of the simulation model based on the B-WIM experimental data.
- We report the multi-span bridge behavior like a frame.
- The newly developed model has improved the understanding of dynamic effects.
Introduction

The main purpose of bridge weigh-in-motion (B-WIM) system is to calibrate vehicle weight and record traffic data using portable technology. The algorithm of Moses [1] and variations have been selected and used in numerous applications of bridge B-WIM testing. The basic principle of the commercial B-WIM algorithm is based on the comparison between measured moments and modeled bending moments. It uses the least squares method to minimize errors between the measured response and the predicted response, then solves the simultaneous equations for vehicle axle weight.

Peters developed the AXWAY [2] and CULWAY systems [3] for weighing trucks and calibrated truck weight by using peak strains from continuous strain data [4]. In 1989, Snyder [5] developed the first commercial B-WIM system based on the Moses algorithm. Dempsey [6], O’Brien [7], Žnidarič [8], and Ojio [10] contributed to the development of the B-WIM system for practice testing. Dempsey et al. [11] tested an orthotropic deck bridge by applying this B-WIM system, and Žnidarič, et al. conducted testing for short-slab bridges [12]. Quilligan et al. [13] developed a two-dimensional algorithm for orthotropic steel decks. In addition, González et al. [14] and Leming et al. [15] introduced a consideration of dynamics into the application. Applications of these B-WIM systems are based on the least-squares errors method of the Moses algorithm. Other approaches have been made to solve this problem [16-19]. Xiao et al. [20] proposed that mounting the longitudinal ribs of an orthotropic box girder bridge to obtain axle weights
with sufficient accuracy. Further, the moving force identification (MFI) theory for predicting axle weights has been developed [16-17; 21-32]. Commercial B-WIM systems still use the Moses algorithm as a basic method, due to system reliability and ease of installation in the field.

Nevertheless, the series of equations of Moses’ algorithm are ill-conditioned. To improve the accuracy of the equations in the B-WIM system, Rowley [33-34] combined the Tikhov regularization method with the original least squares method for determining vehicle weights. O’Brien et al. [35-36] verified that application of the Tikhonov regularization method significantly improved accuracy for calibration of vehicle weights.

Effects such as low signal, environment noise, and dynamic vibration on the bridge can move the measurements from the static response and influence the accuracy of results [7, 37]. With use of B-WIM systems from single orthotropic deck bridges to multi-span bridges or continuous bridges, more applications have been developed. Thus, a higher accuracy for detection of vehicle axle weights and related bridge strains need to be identified for evaluations of bridge integrity.

Current practices and research on the applications of B-WIM are limited to short, single-span, and orthotropic bridges. In this report, we identify the moment strain for a multi-span bridge on US-78 using the B-WIM system.

US-78 Bridge Description

The highway bridge on US-78 at Graysville, Alabama, USA, has three single-spans of 12.8 m (42 ft) each and is supported by two square columns of 945 mm x 945 mm (3 feet x 3 feet) at each bent [38]. The B-WIM testing was conducted by the
Alabama Department of Transportation (ALDOT) in 2008 [39]. The sensors were mounted underneath at the mid-spans of each girder. The bridge information and field testing are shown in Figures 1 and 2. The initial calibration test was performed under test condition (R1-I) according to the European specifications for B-WIM [40]. The two representative vehicles for testing were semi-trailers with a loading capacity of 355.8 KN (80,000 lbs, fully loaded, five-axle trailer truck) as pre-weighed trucks from the ALDOT. The effective signals of 10 runs for each lane were evaluated for both vehicles. Table 1 provides detailed information for these vehicles.

Simplified Model

According to American Association of State Highway and Transportation Officials (AASHTO) manual [41-42] and the standard drawings of ALDOT [38], the multi-span bridge of bridge US-78 was designed as a simply-supported bridge at each span (Figure 3). In this figure, all ends of girders are represented with pin connections. Therefore, the moment at the sensor location for the vehicle moving loads is defined in Equation 1

\[
M_{SM}^{ST} = \begin{cases} 
\sum_{i=1}^{n} \sum_{j=1}^{m} (P_{i,j}(x_{i,j} - \frac{m-1}{L_3} \sum_{j=0}^{m-1} w_{j+1} - \frac{2}{L_3} \sum_{s=1}^{3} L_s))(1 - \frac{a}{L_3}) & a \geq \frac{3}{3} L_s - x_{i,j} \text{(testing span)} \\
\sum_{i=1}^{n} \sum_{j=1}^{m} (P_{i,j}a(1 - \frac{1}{L_3} (x_{i,j} - \frac{m-1}{L_3} \sum_{j=0}^{m-1} w_{j+1} - \frac{2}{L_3} \sum_{s=1}^{3} L_s)) \quad a < \frac{3}{3} L_s - x_{i,j} \text{(testing span)} & (1)
\end{cases}
\]

where, \(P_{i,j}\) is the wheel load at the scan number \(i\) of the B-WIM system for wheel axle number \(j\), \(i = ft\), \(t\) is the scan time of the B-WIM system (seconds), \(f\) is the trigger frequency of the sensor, \(n\) is the total scan number of sensors, \(m\) is the total number of
wheel axles, $s$ is the span number, $a$ is the distance of the sensor from the end of bridge span 3, and $w_{j,j+1}$ is the wheelbase between wheel axle $j$ and $j+1$, $w_{0,1} = 0$.

Proposed Simulation Model

In practice, the simplified structural model of bridge US-78 did not reflect the behavior of the bridge. In order to identify the true moment, the proposed model is presented in Figure 4. For this model, the semi-rigid rotational joint, boundary condition, and flexural stiffness are discussed as follows.

Semi-rigid Rotational Joint

In reality, the influence line of a simply supported bridge is not triangular and may be between the simply supported and fixed cases [8]. The reason is that the supported joints are not in “ideal” single support, with the capacity to transfer the forces and moments. For the connection of a simply-supported bridge, traditional designs neglect the real behavior of connections. Thus, the idealization of a pinned connection was used in the design. However, the predicted response of a bridge may not be realistic. In practice, most connections have the rotational capacity contributing to structure displacements. As shown in Figure 5, for the fixed joint of the bridge US-78, the vertical rebar, bridge girders, and the support columns work together to restrain the joint. However, this connection cannot fully transfer all forces. Therefore, these types of joints can be represented as semi-rigid rotational joints. The semi-rigid connection can be simulated in the analysis of beam-column connections. This flexible connection behavior affects the internal force distribution of the frame structure, and a more reliable prediction
of frame behavior can be obtained by use of a semi-rigid rotational spring.

Therefore, the multi-span of bridge US-78 behaves similarly to a frame. A sophisticated method of frame analysis was adopted in this research. A linear representation of the spring was developed to analyze the bridge frame with a semi-rigid connection for each beam element, as shown in Figure 6. The effects of the connection flexibility are modeled as rotational spring constants $S_j$ and $S_k$, where $j$ and $k$ are the ends of a frame element, and $\Phi_j$ and $\Phi_k$ are rotations incurred by rotational springs.

According to first-order analysis, the stiffness matrix of a member with semi-restraint at the ends can be represented by the correction stiffness matrix based on the rigid connections [43-45].

\[
\begin{bmatrix}
\frac{EA}{L} & 0 & 0 & -\frac{EA}{L} \\
\frac{12EI (\alpha_1 + \alpha_2 + \alpha_3 \alpha_2)}{L^3} & \frac{6EI (2\alpha_1 + \alpha_2 \alpha_3)}{L^3} & 0 & 0 \\
0 & \frac{12EI (\alpha_1 + \alpha_2 + \alpha_3 \alpha_2)}{L^3} & \frac{6EI (2\alpha_1 + \alpha_2 \alpha_3)}{L^3} & 0 \\
0 & 0 & 0 & \frac{12EI (\alpha_1 + \alpha_2 + \alpha_3 \alpha_2)}{L^3} \\
\end{bmatrix}
\]

\[
\text{SYM.}
\]

(2)

If the axle and shear deformations are neglected, the stiffness matrix can be modified as in Equation 3.

\[
k^* = \begin{bmatrix}
\frac{4EI}{L} & \frac{3\alpha_1}{4-\alpha_1\alpha_2} & \frac{2EI}{L} & \frac{3\alpha_1\alpha_2}{4-\alpha_1\alpha_2} \\
\frac{2EI}{L} & \frac{3\alpha_1\alpha_2}{4-\alpha_1\alpha_2} & \frac{4EI}{L} & \frac{3\alpha_2}{4-\alpha_1\alpha_2} \\
\frac{L}{4-\alpha_1\alpha_2} & \frac{2EI}{L} & \frac{3\alpha_1\alpha_2}{4-\alpha_1\alpha_2} & \frac{4EI}{L} \\
\frac{L}{4-\alpha_1\alpha_2} & \frac{2EI}{L} & \frac{3\alpha_1\alpha_2}{4-\alpha_1\alpha_2} & \frac{4EI}{L} \\
\end{bmatrix}
\]

(3)

where $EI$ is the flexural stiffness, and $L$ is the length. In equation 4, the parameters $\alpha_1$ and $\alpha_2$ are the fixity factors at each end of the member, and both factors are related with
rotational spring stiffness, $S_j$ and $S_k$.

\[
\alpha_j = \frac{1}{1 + 3EI/S_jL}; \quad \alpha_k = \frac{1}{1 + 3EI/S_kL}
\]  \hspace{1cm} (4)

The fixity factor $\alpha$ determines the stiffness of the connection relative to the attached beam and to the rotational capacity of moments. Following the conventional matrix procedures of displacement for a rigid-jointed frame, the adjusted end-moments, $M_j$ and $M_k$, for the semi-rigid connection member are defined as follows:

\[
M_j = \frac{3\alpha_1(2-\alpha_2)}{4-\alpha_1\alpha_2}M'_j; \quad M_k = \frac{3\alpha_2(2-\alpha_1)}{4-\alpha_1\alpha_2}M'_k
\]  \hspace{1cm} (5)

where $M'_j$ and $M'_k$ are the end moments for a rigid connection.

The adjusted end moment of the member is defined as

\[
\begin{bmatrix}
M_{jk} \\
M_{kj}
\end{bmatrix} = k \begin{bmatrix}
\theta_j \\
\theta_k
\end{bmatrix} + \begin{bmatrix}
M'_j \\
M'_k
\end{bmatrix}
\]  \hspace{1cm} (6)

where $\theta_j$ and $\theta_k$ are the rotations at end elements $j$ and $k$.

In linear elastic analyses, if the connections are assumed to have a linear force-displacement relationship, and the axial and shear deformation of members are ignored, the solution of member moment can be achieved by the set of linear equations

\[
K\mu = M
\]  \hspace{1cm} (7)

here $K$ is the bridge stiffness matrix, $\mu$ is the deformation of bridge, and $M$ is the external moment.

**Boundary Condition**

With moving vehicles, there are bridge horizontal movements and foundation displacements, with the sensitive parameters influencing the testing strains. Thus, two
influences are simulated in the model.

a. Horizontal movement: Figure 7 shows the boundary conditions of the bridge on US-78 at an expansion joint. The connection was designed as the smooth steel plate lay on the smooth steel pad without vertical ties or horizontal restraint at the expansion joint, so that the bridge has the potential to move in the horizontal direction. A horizontal spring is provided to simulate the horizontal movement at joint “A” (Figure 4), and the horizontal spring constant, \( K_H \), is defined as

\[
K_H = 0.5 \mu_b W_b,
\]  

(8)

where \( \mu_b \) is the friction coefficient between the smooth steel plate and the smooth steel pad [46], and \( W_b \) is the bridge weight of the related span.

b. Vertical Displacement: Soil can be simulated as an elastic material for vertical displacement of the foundation. At each foundation location, a vertical spring is provided at joints “A”, “E”, “F” and “D” (Figure 4) to simulate the foundation displacement, where the spring constant can be determined according to the type of soil and types of foundation piers. Using elastic theory, the coefficient of subgrade reaction, \( K_S^* \), is defined as follows [47-50]:

\[
K_S^* = \frac{4E_S^*}{\pi D^*(1 - \nu^*)^2}
\]  

(9)

where \( E_S \) is Yong’s modulus of soil, \( \nu \) is Poisson’s ratio of soil, and \( D \) is pier diameter. * indicates a foundation joint.

*Flexural Stiffness (EI)*

Bridge dynamic behavior responses are related to bridge stiffness. However, the
bridge flexural stiffness \((EI)\) varies with different situations, such as boundary conditions, concrete properties, and the integrity of the bridge structure. The bridge flexural stiffness can be predicted with basic geometries. Although many existing bridges were built 30-40 years ago, the bridge flexural stiffness can change with age, damage, and bridge geometry as constructed with by different vendors. Therefore, the actual flexural stiffness may not match with the design information. Alternatively, the flexural stiffness can be obtained by using the model of bridge single-degree freedom, as seen in Figure 8. Ambient excitation has been suggested as a tool for nondestructive testing of bridges [51] in measuring traffic vibrations. The bridge’s fundamental frequency, \(f\), can be determined as a function of its effective mass \(M_b\) and effective flexural stiffness \(K_b\):

\[
f = \frac{1}{2\pi} \sqrt{\frac{K_b}{M_b}}
\]  

(10)

The natural frequency can be verified, based on the extracted free vibration signals from the B-WIM data (Appendix A). The bridge flexural stiffness, \(EI\), a function of the bending moment, corresponds with varying boundary conditions.

\[
(EI)_n^f = \lambda_n \frac{m f_n^2 L^4}{\pi^2}
\]

(11)

where \(m\) is the bridge unit mass, \(\lambda_n\) is the coefficient for mode \(n\), and \(L\) is the bridge span.

In the present study, only the flexural stiffness \((EI)\) related to the first bending mode is evaluated, so

\[
(EI)_1^f = \lambda_1 \frac{m f_1^2 L^4}{\pi^2}
\]

(12)

The “effective” flexural stiffness, \(EI\), needs to be verified by both \((EI)_1^f\) and
\((EI)^g\) (the results of analysis of the design geometries). The minimum value is adoptable. The effective \(EI\) is defined as follows, and it replaces the \(EI\) of Equations 3 and 4.

\[
EI = \Phi\{(EI)_1^f, (EI)^g\}
\]

where \(\Phi\) indicates the minimum values for \((EI)_1^f\) and \((EI)^g\).

Formula for the Proposed Simulation Model

According to the assumption of the boundary condition and a semi-rigid connection, the structural system of the bridge on US-78 is modeled as a frame structure (Figure 4), subjected to a random wheel \(P_{i,j}\) at location \(x_{i,j}\), where \(i\) is the scan number of the B-WIM system, and \(j\) is the wheel number. By applying the slope-deflection method, the end moments of each member are defined as

\[
M_{AB}(i, j) = \frac{2(EI)_1}{L_4}(\theta_B(i, j)K^1_{AB} + \frac{3\Delta_A(i, j)}{L_4} - \frac{3\Delta_E(i, j)}{L_4}) + M^L_A \beta^1_A
\]

(14)

\[
M_{BA}(i, j) = \frac{2(EI)_1}{L_4}(2\theta_B(i, j)K^1_B + \frac{3\Delta_A(i, j)}{L_4} - \frac{3\Delta_E(i, j)}{L_4}) + M^R_B \beta^1_B
\]

(15)

\[
M_{BE}(i, j) = \frac{2(EI)_4}{L_4}(2\theta_B(i, j) - \frac{3\Delta_H(i, j)}{L_4})
\]

(16)

\[
M_{EB}(i, j) = \frac{2(EI)_4}{L_4}(\theta_B(i, j) - \frac{3\Delta_H(i, j)}{L_4})
\]

(17)

\[
M_{BC}(i, j) = \frac{2(EI)^2}{L_2}(2\theta_B(i, j)K^2_{BC} + \theta_C(i, j)K^2_{BC} - \frac{3(\Delta_F(i, j) - \Delta_E(i, j))}{L_2}) + M^L_C \beta^2_C
\]

(18)

\[
M_{CB}(i, j) = \frac{2(EI)^2}{L_2}(\theta_B(i, j)K^2_{CB} + 2\theta_C(i, j)K^2_{CB} - \frac{3(\Delta_F(i, j) - \Delta_E(i, j))}{L_2}) + M^R_C \beta^2_C
\]

(19)
\[
M_{CF}(i, j) = \frac{2(El)^5}{L_5} \left(2\theta_C(i, j) - \frac{3\Delta_H(i, j)}{L_5}\right)
\]

\[
M_{FC}(i, j) = \frac{2(El)^5}{L_5} \left(\theta_C(i, j) - \frac{3\Delta_H(i, j)}{L_5}\right)
\]

\[
M_{CD}(i, j) = \frac{2(El)_3}{L_3} \left(2\theta_C(i, j)K_C^3 + \frac{3\Delta_F(i, j)}{L_3} - \frac{3\Delta_D(i, j)}{L_3}\right) + M_3^L \beta_C^3
\]

\[
M_{DC}(i, j) = \frac{2(El)_3}{L_3} \left(\theta_C(i, j)K_{CD}^3 + \frac{3\Delta_F(i, j)}{L_3} - \frac{3\Delta_D(i, j)}{L_3}\right) + M_3^R \beta_D^3
\]

where A, B, C, D, E, and F indicate joint numbers; \(\theta_B(i, j)\) and \(\theta_C(i, j)\) denote the rotation at joints B and C for load \(p_{i,j}\), respectively; \(\Delta_H(i, j)\) indicates the horizontal movement for load \(p_{i,j}\); and \(\Delta_A(i, j)\), \(\Delta_E(i, j)\), \(\Delta_F(i, j)\), and \(\Delta_D(i, j)\) indicate the vertical displacements at joints A, E, F, and D for load \(p_{i,j}\), respectively.

The fixity factor, \(\alpha\), and the rotational spring stiffness, \(S\), for each member are defined as follows:

For member 1: \[\alpha_A^1 = \frac{1}{1+3(El)_1/S_{AB}L_1}; \quad \alpha_B^1 = \frac{1}{1+3(El)_1/S_{BA}L_1}\]

For member 2: \[\alpha_A^2 = \frac{1}{1+3(El)_2/S_{BC}L_2}; \quad \alpha_B^2 = \frac{1}{1+3(El)_2/S_{CB}L_2}\]

For member 3: \[\alpha_C^3 = \frac{1}{1+3(El)_3/S_{CD}L_3}; \quad \alpha_D^3 = \frac{1}{1+3(El)_3/S_{DC}L_3}\]

The modified factors of stiffness matrix, \(K\), and adjusted moment factor, \(\beta\), for each member are defined as follows:

For member 1: \[K_A^1 = \frac{3\alpha_A^1}{4-\alpha_A^1\alpha_B^1}; \quad K_{AB}^1 = K_{BA}^1 = \frac{3\alpha_A^1\alpha_B^1}{4-\alpha_A^1\alpha_B^1}; \quad K_A^1 = \frac{3\alpha_B^1}{4-\alpha_A^1\alpha_B^1}; \]

\[\beta_B^1 = \frac{3\alpha_B^1(2-\alpha_A^1)}{4-\alpha_A^1\alpha_B^1}\]
For member 2: 
\[ K_B^2 = \frac{3\alpha_B^2}{4 - \alpha_B^2 \alpha_C^2} \quad K_{BA}^2 = K_{AB}^2 = \frac{3\alpha_B^2 \alpha_C^2}{4 - \alpha_B^2 \alpha_C^2} \quad K_C^2 = \frac{3\alpha_C^2}{4 - \alpha_B^2 \alpha_C^2} \]
\[ \beta_B^2 = \frac{3\alpha_B^2(2-\alpha_C^2)}{4 - \alpha_B^2 \alpha_C^2} \quad \beta_C^2 = \frac{3\alpha_C^2(2-\alpha_B^2)}{4 - \alpha_B^2 \alpha_C^2} \]

For member 3: 
\[ K_C^3 = \frac{3\alpha_C^3}{4 - \alpha_C^3 \alpha_D^3} \quad K_{CD}^3 = K_{DC}^3 = \frac{3\alpha_C^3 \alpha_D^3}{4 - \alpha_C^3 \alpha_D^3} \quad K_C^3 = \frac{3\alpha_D^3}{4 - \alpha_C^3 \alpha_D^3} \]
\[ \beta_C^3 = \frac{3\alpha_C^3(2-\alpha_D^3)}{4 - \alpha_C^3 \alpha_D^3} \]

The fixed end moment of the member for span \( s \) (\( s \) denotes spans No. 1, 2, and 3) is defined as

\[
M_s^L = \begin{cases} 
\frac{P_{i,j}x_{i,j}(L_s - x_{i,j})^2}{(L_s)^2} & \text{When } P_{i,j} \text{ locates at span } s \\
0 & \text{others span}
\end{cases}
\]

\[
M_s^R = \begin{cases} 
\frac{P_{i,j}x_{i,j}^2(L_s - x_{i,j})}{(L_s)^2} & \text{When } P_{i,j} \text{ locates at span } s \\
0 & \text{others span}
\end{cases}
\]

From Figure 4, the equations of each joint and force balance are given

\[ M_{BA} + M_{BE} + M_{BC} = 0 \] (24)

\[ M_{CB} + M_{CF} + M_{CD} = 0 \] (25)

\[ \frac{M_{BE} + M_{EB}}{L_4} + \frac{M_{CF} + M_{FC}}{L_5} = K_H \Delta_H + \mu r P_{i,j} \] (26)

\[ \frac{M_{AB} + M_{BA} + P_{i,j}x_{i,j}}{L_4} + \frac{P_{i,j}(L_2 - x_{i,j}) - (M_{BC} + M_{CB})}{L_2} = K_E^S \Delta_E \] (27)

\[ \frac{M_{CB} + M_{BC} + P_{i,j}x_{i,j}}{L_2} + \frac{P_j(L_3 - x_{i,j}) - (M_{CD} + M_{DC})}{L_3} = K_F^S \Delta_F \] (28)
\[
P_i(L_1 - x_{i,j}) - (M_{BA} + M_{AB}) = K_A^S \Delta_A
\]

\[
P_{i,j}x_{i,j} + (M_{CD} + M_{DC}) = K_D^S \Delta_D
\]

Substituting Equations 14 – 23 into Equations 24 – 30, we obtain

\[
\frac{(4(EI)_1^3 K_b^3 + 4(EI)_1^3 K_b^3 + 4(EI)_1^3 K_c^3 + 4(EI)_1^3 K_c^3) \theta_b}{L_2^2} + \frac{2(EI)_2^3 K_b^3 \theta_c}{L_2^2} - \frac{2(EI)_2^3 K_c^3 \theta_c}{L_2^2} - \frac{6(EI)_4^3 K_b^3}{L_4^2} \Delta_h + \frac{6(EI)_4^3 K_c^3}{L_4^2} \Delta_h - \frac{6(EI)_2^3 K_b^3}{L_2^2} \Delta_h - \frac{6(EI)_2^3 K_c^3}{L_2^2} \Delta_h = -M^b_b \beta^b_b - M^b_c \beta^b_c
\]

\[
\frac{2(EI)_2^3 K_b^3 \theta_b}{L_2^2} + \frac{(4(EI)_1^3 K_b^3 + 4(EI)_1^3 K_c^3 + 4(EI)_1^3 K_c^3) \theta_b}{L_2^2} - \frac{2(EI)_2^3 K_b^3 \theta_b}{L_2^2} - \frac{6(EI)_4^3 K_b^3}{L_4^2} \Delta_h + \frac{6(EI)_4^3 K_c^3}{L_4^2} \Delta_h - \frac{6(EI)_2^3 K_b^3}{L_2^2} \Delta_h - \frac{6(EI)_2^3 K_c^3}{L_2^2} \Delta_h
\]

\[
\frac{6(EI)_4^3 \theta_b}{L_4^2} + \frac{6(EI)_4^3 \theta_c}{L_4^2} - \frac{12(EI)_4^3 \theta_c}{L_4^2} + \frac{12(EI)_5^3 \theta_c}{L_5^2} + K_{S}^{\prime} \Delta_h = \mu_i P_{i,j}
\]

\[
\frac{(4(EI)_1^3 K_b^3 + 2(EI)_1^3 K_b^3 + 4(EI)_1^3 K_c^3 + 2(EI)_2^3 K_c^3) \theta_b}{L_2^2} - \frac{4(EI)_2^3 K_b^3 + 2(EI)_2^3 K_c^3 \theta_c}{L_2^2} + \frac{12(EI)_2^3 K_c^3}{L_2^2} \Delta_h - \frac{12(EI)_2^3 K_c^3}{L_2^2} \Delta_h = -\left(\frac{M^b_b \beta^b_b + M^b_c \beta^b_c}{L_1^2} + \frac{L_1^2}{L_1^2} P_{i,j}x_{i,j} + \frac{M^b_c \beta^b_c}{L_2^2} + \frac{M^b_b \beta^b_b}{L_2^2} - \frac{P_{i,j} (L_2 - x_{i,j})}{L_2}\right)
\]

\[
\frac{(4(EI)_2^3 K_b^3 + 2(EI)_2^3 K_b^3 + 4(EI)_2^3 K_c^3 + 2(EI)_2^3 K_c^3) \theta_b}{L_2^2} - \frac{4(EI)_2^3 K_b^3 + 2(EI)_2^3 K_c^3 \theta_c}{L_2^2} + \frac{12(EI)_2^3 K_c^3}{L_2^2} \Delta_h - \frac{12(EI)_2^3 K_c^3}{L_2^2} \Delta_h = -\left(\frac{M^b_b \beta^b_b + M^b_c \beta^b_c}{L_2^2} + \frac{L_2^2}{L_2^2} P_{i,j}x_{i,j} + \frac{M^b_c \beta^b_c}{L_2^2} + \frac{M^b_b \beta^b_b}{L_2^2} - \frac{P_{i,j} (L_2 - x_{i,j})}{L_2}\right)
\]

\[
\frac{(4(EI)_2^3 K_b^3 + 2(EI)_2^3 K_b^3 + 4(EI)_2^3 K_c^3 + 2(EI)_2^3 K_c^3) \theta_b}{L_2^2} - \frac{4(EI)_2^3 K_b^3 + 2(EI)_2^3 K_c^3 \theta_c}{L_2^2} + \frac{12(EI)_2^3 K_c^3}{L_2^2} \Delta_h - \frac{12(EI)_2^3 K_c^3}{L_2^2} \Delta_h = -\left(\frac{M^b_b \beta^b_b + M^b_c \beta^b_c}{L_2^2} + \frac{L_2^2}{L_2^2} P_{i,j}x_{i,j} + \frac{M^b_c \beta^b_c}{L_2^2} + \frac{M^b_b \beta^b_b}{L_2^2} - \frac{P_{i,j} (L_2 - x_{i,j})}{L_2}\right)
\]


\[-\frac{4(EL_1^1)}{L_1^2} K_B^1 + \frac{2(EL_1^1)}{L_1^2} K_A^1 \theta_B^1 - \left(\frac{12(EL_1^1)}{L_1^3} + K_A^1\right) \Delta_A^1 + \frac{12(EL_1^1)}{L_1^3} \Delta_E^1 = \frac{M_1^1 \beta_A^1 + M_2^1 \beta_B^1}{L_1^3} - \frac{P_{1,j} \left(L_1 - x_{i,j}\right)}{L_1^3}\]

(37)

\[\frac{4(EL_2^3)}{L_2^3} K_C^3 + \frac{2(EL_2^3)}{L_2^3} K_D^3 \theta_C^3 + \left(\frac{12(EL_2^3)}{L_2^3} + K_D^3\right) \Delta_D^3 = -\left(\frac{M_1^3 \beta_C^3 + M_2^3 \beta_D^3}{L_2^3}\right) - \frac{P_{1,j} \left(L_1 - x_{i,j}\right)}{L_3^2}\]

(38)

The assembly stiffness matrix \( K \) for the frame structure is

\[
K = \begin{bmatrix}
    a_{11} & a_{12} & a_{13} & a_{14} & a_{15} & a_{16} & a_{17} \\
    a_{21} & a_{22} & a_{23} & a_{24} & a_{25} & a_{26} & a_{27} \\
    a_{31} & a_{32} & a_{33} & a_{34} & a_{35} & a_{36} & a_{37} \\
    a_{41} & a_{42} & a_{43} & a_{44} & a_{45} & a_{46} & a_{47} \\
    a_{51} & a_{52} & a_{53} & a_{54} & a_{55} & a_{56} & a_{57} \\
    a_{61} & a_{62} & a_{63} & a_{64} & a_{65} & a_{66} & a_{67} \\
    a_{71} & a_{72} & a_{73} & a_{74} & a_{75} & a_{76} & a_{77}
\end{bmatrix}
\]

(39)

where \( EI \) is the effective flexural stiffness, and \( L \) is the length

\[
a_{11} = \frac{4(EL_1^1)}{L_1^4} K_B^1 + \frac{4(EL_2^3)}{L_2^2} K_C^3 + \frac{4(EL_4^4)}{L_4^2}, \quad a_{12} = -\frac{6(EL_4^4)}{L_4^2}, \quad a_{13} = -\frac{6(EL_2^3)}{L_2^3}, \quad a_{14} = \frac{6(EL_1^1)}{L_1^4},
\]

\[
a_{15} = \frac{6(EL_2^2)}{L_2^2}, \quad a_{16} = -\frac{6(EL_2^2)}{L_2^2}, \quad a_{17} = 0;
\]

\[
a_{21} = \frac{2(EL_2^2)}{L_2^2} K_B^2, \quad a_{22} = \frac{4(EL_2^2)}{L_2^3} K_C^2 + \frac{4(EL_3^3)}{L_3^3}, \quad a_{23} = -\frac{6(EL_3^3)}{L_3^3}, \quad a_{24} = 0,
\]

\[
a_{25} = \frac{6(EL_2^2)}{L_2^2}, \quad a_{26} = -\frac{6(EL_2^2)}{L_2^2}, \quad a_{27} = -\frac{6(EL_2^2)}{L_2^2};
\]

\[
a_{31} = \frac{6(EL_2^2)}{L_2^2}, \quad a_{32} = \frac{6(EL_2^2)}{L_2^2}, \quad a_{33} = -\frac{12(EL_4^4)}{L_4^3} + \frac{12(EL_3^3)}{L_3^3} + K_B^1, \quad a_{34} = 0, \quad a_{35} = 0, \quad a_{36} = 0, \quad a_{37} = 0,
\]

\[
a_{41} = \frac{4(EL_1^1)}{L_1^4} K_B^1 + \frac{2(EL_1^1)}{L_1^2} K_A^1 - \frac{4(EL_2^2)}{L_2^3} K_C^2 - \frac{2(EL_2^2)}{L_2^3} K_C^2, \quad a_{42} = -\frac{4(EL_2^2)}{L_2^3} K_C^2 - \frac{24(EL_2^2)}{L_2^3} K_C^2,
\]

\[
a_{43} = 0, \quad a_{44} = \frac{12(EL_1^1)}{L_1^4}, \quad a_{45} = -\frac{12(EL_1^1)}{L_1^4} + \frac{12(EL_2^2)}{L_2^3} - K_C^2, \quad a_{46} = \frac{12(EL_2^2)}{L_2^3}, \quad a_{47} = 0;
\]

\[\text{56}\]
\[
a_{51} = \frac{4(EI)}{L_2^2} K_B^2 + \frac{2(EI)}{L_2^2} K_{CB}, \quad a_{52} = \frac{4(EI)}{L_2^2} K_C^2 + \frac{2(EI)}{L_2^2} K_{BC}^2 - \frac{4(EI)}{L_3^3} K_C^3 - \frac{2(EI)}{L_3^3} K_{CD}^3,
\]

\[
a_{53} = 0, a_{54} = 0, a_{55} = \frac{12(EL)}{L_2^3}, a_{56} = -\frac{12(EL)}{L_2^3} + \frac{12(EL)}{L_3^3} - K_F, a_{57} = \frac{12(EL)}{L_3^3};
\]

\[
a_{61} = -\frac{4(EL)}{L_4^4} K_B^4 - \frac{2(EL)}{L_4^4} K_{BA}^4, a_{62} = 0, a_{63} = 0, a_{64} = -\frac{12(EL)}{L_4^3} - K_A, a_{65} = \frac{12(EL)}{L_4^3}, a_{66} = 0
\]

\[
a_{67} = 0
\]

\[
a_{71} = 0, a_{72} = \frac{4(EL)}{L_3^3} K_C^3 + \frac{2(EL)}{L_3^3} K_{CD}^3, a_{73} = 0, a_{74} = 0, a_{75} = 0, a_{76} = \frac{12(EL)}{L_3^3}, a_{77} = -\frac{12(EL)}{L_3^3} - K_D
\]

and the joint external moment \(M(i,j)\) for the frame structure is

\[
M(i, j) = \begin{bmatrix}
-M_i^R \beta_i^1 - M_{ij}^R \beta_i^2 \\
-M_j^R \beta_j^2 - M_{ij}^R \beta_j^3 \\
\mu_i P_{ij} \\
\frac{-M_i^R \beta_i^1 - M_i^R \beta_i^1}{L_4} - \frac{P_{ij} x_{i,j}}{L_4} + \frac{M_i^R \beta_i^2}{L_2} + \frac{M_i^R \beta_i^2}{L_2} - \frac{P_{ij} (L_2 - x_{i,j})}{L_2} \\
\frac{-M_j^R \beta_j^2}{L_2} - \frac{M_j^R \beta_j^2}{L_2} - \frac{P_{ij} x_{i,j}}{L_2} + \frac{M_j^R \beta_j^3}{L_3} + \frac{M_j^R \beta_j^3}{L_3} - \frac{P_{ij} (L_3 - x_{i,j})}{L_3} \\
\frac{M_i^L \beta_i^1}{L_4} + \frac{M_i^R \beta_i^1}{L_4} - \frac{P_{ij} (L_4 - x_{i,j})}{L_4} \\
\frac{-M_j^L \beta_j^3}{L_3} - \frac{M_j^R \beta_j^3}{L_3} - \frac{P_{ij} x_{i,j}}{L_3}
\end{bmatrix}
\]  

(40)

Thus, following linear Equation 7, the \(M-u\) relationship is defined as
\[
\begin{bmatrix}
\theta_B(i, j) \\
\theta_C(i, j) \\
\Delta_H(i, j) \\
\Delta_A(i, j) \\
\Delta_E(i, j) \\
\Delta_F(i, j) \\
\Delta_D(i, j)
\end{bmatrix} = [M(i, j)]
\]  

(41)

Solving Equation 41, each parameter can be obtained

\[
\begin{bmatrix}
\theta_B(i, j) \\
\theta_C(i, j) \\
\Delta_H(i, j) \\
\Delta_A(i, j) \\
\Delta_E(i, j) \\
\Delta_F(i, j) \\
\Delta_D(i, j)
\end{bmatrix} = [K]^{-1}[M(i, j)]
\]  

(42)

Substituting the results of equation 42 into equations 14-23, the end moments of each beam are obtained. Finally, the moment at the sensor location of span 3, \(M_{PM}(i, j)\), for any random wheel load \(P_{i, j}\), is equal to

\[
M_{PM}(i, j) = \begin{cases} 
\frac{a}{L_3} (P_{i, j} (x_{i, j} - \sum_{x=1}^{2} L_x) + M_{DC}(i, j) - M_{CD}(i, j)) & a \leq \sum_{x=1}^{3} L_x - x_{i, j} \\
\frac{a}{L_3} (P_{i, j} (\sum_{x=1}^{3} L_x - x_{i, j}) (L_3 - a) + M_{DC}(i, j) - M_{CD}(i, j)) & a > \sum_{x=1}^{3} L_x - x_{i, j}
\end{cases}
\]  

(43)

And, for the multi-wheel vehicle moving on the bridge, the final moving moment at the sensor, \(M_{PM}\), is

\[
M_{PM} = \begin{cases} 
\sum_{i=1}^{a} \sum_{j=1}^{m} \left( \frac{a}{L_3} \left[ P_{i, j} (x_{i, j} - \sum_{x=1}^{2} L_x) + M_{DC}(i, j) - M_{CD}(i, j) \right] \right) & a \leq \sum_{x=1}^{3} L_x - x_{i, j} \\
\sum_{i=1}^{a} \sum_{j=1}^{m} \left( \frac{a}{L_3} \left[ P_{i, j} (\sum_{x=1}^{3} L_x - x_{i, j}) (L_3 - a) + M_{DC}(i, j) - M_{CD}(i, j) \right] \right) & a > \sum_{x=1}^{3} L_x - x_{i, j}
\end{cases}
\]  

(44)
where $PM$ indicates proposed model, and $M_{CD}(i,j)$ and $M_{DC}(i,j)$ are the end moments of elements from Equations 22 and 23, respectively.

**Simulation of Dynamic Effects on the Proposed Model**

*Dynamic moment of the frame structure*

When a vehicle moves on a bridge, the interaction between the road profile, the vehicle suspension system, and the bridge vibration influence the reading of the sensor. The bridge vibrations include longitudinal and transverse vibrations. Therefore, the total dynamic moment for proposed model is defined as the sum of the moment from the frame structure and the dynamic vibration moment for any random wheel load $P_{i,j}$

$$M_{PM}^{DY}(i,j) = M_{PM}(i,j) + M_{PM}^{V}(i,j)$$  \hspace{1cm} (45)

In addition, for multi-wheeled vehicles moving on the bridge, the final moving moment at the sensor is defined as

$$M_{PM}^{DY} = \sum_{i=1}^{n} \sum_{j=1}^{m} [M_{PM}(i,j) + M_{PM}^{V}(i,j)]$$  \hspace{1cm} (46)

Where $M_{PM}^{V}(i,j) = M_{PM}^{LV}(i,j) + M_{PM}^{TV}(i,j)$  \hspace{1cm} (47)

where $DY$ indicates dynamic moment, $PM$ indicates proposed model, $V$ indicates vibration moment, $LV$ indicates longitudinal vibration, and $TV$ indicates transverse vibration.

*Vibration*

Once the vehicle tires touch the bridge, the dynamic load activates the bridge, and the bridge responds with the activated force. The bridge testing beam is assumed to be an
Euler-Bernoulli beam with moving load $P_{i,j}$. The equation of motion can be written as

$$ M \dddot{Y}(i, j) + C \ddot{Y}(i, j) + K Y(i, j) = P_{i,j} \tag{48} $$

where $M$ is mass per unit length, $C$ is damping of the beam, $K$ is the stiffness of Euler-Bernoulli beam, and $Y(i,j)$ is the displacement function of the beam.

Two effects of vibration are considered in the dynamic model: longitudinal and transverse vibration.

**Longitudinal Vibration**

The longitudinal vibration includes free vibration and forced vibration.

**Free vibration:** Before the vehicle wheel touches or after it leaves the testing span, the testing span has no activated load, $P_{i,j} = 0$, and this is defined as free vibration and equation 48 is

$$ M \dddot{Y}(i, j) + C \ddot{Y}(i, j) + K Y(i, j) = 0 \tag{49} $$

The solution of the differential equation 49 is defined in Equation 50 [52],

$$ Y(i, j)^{\text{Free}} = C e^{-\frac{\omega}{f} j} \cos(\omega_D \frac{i}{f} - \alpha_{i,j}) \tag{50} $$

$$ C = \sqrt{y_0^2 + \left(\frac{\nu_0^2 + y_0 \xi \omega}{\omega_D^2}\right)^2} \tag{51} $$

where $i$ is the scan number of the B-WIM system, $f$ is its trigger frequency, time $t = i/f$, $\omega$ is the natural frequency of the bridge, $\omega_D$ is the damping frequency of bridge, $\xi$ is the damping ratio of frequency, $y_0$ is the initial displacement of the bridge, and $\nu_0$ is the initial vibration velocity of the bridge.

While, when the tire arrives at a bridge expansion joint, the gap of the joint will cause
the tire to impact the bridge in the horizontal direction, and this sudden impact may increase or decrease the free vibration amplitude, and change the vibration direction. Therefore, another instant impact factor at the expansion joint gap, \( D_{i,j} \), is defined as

\[
D_{i,j} = 1 + \frac{H_{i,j}^l}{W_{i,j}} \tag{52}
\]

where \( H_{i,j}^l \) is the horizontal impact force at the expansion gap, which is related to the gap of expansion joint and to vehicle axle weight, speed, and tire pressure. Only when \( i \) is located at the expansion joint, the \( H_{i,j}^l \) has a value; at other locations, \( H_{i,j}^l = 0 \). \( W_{i,j} \) is the wheel axle weight.

Incorporating the instant impact factor \( D_{i,j} \) into Equation 50, this equation becomes

\[
y(i, j)^{\text{Free}} = C D_{i,j} e^{-\xi_0 \frac{t}{f}} \cos(\omega_D \frac{t}{f} - \alpha_{i,j}) \tag{53}
\]

**Forced vibration:** When the vehicle is acting on the testing bridge, this vibration is simulated as forced vibration with periodic excitations \( P_{i,j} \) (Equation 48). The forced vibration displacement, \( Y(i, j)^{\text{Forced}} \), is defined as [52]

\[
Y(i, j)^{\text{Forced}} = \frac{P_{i,j}}{k} e^{-\xi_0 \frac{t}{f}} \cos(\sigma \frac{t}{f} + \beta(i, j) - \theta(i, j)) \cdot \frac{1}{\sqrt{(1 - r^2)^2 + (2r \xi_0)^2}} \tag{54}
\]

where \( \sigma \) is the frequency of the motion, \( \omega \) is the natural frequency of the bridge, \( \beta \) and \( \theta \) are phase angles, \( k \) is the stiffness constant of the bridge, and \( r = \sigma / \omega \) is the frequency ratio.

Therefore, the related longitudinal vibration moment can be defined as
\[ M_{FM}^{TV}(i, j) = A_{F, j}^{Free} y(i, j)^{Free} + A_{F, j}^{Forced} y(i, j)^{Forced} \]  

(55)

where the coefficient between moment and displacement is

\[ A_{F, j}^{Free} = \frac{12(EL)^3a}{(L_3a^2 + aL_3^2 - a^3)} \]

\[ A_{F, j}^{Forced} = \begin{cases} 
\frac{6(EL)_3}{(2L_3x_{i,j} - x_{i,j}^2 - (L_3 - a)^2)} & a \geq \sum_{i=1}^{\frac{n}{s}} L_s - x_{i,j} \\
\frac{6(EL)_3}{(L_3^2 - a^2 - x_{i,j}^2)L_3} & a < \sum_{i=1}^{\frac{n}{s}} L_s - x_{i,j}
\end{cases} \]

Transverse Vibration

Figure 9 shows the transverse vibration model and signals. When a vehicle travels in one lane, and another lane is empty (Figure 9a), this unbalanced load activates the transverse vibration, i.e., girders 1 and 2 have negative deformations of \( Y(i, j)_1 \) and \( Y(i, j)_2 \), respectively, and girders 3 and 4 have positive deformations of \( Y(i, j)_3 \) and \( Y(i, j)_4 \), respectively. Figure 9b shows the strains of girder 4 from B-WIM testing of the bridge on US-78. Figure 9c shows the 3-D plots of strains from all girders (looking from the top). The skewed signals demonstrate that transverse vibrations exist in bridge vibrations. As indicated in the figure, transverse vibrations influence the strain sensor readings at each girder. From Figure 9a, since each girder has the vertical deformation \( Y(i, j)_r \), the total transverse moment is defined as

\[ M_{PM}^{TV}(i, j) = \sum_{r=1}^{\frac{n}{s}} (A_{i, j}^{TV} Y(i, j)_r^{TV}) \]  

(56)

where TV indicates transverse vibration, \( r \) is the girder number, and \( n \) is the total number of girders, \( A_{i, j}^{TV} \) is the moment coefficient for transverse vibration, \( Y(i, j)_r^{TV} \) is the vertical displacement of transverse vibration.
From the bridge transverse model, the transverse deformation of the bridge is anti-symmetric, and the moment coefficient is constant for each girder, so

\[ M_{PM}^{TV}(i, j) = \sum_{r=1}^{4} (A_{i,j,r}^{TV} Y(i, j)_r^{TV}) = 0 \]  \hspace{1cm} (57)

Therefore, for this anti-symmetric bridge, the moment of transfer vibration is zero.

**Effect of Interaction of the Road Profile and the Vehicle Suspension System**

The elastic stiffness of the bridge is usually higher than the stiffness of the vehicle suspension system. For example, the first foundational frequency for the bridge on highway US-78 is 13.65 Hz (Appendix A), while the frequencies of vehicle suspension systems range from 2 to 5 Hz [4, 53]. In order to simplify the simulation model, the vibration interaction between the bridge and a vehicle can be simplified as a vehicle interaction with a rigid road surface (Figure 10). This figure shows the simplified model that includes the interaction of the bridge road roughness and vehicle suspension system. Equations 58, 59, and 60 present the vertical dynamic loads with the interaction between road profile and vehicle suspension system

\[ U_g = \sum_{r=1}^{\infty} (U_{go,r} \sin(\omega_r + \phi_r)) \]  \hspace{1cm} (58)

\[ F^D_g(i, j) = -W(i, j) + F_K(i, j) + F_C(i, j) \]  \hspace{1cm} (59)

\[ F^D_g(i, j) = -W(i, j) + \sum_{r=1}^{\infty} [U_{st,r}(i, j) R_{st,r}(i, j) K_V (\sin(\omega_r t - \phi_r) + C_V \omega_r \cos(\omega_r t - \phi_r))] \] \hspace{1cm} (60)

where \( U_{go} \) is the displacement amplitude of harmonic ground motion; \( U_{st} \) is the wheel displacement; and \( R_d \) is the deformation response factor; \( r \) is the \( r-th \) of the road profile; \( \omega \) and \( \phi \) are the frequency and phase angle of road profile, respectively; \( K_V \) is the
combined stiffness of vehicle suspension system; and $C_v$ is the combined damping of the vehicle suspension system.

Therefore, the effects from interaction of the road profile and the vehicle suspension system are simulated in Equation 45 by evaluating $M_{PM}(i, j)$, which can be obtained by substituting $P_{i,j}$ of Equation 40 with $F_g^D(i, j)$.

**Horizontal Friction Load**

The vehicle tire acts above the bridge surface with the friction load (Figure 11), which relies on the friction coefficient. This friction includes two statuses: tire rolling friction and static friction (fully brake) [46]. Under normal traffic conditions, the tire rolling friction coefficient $\mu$ is about 0.01-0.015 [54], and at fully brake status, the static friction coefficient $\mu_r$ can be a maximum value of 0.8 for heavy trucks [46]. The friction load $F_f$, a function of the tire reacting force, is defined in Equation 61, and it replaces the static friction load of Equation 40.

$$F_f^D(i, j) = \mu_r F_g^D(i, j)$$  \hspace{1cm} (61)

where $F_g^D(i, j)$ is the dynamic load in Equation 60.

Therefore, the horizontal friction load effects are simulated in Equation 45 by evaluating $M_{PM}(i, j)$, which can be obtained by substituting $\mu_r P_{i,j}$ of Equation 40 with $\mu_r F_g^D(i, j)$. 
**Time Delay**

Time is required to transfer energy from the activated locations of the vehicle wheel to the testing sensors by the concrete shear wave, the speed for which is about 1067 ~ 1829 m/s (3500 ~ 6000 ft/s) [55-56], and the testing sensor frequency is 512 Hz. Therefore, there will be a time delay for the event. For example, if a sensor location is 30.5 m (100 ft) away from the wheel location, there is a time delay, \( \Delta t \), of about 0.019 ~ 0.027 seconds to transfer the wheel force to the sensors, thus delay steps of the sensor are about 8.7 ~ 14.7, which indicates that the sensor cannot obtain the wheel force response at the same time. It also indicates that multi-wheel forces cannot arrive to the sensors at the same time because of the wheelbase (distance between two axles). Thus, the actual scan time from the B-WIM system will be different from the time of the idealization model. Therefore, the time difference or distance difference needs to be adjusted to fit the moment strains obtained from the sensors. So the effective time, \( t'_{i,j} \), for the testing sensor is defined as

\[
t'_{i,j} = t_{i,j} - \frac{L^a_{i,j}}{V_s}
\]  

(62)

where \( L^a_{i,j} \) is the distance between the sensor and the wheel load \( P_{i,j} \), and \( V_s \) is the concrete shear wave speed.

The adjusted location, \( x'_{i,j} \), for wheel load \( P^D_{i,j} \), is defined as follows, and it replaces the wheel location, \( x_{i,j} \), in Equations 40 to 44.

\[
x'_{i,j} = V_{i,j} t'_{i,j} = V_{i,j} (t_{i,j} - \frac{L^a_{i,j}}{V_s})
\]  

(63)
where \( V_{i,j} \) is the vehicle velocity at scan number \( i \) of the B-WIM system for the wheel axle \( j \), and \( t(i,j) \) is scan time of the B-WIM system.

The time delay effects are simulated in Equation 45 by evaluating \( M_{PM}(i,j) \), which can be obtained by substituting \( x_{i,j} \) of Equation 44 with \( x_{i,j}' = V_{i,j}(t_{i,j} - \frac{L_{i,j}^a}{V_x}) \).

Based on the simulation of all the factors described above, equation 45 can be represented as follows:

\[
M_{PM}^{LV} = \sum_{i=1}^{n} \sum_{j=1}^{m} \left( \frac{a}{L_3} P_{i,j} (x_{i,j}' - \frac{2}{3} L_a) \right) \\
+ \sum_{i=1}^{n} \sum_{j=1}^{m} \left( \frac{2EI_3}{L_3} \left[ \theta_C(i,j) K_{CD}^2 + \frac{3A_F(i,j)}{L_3} - \frac{3A_D(i,j)}{L_3} \right] + \left( \frac{P_{i,j} x_{i,j}'^2 (L_a - x_{i,j})}{(L_a)^2} \right) \beta_D^3 \right) \\
- \sum_{i=1}^{n} \sum_{j=1}^{m} \left( \frac{2EI_3}{L_3} \left[ 2\theta_C(i,j) K_C^3 + \frac{3A_F(i,j)}{L_3} - \frac{3A_D(i,j)}{L_3} \right] + \left( \frac{P_{i,j} x_{i,j}'^2 (L_a - x_{i,j})^2}{(L_a)^2} \right) \beta_C^3 \right) \\
+ \sum_{i=1}^{n} \sum_{j=1}^{m} \left( M_{PM}^{LV}(i,j) + M_{PM}^{TV}(i,j) \right) \\
\text{subject to } a \leq \sum_{j=1}^{m} L_a - x_{i,j}
\]

\[
M_{PM}^{TV} = \sum_{i=1}^{n} \sum_{j=1}^{m} \left( \frac{a}{L_3} P_{i,j} (\frac{3}{5} L_a - x_{i,j}) (L_3 - a) \right) \\
+ \sum_{i=1}^{n} \sum_{j=1}^{m} \left( \frac{2EI_3}{L_3} \left[ \theta_C(i,j) K_{CD}^2 + \frac{3A_F(i,j)}{L_3} - \frac{3A_D(i,j)}{L_3} \right] + \left( \frac{P_{i,j} x_{i,j}'^2 (L_a - x_{i,j})}{(L_a)^2} \right) \beta_D^3 \right) \\
- \sum_{i=1}^{n} \sum_{j=1}^{m} \left( \frac{2EI_3}{L_3} \left[ 2\theta_C(i,j) K_C^3 + \frac{3A_F(i,j)}{L_3} - \frac{3A_D(i,j)}{L_3} \right] + \left( \frac{P_{i,j} x_{i,j}'^2 (L_a - x_{i,j})^2}{(L_a)^2} \right) \beta_C^3 \right) \\
+ \sum_{i=1}^{n} \sum_{j=1}^{m} \left( M_{PM}^{LV}(i,j) + M_{PM}^{TV}(i,j) \right) \\
\text{subject to } a > \sum_{j=1}^{m} L_a - x_{i,j}
\]

(64)

**WIM Testing Moment**

The bridge on US-78 was subjected to B-WIM testing, and the strain signals were collected and analyzed. Based on the relationship between the moment strain \( \varepsilon = \frac{\sigma}{E} \) and
stress $\sigma = \frac{Mr}{I}$, the B-WIM testing moment at each girder $M_{WIM}^{(k)}$, and the total moment $M_{WIM}$ are defined as

$$M_{WIM}^{(k)} = \sum_{i=1}^{n} \sum_{j=1}^{m} \frac{\varepsilon_k(i, j) (EI)_{k}}{r_k} = \sum_{i=1}^{n} \sum_{j=1}^{m} \frac{\varepsilon_k(i, j) EI D_k}{r_k}$$ (65)

$$M_{WIM} = \sum_{k=1}^{w} \sum_{i=1}^{n} \sum_{j=1}^{m} \frac{\varepsilon_k(i, j) EI D_k}{r_k}$$ (66)

where $\varepsilon_k(i, j)$ is the moment strain for the wheel $i$ at location $j$ of girder $k$, $r_k$ is the bottom arm based on the effective girder cross section, $EI$ is the effective flexural stiffness, $D_k$ is the girder distribution factor, $w$ is the total girder number, $n$ is total scan number of the B-WIM system, and $m$ is the total wheel load number.

Results

Bridge Parameters

Base on the vibration signals of the field B-WIM testing, the bridge damping ratio, $\xi = 0.04$ (Appendix B), and the bridge natural frequency of first bending, $f_1 = 13.65$ Hz (Appendix A), were identified. The bridge flexural stiffness was identified as shown in Table 2. The wheel rolling friction coefficient of $\mu = 0.015$ was applied in this simulation.

To determine the fixity factor of rotational spring, the analysis steps are following:

1. Select one B-WIM testing result and plot the testing moment.
2. In the simulation program trial, the fixity factor ranged from 0 to 1.0 for each member one at a time, the other parameters are automatically revised with the program.
3. Try the moment from proposed model to match the field-testing moment from the
B-WIM system and find the convergence of fixity factor, then finalize the fixity factor of each member.

4. Perform the next run.

Upon applying the initial rotational stiffness for each connection joint, and, by trial and error using 20 testing results, the rotational spring constant, the fixity factor, and the horizontal and vertical spring constants shown in Table 3 and 4 were derived.

**Moment Summaries and Comparisons**

For the bridge on US-78, 20 B-WIM tests were conducted in 2008, and the data were analyzed. Figure 12 shows the moment of the simplified model (SM), the static moment of the proposed model (PMST), the dynamic moment of the proposed model (PMDY), and the moment of B-WIM for one case; the other 19 tests gave similar results (not shown). The result comparisons are presented as follows:

1. This figure shows that the simplified model moment (the related moment influence line was applied by a current commercial B-WIM system) is not close to the B-WIM testing moment curve. At spans 1 and 2, there are no moments for the simplified model. However, there are moments for spans 1 and 2.

2. Both the static moment and dynamic moment from proposed model are close to the B-WIM testing moment curve. When the vehicles cross spans 1 and 2 (negative X coordinates), a negative moment exists in both the proposed model and B-WIM testing. At spans 2 and 3, the static and dynamic moments are similar to B-WIM testing. At span 1, however, there is a large error in the effect of outside resource of activated loads. The results demonstrate that the connection joints are not
ideally simple connections but have moment transfer. In addition, each part of the bridge does not work individually but interacts by each span.

3. From the curves of PMST and PMDY, the fact that some locations have significant moment differences is the reason that vehicle dynamic force varies with the interaction between vehicle suspension system and road roughness. For bridge vibration, the road profile is the most influenced factor. As seen in Figure 13, the static moment from the proposed model is close to the field B-WIM testing results, except that moments are significantly different at expansion joints, since the dynamic forces vary suddenly. From this figure, it is evident that, when the road profiles are applied into the model, the dynamic moment matches better with the actual B-WIM testing. Therefore, the road profile is an important factor influencing the moment strain.

4. Figure 13 summarizes the moment influence line for the proposed model. This figure shows that each moment influence line changes slightly for individual runs by reason of different running routines, speeds, and other factors.

*Sensitivity of Semi-rigid Rotational Joint and Horizontal Movement Restraint*

Figure 14 shows the comparison of the moment influence lines for various parameters. For the simplified model, only the triangle moment influence line occurs at testing span 3; spans 1 and 2 do not show an influence line. For the proposed model, the moment influence lines are in spans 1, 2, and 3. However, if the horizontal movement is restrained, for the proposed model, the moment influence line does not exist for span 1. Thus, this bridge has potential moment in the horizontal direction. This figure also shows
that there is a 12% moment difference at maximum moment location between the simplified model and the proposed model. By comparison, a bridge with the “semi-rigid joint” and “horizontal movement” will be more real for bridge behavior.

*Sensitivity of Shear Wave Speed*

A time delay exists with energy transfer in different materials. For short bridges, the time delay has little influence on moment signals. For long bridges, however, time delay is an issue for the moment influence line, which will cause the entire moment curve to be shifted in a horizontal direction and influence the accuracy of weight calibration. Figure 15 shows that the moment influence line shifts with different shear wave speeds. When the concrete shear wave changes from 914 m/s (3000 ft/s) to an infinite value (indicating no influence of time delay), the moment difference is about 1%. Therefore, the speed of shear wave will cause the moment curve to shift in a horizontal direction with less moment variation. However, this slight moment shift has some effect on the weight calibration (Companion paper II).

*Sensitivity of Foundation Displacement*

Based on results of 20 field tests, the influence lines for different vertical spring constants are compared in Figure 16. From this figure, when the vertical spring constant is greater than 1.5E+9 KN/m, however, there is no significant change for the moment signals. If the spring constant varies from 5.8E+4 KN/m to 1.5E+9 KN/m, the influence line has a significant change at spans 1 and 2 and about 5% moment difference at span 3. Therefore, the foundation vertical displacement or the soil capacity has an effect on the
moment influence line.

Conclusions

A proposed model for the effects of B-WIM testing was developed and verified based on site-specific B-WIM data. Various conditions, such as the vehicle suspension system, multi-span bridges, semi-rigid connections, road roughness, and boundary conditions, were simulated in the model, which demonstrated that semi-rigid rotational joints, the road profile, and foundation displacement significantly influence the dynamic vibration and the signals of B-WIM testing and that time delay has less influence on the moment variation. Moreover, the proposed model demonstrates that the specific multi-span bridge vibration is similar with frame structure and provides a good understanding about the bridge moment effects. Finally, the identified information will be applied to weight calibration research in the companion paper (Part II).

Acknowledgements

The authors gratefully acknowledge funding and support provided by National Science Foundation (NSF) for this research project (CMMI-1100742).
References


Integral bridge, Dissertation.


Table 1. Initial vehicle calibration information for the bridge on highway 78.

<table>
<thead>
<tr>
<th>Vehicle Number</th>
<th>GVV</th>
<th>1st axle (kN)</th>
<th>2nd axle (kN)</th>
<th>3rd axle (kN)</th>
<th>4th axle (kN)</th>
<th>5th axle (kN)</th>
<th>W1-W2 (m)</th>
<th>W2-W3 (m)</th>
<th>W3-W4 (m)</th>
<th>W4-W5 (m)</th>
<th>Traffic Speed Range (km/h)</th>
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</thead>
<tbody>
<tr>
<td>1</td>
<td>351.4</td>
<td>49.15</td>
<td>69.61</td>
<td>71.61</td>
<td>80.95</td>
<td>80.06</td>
<td>4.32</td>
<td>1.35</td>
<td>11.2</td>
<td>1.30</td>
<td>89-105</td>
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<td>2</td>
<td>347.8</td>
<td>44.70</td>
<td>71.17</td>
<td>70.28</td>
<td>81.40</td>
<td>80.28</td>
<td>4.34</td>
<td>1.35</td>
<td>11.1</td>
<td>1.29</td>
<td>89-105</td>
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</table>

Table 2. Bridge properties.

<table>
<thead>
<tr>
<th>Dimension (m)</th>
<th>Bridge Elastic Stiffness (kN-m²)</th>
<th>Natural Frequency (Hz)</th>
<th>Tire Rolling Friction μ</th>
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</thead>
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<tr>
<td>Beam</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>L₁= 12.8</td>
<td>EI₁= 1.07E+07</td>
<td>f₁=13.65</td>
<td>~ 0.015</td>
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<td>L₂= 12.8</td>
<td>EI₂= 1.07E+07</td>
<td>f₂=18.20</td>
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</tr>
<tr>
<td>L₃= 12.8</td>
<td>EI₃= 1.07E+07</td>
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<td></td>
<td></td>
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<tr>
<td>L₄= 9.14</td>
<td>EI₄= 2.5E+06</td>
<td></td>
<td></td>
</tr>
<tr>
<td>L₅= 9.14</td>
<td>EI₅= 2.5E+06</td>
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</table>

Table 3. Semi-rigid rotational spring constant in frame model.

<table>
<thead>
<tr>
<th>Semi-rigid connection joint</th>
<th>Kₜ,AB (MN-m/Rad)</th>
<th>Kₜ,BA (MN-m/Rad)</th>
<th>Kₜ,BC (MN-m/Rad)</th>
<th>Kₜ,CD (MN-m/Rad)</th>
<th>Kₜ,DC (MN-m/Rad)</th>
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</thead>
<tbody>
<tr>
<td>Spring constant</td>
<td>0~85</td>
<td>765~1146</td>
<td>0</td>
<td>255~412</td>
<td>765~1146</td>
</tr>
<tr>
<td>Fixity factor α</td>
<td>0~0.1</td>
<td>0.5~0.6</td>
<td>0</td>
<td>0.25~0.35</td>
<td>0.5~0.6</td>
</tr>
</tbody>
</table>

Table 4. Horizontal and vertical spring constant in frame model.

<table>
<thead>
<tr>
<th>Spring constant (MN/m)</th>
<th>Kₐ</th>
<th>Kₑ</th>
<th>Kₕ</th>
<th>Kₜ</th>
<th>Kₜ₀</th>
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<tbody>
<tr>
<td>5.84E+03</td>
<td>2.92E+03</td>
<td>2.92E+03</td>
<td>5.84E+03</td>
<td>1.46E+00</td>
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</tr>
</tbody>
</table>
Figure 1. Sensor layout on bridge US-78

Figure 2. Field B-WIM testing for the bridge on US-78.

(a) bridge view, (b) field B-WIM testing
Figure 3. Simplified model.

Figure 4. Proposed simulation model for highway US-78.
Figure 5. Fixed boundary conditions of the bridge on US-78.

Figure 6. Displacements of semi-rigid frame element.
Figure 7. Expansion boundary conditions of the bridge on US-78.

Figure 8. Single degree of freedom (SDOF) idealization.
Figure 9. Transverse vibration

(a) transverse vibration model, (b) transverse vibration signal, (c) 3-D transverse vibration signals.
Figure 10. Vehicle and road interaction model.

Figure 11. Horizontal and vertical force model.
Figure 12. Bridge moment comparisons.
Figure 13. Summary of moment influence line for B-WIM testing.
Figure 14. Moment influence line comparison for different models.
Figure 15. Moment influence line comparison for different shear wave speeds.
Figure 16. Moment influence line comparison for different vertical spring constants.
APPENDIX A

Natural Fundamental Frequency

Data for the B-WIM testing strain were exported to signal processing software, DADiSP (DADiSP, 2000) for post-processing and analysis. The steps were as follows:

1. Eliminated forced vibration signals and kept free vibration signals only.
2. For output signal, eliminated data after the free vibration died down.
3. Converted signals into frequency domain by using fast Fourier transformation (FFT).

The natural frequency could be determined by DADiSP software, as shown in Figure A1. The natural frequency of the first mode was 13.65 hz, and that for the second natural frequency was 18.2 hz.

Figure A1. Natural frequency analysis of B-WIM for free vibration
(Double click, enlarge)
APPENDIX B

Bridge Damping Ratio

By applying the free vibration of B-WIM data for the bridge on highway US-78 (Figure B1), the damping ratio of this bridge can be determined. Since the decay of motion is slow for this case of a lightly damped system, multi cycles, \( j \), of the motion were used to determine the damping ratio instead of the successive amplitude. The damping ratio is given by

\[
\delta = \frac{1}{j} \ln \left( \frac{\mu_i}{\mu_i + j} \right) \cong 2\pi\xi
\]  

(B-1)

From Figure 1, \( \mu_i = 1.77 \) and \( \mu_{ij} = 0.86 \) for \( j=4 \), and central coordinate line is 0.32. The damping ratio can be determined by

\[
\delta = \frac{1}{4} \ln \left( \frac{1.77 - 0.32}{0.86 - 0.32} \right) \cong 2\pi\xi
\]

\[\xi = 0.0393\]

Similar results were achieved for analyses of the other 19 cases. Thus, for this bridge, a damping ratio of 4\% is applied to the program.
Figure B1. Damping ratio.
FIELD VERIFICATION OF A FILTERED MEASURED MOMENT APPROACH TO THE BRIDGE WEIGH-IN-MOTION ALGORITHM, PART II: BRIDGE WEIGH-IN-MOTION ALGORITHM

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Format adapted for dissertation
Abstract

Most of the commercially available Bridge Weigh-in-Motion (B-WIM) systems are based on an algorithm developed by Moses (1979). The performance of this method was generally acceptable for estimating gross vehicle weight (GVW) but can be inadequate for estimating single axle load. Two alternative algorithms for the identification of vehicle axle loads on slab-on-girder bridges are presented in this paper. Both algorithms are based on the simulation of the entire bridge, representing a realistic boundary and field environment. The first algorithm includes adjusting the experimental WIM moment at the testing span from simulation model followed by vehicle weight calculation. The second algorithm includes filtering the experimental B-WIM moment from influenced factors (including effects from vibration, dynamic load, and time delay calculated from simulation model) followed by vehicle weight calculation. Both approaches demonstrate significant improvements on the accuracy of weight for the bridge on US-78, applied by WIM testing. The companion paper, Part I, includes details of the proposed simulation model. This paper focuses on the vehicle weight algorithms.

Research Highlights

- We propose two alternative algorithms for vehicle weight identification.
- We identify the influence line from the simulation model of frame structure.
- We demonstrate both algorithms have significant improvement for vehicle weight identification.
Introduction

It is essential that an authority responsible for maintaining a region’s transportation infrastructure has accurate estimates of the characteristics of the traffic fleet that makes use of all components of the infrastructure. This information has many applications, not just concerning planning, designing, and assessing an area’s transportation, but also in relation to the economic and social development of the area. With regards to the bridge stock of this infrastructure, the important characteristics of the traffic fleet include gross weight, axle loads, axle spacing (and wheelbase), and vehicle velocity. Bridge Weigh-in-Motion (B-WIM) systems are a method for obtaining these data.

Weigh-in-Motion

Heavy vehicles can have adverse effects on road surfaces and bridges. Legal limits for loads per axle, Gross Vehicle Weight (GVW), and overload enforcement reduce the number of excessively heavy vehicles on a region’s roads. Whether or not enforcement is effective, it is essential to get unbiased, reliable data on the traffic fleet for design and assessment purposes. Pavement-based WIM systems and Bridge-based Weigh-in-Motion (B-WIM) pro-vide methods of automatically weighing trucks at full highway speeds, and in the case of Nothing-On-Road (NOR) B-WIM, this can even be achieved without the knowledge of the vehicle drivers.
**Bridge-based Weigh-in-Motion**

The concept of using bridges as weighing scales was first proposed by Moses [1]. B-WIM systems consist of strain transducers attached to the soffit of a bridge recording strain at set intervals defined by the scanning frequency of the system (typically 256Hz or 512Hz). Road surface mounted axle detectors, or extra strain transducers attached to the soffit in the case of NOR B-WIM [2], measure the vehicle’s velocity and axle spacing. An algorithm then uses these strain readings, axle spacing, and velocity to infer the static axle loads. A detailed description of the process of inferring the static axle weights follows in section 2.

**Dynamic Increment of Load**

The use of a parameter to provide for dynamic amplification of the effect of traffic load is common among design guides and codes. There are many factors, such as vehicle characteristics (axle spacing, suspension parameters, etc.), vehicle velocity, bridge natural frequencies, boundary conditions, and road profile that contribute to the magnitude of the dynamic load increment associated with a traffic loading event. The Eurocode [3] applies Dynamic Amplification Factor (DAF) to the traffic load model, and AASHTO [4] defines a Dynamic Load Allowance (DLA) that is applied to the static traffic load. These DAFs or DLAs are necessarily conservative to allow for the large variability in dynamic amplification.

A B-WIM system is particularly well-suited to the study and quantification of the dynamic increment of load effect, as it directly measures the total load effect (total = dynamic + static) and can infer the static load effect using the calculated axle weights.
Numerous studies have examined the relationship between dynamic increment and load effect [5-9]. However, the corresponding influence line is not triangular; it is related to something between the simply-supported and fixed structure [10]. The reason is moment restrain due to the effect of the supported geometry, joint rotation through the depth of girder. O’Brien et al. [11] developed the method of calculating the influence line for a bridge using the measurements obtained from the bridge. The measured influence line could be obtained by filtering “noise” frequency of pre-weighted calibration truck action. In addition, soft loading testing using B-WIM system-based traffic data to obtain safety index and rating factor of the existing bridges was used to analyze the optimization of the influence line, even without knowing the true boundary conditions [8].

Also, Kim et al. [12] used two artificial neural networks as a pattern recognition technique to estimate vehicle characteristics from the time signals are recorded for a simply supported concrete bridge and a cable-stayed bridge. Yamaguchi et al. [13] applied B-WIM to a curved bridge with skew. The axle loads were obtained by a modification of Moses’ algorithm using local strain influence lines. The algorithm presents the gross weight with errors <10%. Deng and Cai [14a, 14b] introduced a method that separates the total response of a bridge into an inertial component due to the momentum of the bridge and an interaction component due to the vehicles moving on the bridge (damping forces are ignored). The study concludes that lack of consideration of the inertial effects (usually disregarded in other MFI applications) introduced unacceptable errors to load estimations, while the algorithm proposed remained accurate independent of vehicle speed, road surface condition, and moderate levels of noise. However, Wang and Qu [15] proposed the concept of the dynamic influence line to
calculate axle loads, even when the effects of bridge dynamics are relatively large. The dynamic influence line corresponds to the measured response when a unit load crosses the bridge at a certain speed. In this, the dynamics of the bridge are included, so it has a free-vibration part after the vehicle has left the bridge. The dynamic influence line cannot be obtained from measurements. Therefore, a sufficiently accurate model of the modes of the structure is required for obtaining the influence line.

Any direct measurement of dynamic increment is sensitive to the accuracy of the B-WIM estimate of static axle weights. More accurate studies about the dynamic increment of load effect can be conducted by improving the accuracy of B-WIM systems. In this paper, we will discuss three algorithms in details: algorithm #1 is applied in current commercial B-WIM system based on the Moses method; algorithm #2 involves adjusting the moment based on field data to obtain vehicle weight; and algorithm #3 involve filtering B-WIM experimental moment from the dynamic effects calculated by the simulation model.

**B-WIM Algorithm**

*Algorithm #1: Application Based on the Moses Method*

The main advantage of B-WIM systems is that the vehicle is in contact with the apparatus (i.e., the bridge) for periods of the order of one second. B-WIM systems maximize this advantage by smoothing out the dynamic component, even if there is no active attempt to remove it.

The algorithm developed by Moses in the late 1970s remains the basis of modern B-WIM systems. The algorithm is based on the assumption that a moving load will
induce strains in a structure proportional to the sum of products of axle weights and corresponding influence line ordinate values. The influence line refers to the point of measurement, often taken as around mid-span, in which strains are greatest. Recording strains at regular intervals, defined by the scan number, gives a typical number of strain values of the order of hundreds. An error function is defined as the sum of the squares of the differences between theory and measurement [1]:

\[ \phi = \sum_{k=1}^{S} (M_k^m - M_k^{th})^2 \]  

(1)

Where \( S \) is the total number of scans, \( M_k^m \) = measured bending moment (proportional to strain) in scan \( k \) and \( M_k^{th} \) = theoretical bending moment in scan \( k \).

The theoretical response, used in Equation 1, \( M_k^{th} \) is calculated as the sum of products of the individual axle weights and the corresponding influence line ordinates for the location of each axle of the truck at the time of each scan. The influence line used for these calculations has a particularly significant bearing on the resulting axle weight calculations.

The theoretical influence line is ideally a “triangular” line for a simply-supported structure (Figure 1). The influence line is defined in Equation 2 and plotted in Figure 2.

\[ X_1(x) = \begin{cases} 
\frac{ax}{L} & \text{x} \leq L - a \\
\frac{(L-a)(L-x)}{L} & \text{x} > L - a 
\end{cases} \]  

(2)

Where \( X_1(x) \) indicates influence line for algorithm #1, \( x \) is the load location, \( L \) is the bridge span, \( a \) is the distance between sensor to beam end.

For theoretical simulations described herein (simulating a simply-supported structure), the influence line is applied to calculate the static response for the current
commercial system by the Moses algorithm, so this testing method is labeled “Algorithm #1” based on commercial equipment results.

**Algorithm #2: Application of Adjusted Moment Based on BWIM Test Data**

Figure 3 shows a typical plot of B-WIM moment at sensor location based on field testing. As demonstrated in the figure, there is always an end moment at joint C and D, which is disregarded in the calculation based on algorithm #1. Algorithm #2 involves adjusting the end moment at testing span. Moreover, the Moses method of algorithm #1 is based on the simply supported model, while the multi-span of the bridge on US-78 behaves like a frame. Therefore, the influence line at testing span is also calculated based on simulation model of frame. The steps are as follows:

**a. Obtain the bending moment at sensor location from simulation model.**

Figure 4 shows the fixed joint of bridge US-78 [16], the vertical rebar, bridge girders, and the support columns that work together to partly restrain the joint. This type of joint can be considered a semi-rigid rotational joint. The semi-rigid connection can be simulated in the analysis of beam-column connection. Therefore, a sophisticated method of frame analysis is adopted. The simulation model is shown in Figure 5. The bending moment at sensor location is shown in Equation 3 [17]

\[
M_{SM}(i, j) = \begin{cases} 
\frac{a}{L_3} (P_{i,j}(x_{i,j} - \sum_{x=1}^{3} L_x) + M_{DC}(i, j) - M_{CD}(i, j)) & a \leq \sum_{x=1}^{3} L_x - x_{i,j} \\
\frac{a}{L_3} (P_{i,j}(\sum_{x=1}^{3} L_x - x_{i,j})(L_3 - a) + M_{DC}(i, j) - M_{CD}(i, j)) & a > \sum_{x=1}^{3} L_x - x_{i,j}
\end{cases}
\]

(3)

Where SM indicates simulation model, \(M_{CD}(i, j)\) and \(M_{DC}(i, j)\) are the end moment of member where the sensor is located (Figure 4, member of CD). The details of derivative
for Equation 3 are not presented for the sake of brevity; they can be found in other source material [17].

b. Obtain the IL for entire bridge.

Substituting \( P_{i,j} = 1 \) and \( j = 1 \) into Equation 3, the influence line of the entire bridge, \( X(i,j) \), is obtained

\[
X(i,j) = \begin{cases} 
\frac{a}{L_3} ((x_{i,j} - \sum_{s=1}^{3} L_s) + M_{DC}(i,j) - M_{CD}(i,j)) & j = 1, \quad a \leq \sum_{s=1}^{3} L_s - x_{i,j} \\
\frac{a}{L_3} ((\sum_{s=1}^{3} L_s - x_{i,j})(L_3 - a) + M_{DC}(i,j) - M_{CD}(i,j)) & j = 1, \quad a > \sum_{s=1}^{3} L_s - x_{i,j}
\end{cases}
\]

(4)

Where \( x_{i,j} \) is the wheel location at the scan number \( i \) of WIM system for the wheel number \( j \), \( L_s \) is the length for span number \( s \).

c. Obtain the IL for testing span.

The influence line at the testing span can be obtained by limiting the range between joint C and D

\[
X_2(i,j) = \begin{cases} 
\frac{a}{L_3} ((x_{i,j} - \sum_{s=1}^{3} L_s) + M_{DC}(i,j) - M_{CD}(i,j)) & j = 1, \quad a \leq \sum_{s=1}^{3} L_s - x_{i,j} \& x_{i,j} > \sum_{s=1}^{3} L_s \\
\frac{a}{L_3} ((\sum_{s=1}^{3} L_s - x_{i,j})(L_3 - a) + M_{DC}(i,j) - M_{CD}(i,j)) & j = 1, \quad a > \sum_{s=1}^{3} L_s - x_{i,j} \& x_{i,j} > \sum_{s=1}^{3} L_s
\end{cases}
\]

(5)

Where \( X_2(i,j) \) indicates the influence line for algorithm #2.

d. Obtain the end moment of testing member from field testing.

B-WIM testing was conducted on the bridge on US-78, and the strain signals were collected and analyzed. Based on the relationship between the moment strain \( \varepsilon = \frac{\sigma}{E} \) and
stress $\sigma = \frac{Mr}{I}$, the WIM testing moment at the testing location for each girder, $M_{WIM}(k)$, and the total bending moment at the sensor location, $M_{WIM}$, are defined as

$$M_{WIM}(k) = \sum_{i=1}^{n} \sum_{j=1}^{m} \frac{e_k(i, j) (EI_k)}{r_k} = \sum_{i=1}^{n} \sum_{j=1}^{m} \frac{e_k(i, j) EI D_k}{r_k}$$  \hspace{1cm} (6)$$

$$M_{WIM} = \sum_{k=1}^{w} \sum_{i=1}^{n} \sum_{j=1}^{m} \frac{e_k(i, j) EI D_k}{r_k}$$  \hspace{1cm} (7)$$

Where, $e_k(i, j)$ is the moment strain for the wheel $i$ at location $j$ of girder $k$; the $r_k$ is the bottom arm based on the effectiveness of the girder cross section, $EI$ is the effective flexural stiffness, $D_k$ is the girder distribution factor, $w$ is the total girder number, $n$ is total scan number of WIM system, and $m$ is the total wheel load number.

If the wheel location $x$ is applied, the Equation 7 is derived as

$$M_{WIM}(x) = \sum_{k=1}^{w} \sum_{i=1}^{n} \sum_{j=1}^{m} \frac{e_k(i, j) EI D_k}{r_k}$$  \hspace{1cm} (8)$$

Where $x = \frac{V_i}{f}$, $V$ is vehicle speed, $f$ is the scan frequency of WIM system.

Therefore, the moment at location joint C, $x_C = \sum_{s=1}^{2} L_s$, is

$$M_{WIM}^C = \sum_{k=1}^{w} \sum_{i=1}^{n} \sum_{j=1}^{m} \frac{e_k(i, j) EI D_k}{r_k} \bigg|_{x = \sum_{s=1}^{2} L_s}$$  \hspace{1cm} (9)$$

The moment location joint D, $x_D = \sum_{s=1}^{3} L_s$, is

$$M_{WIM}^D = \sum_{k=1}^{w} \sum_{i=1}^{n} \sum_{j=1}^{m} \frac{e_k(i, j) EI D_k}{r_k} \bigg|_{x = \sum_{s=1}^{3} L_s}$$  \hspace{1cm} (10)$$
e. Adjusted WIM moment.

The adjusted bending moment for random location \( x \) at testing span can be obtained as follows:

\[
M_{WIM}^{Adjusted}(x) = M_{WIM}(x) + M_{WIM}^{C} - \frac{L_{A}x}{M_{WIM}^{C} - M_{WIM}^{D}}
\]  

(11)

f. Calculate vehicle weight.

Therefore, the adjusted moment \( M_{WIM}^{Adjusted}(x) \) can be defined as the sum of products of the individual axle weights and corresponding moment influence line.

\[
[P_{i,j}][X_{2}(i, j)] = [M_{WIM}^{Adjusted}(x)]
\]  

(12)

Equation 12 can be equal to derive wheel weight as function

\[
[P_{i,j}] = [X_{2}(i, j)]^{-1}[M_{WIM}^{Adjusted}(x)]
\]  

(13)

Where \( P_{i,j} \) is the wheel axle load at scan number \( i \) for wheel axle number \( j \).

Equation 13 is a set of equations. By applying the matrix Singular Value Decomposition (SVD) method [18], the vehicle weight can be obtained. This method, which calculates the wheel weight based on the adjusted WIM moment and the corresponding influence line, is labeled “Algorithm #2”.

Algorithm #3: Application of Filtered Moment Based on the Simulation Model

In algorithm #2, the bending moment at sensor location is adjusted based on the member end moment; however, this adjustment does not follow the complete relationship between the moving wheel load and the associated moment at sensor location. For example, WIM moments at sensor location can be affected by the dynamic effect
(namely, bridge vibration, dynamic load, and time delay) based on the simulation model of the entire bridge frame. Algorithm #3 is proposed based on filtering out all of the above factors to obtain the filtered moments. The steps are as follows:

a. Obtain the IL of entire bridge.

The influence line at the testing sensor for the entire frame is derived in Equation 4. The influence line for algorithm #3, \( X_3(i, j) \), is defined as:

\[
X_3(i, j) = X(i, j)
\]  

\[ (14) \]

b. Obtain dynamic moment at the sensor location from the simulation model.

As previously discussed, when a vehicle crosses a bridge with high velocity on the highway, the testing sensors are sensitive to many effects, such as: 1) static load (vehicle wheel load); 2) longitudinal vibration of bridge; 3) transverse vibration of bridge; 4) dynamic load from the interaction between vehicle suspension and road profile; 5) horizontal friction load, and 6) time delay. The details for each effect are discussed below.

b-1) Obtain static moment.

When the static wheel load is moving on the bridge, based on the simulation model of the entire frame, the static moment, \( M_{St} \), can be calculated as [17]:

\[
M_{St} = \ldots
\]
where $\theta_B(i, j)$ and $\theta_C(i, j)$ denote the rotation at joint $B$ and $C$ for load $p_{i,j}$, respectively, $\Delta_F(i, j)$ and $\Delta_D(i, j)$ indicate vertical displacement at joint $F$ and $D$ for load $p_{i,j}$, respectively.

**b-2) Obtain longitudinal vibration moment.**

The longitudinal vibration exists in the period when the vehicle activates the bridge, the vibration moment at the sensor location is defined as [17]:

$$M_{SM}^{LV} = \sum_{i=1}^{a} \sum_{j=1}^{\beta_0} \left( \sum_{i=1}^{\alpha_0} P_{i,j} \left( \frac{3}{L_3} \left( L_3 - x_{i,j} \right) \right) \right) - \sum_{i=1}^{a} \sum_{j=1}^{\beta_0} \left( \frac{2EI}{L_3} \left[ \theta_C(i, j)K^2_{CD} + \frac{3\Delta_F(i, j)}{L_3} - \frac{3\Delta_D(i, j)}{L_3} \right] \right) + \frac{P_{i,j}x_{i,j}^2}{(L_3)^2} \beta_D \right) \right) a \leq \sum_{i=1}^{a} L_3 - x_{i,j}

$$

$$M_{SM}^{LV} = \sum_{i=1}^{a} \sum_{j=1}^{\beta_0} \left( \sum_{i=1}^{\alpha_0} P_{i,j} \left( \frac{3}{L_3} \left( L_3 - x_{i,j} \right) \right) \right) - \sum_{i=1}^{a} \sum_{j=1}^{\beta_0} \left( \frac{2EI}{L_3} \left[ \theta_C(i, j)K^2_{CD} + \frac{3\Delta_F(i, j)}{L_3} - \frac{3\Delta_D(i, j)}{L_3} \right] \right) + \frac{P_{i,j}x_{i,j}^2}{(L_3)^2} \beta_D \right) \right) a > \sum_{i=1}^{a} L_3 - x_{i,j}

$$

(15)

Where $y(i, j)^{Free} = CD_{i,j} e^{-\frac{\xi_0 t}{f}} \cos(\omega_D \frac{t}{f} - \alpha_{i,j})$
\[ A_{i,j}^{\text{Free}} = \frac{12(El)_{i} a}{(L_{3} a^{2} + aL_{3}^{2} - a^{3})}, \]

\[ A_{i,j}^{\text{Forced}} = \begin{cases} \frac{6(El)_{i}}{(2L_{3}x_{i,j} - x_{i,j}^{2} - (L_{3} - a)^{2})} & a \geq \frac{3}{s=1} \sum L_{s} - x_{i,j} \\ \frac{6(El)_{i}}{(L_{3}^{2} - a^{2} - x_{i,j}^{2})L_{3}} & a < \frac{3}{s=1} \sum L_{s} - x_{i,j} \end{cases} \]

To filter the moment of the longitudinal vibration, setting \( P_{i,j} = 1 \) and substituting into equation 16, the influence line for longitudinal vibration, \( \Delta X_{PM}^{DY}(LV) \), is defined as:

\[ \Delta X_{PM}^{DY}(LV) = \sum_{i=1}^{n} \sum_{j=1}^{m} \left( M_{PM}^{LV}(i, j) \right) \bigg|_{P_{i,j}=1} \]

Therefore, the moment from longitudinal vibration effect can be defined as:

\[ M_{PM}^{DY}(LV) = P_{i,j} \Delta X_{PM}^{DY}(LV) \]

(17)

(18)

b-3) Obtain transverse vibration moment.

The bridge on US-78 is antisymmetric, the moment from transverse vibration is defined as: (Zhao, 2012)

\[ M_{PM}^{TV}(i, j) = \sum_{r=1}^{4} (A_{i,j}^{TV}, y(i, j)_{r})^{TV} = 0 \]

(19)

For this antisymmetric bridge, the influence line of transverse vibration,

\[ \Delta X_{PM}^{DY}(TV) \]

and the transverse vibration moment, \( M_{SM}^{DY}(TV) \), are:

\[ \Delta X_{PM}^{DY}(TV) = 0 \]

(20)

\[ M_{SM}^{DY}(TV) = 0 \]

(21)

b-4) Obtain the moment by the interaction of road profile and vehicle suspension system.
When a vehicle crosses the bridge, the vehicle’s suspension system will interact with the road profile, the tire dynamic load can be defined as [17]:

\[
F_g^D(i, j) = -W(i, j) + \sum_{r=1}^{\infty} \left[ U_{x, r}(i, j) R_{y, r}(i, j) K_v (\sin(\phi_r t - \phi_r) + C_v \omega_r \cos(\phi_r t - \phi_r)) \right]
\]  

(22)

As a result, the increment of the dynamic weight beyond the wheel load is defined as:

\[
\Delta W(i, j) = F_g^D(i, j) + W(i, j) + \sum_{r=1}^{\infty} \left[ U_{x, r}(i, j) R_{y, r}(i, j) K_v (\sin(\phi_r t - \phi_r) + C_v \omega_r \cos(\phi_r t - \phi_r)) \right]
\]  

(23)

Substituting Equation 23 into Equation 15 and setting \( P_{x, j} = 1 \), we obtained the influence line for the additional dynamic load at the sensor location. It is defined as \( \Delta X_{PM}^{DY}(\Delta W) \). The moment from the additional dynamic load can be presented as:

\[
M_{PM}^{DY}(\Delta W) = P_{x, j} \Delta X_{PM}^{DY}(\Delta W)
\]  

(24)

**b-5) Obtain the moment from horizontal friction load.**

Horizontal friction load exists when a vehicle’s tire makes contact with the road surface in a horizontal direction. The amount of friction load is based on the tire’s contacted force and the friction coefficient. The friction load is defined as [17]:

\[
F_f^D(i, j) = \mu_r F_g^D(i, j)
\]  

(25)

Substituting equation 25 into equation 15, using \( \mu_r F_g^D(i, j) \) to replace \( \mu_r P_{x, j} \), and setting \( P_{x, j} = 1 \), we obtain the moment influence line of the additional friction load at the sensor location, defined as: \( \Delta X_{PM}^{DY}(Friction) \). The moment from the friction effect at the sensor location can be expressed as:

\[
M_{PM}^{DY}(Friction) = P_{x, j} \Delta X_{PM}^{DY}(Friction)
\]  

(26)
b-6) Obtain the moment effect from time delay.

Time delay exists for each moving wheel, so the wheel location is defined as [17]:

\[
X_{i,j} = V_i (t_i - \frac{L_i}{V_i})
\]

(27)

If setting \( t_i = 0 \) in Equation 27, the adjusted location is derived as:

\[
\Delta X_{i,j} = -V_i \frac{L_i}{V_i}
\]

(28)

Substituting Equation 28 into Equation 15 and setting \( P_{i,j} = 1 \), the moment influence line for the time delay at the sensor location is defined as \( \Delta X_{\text{PM}}^{\text{DY}} \text{ (Time)} \). Therefore, the affected moment of the time delay can be expressed as:

\[
M_{\text{PM}}^{\text{DY}} (\Delta T) = P_{i,j} \Delta X_{\text{PM}}^{\text{DY}} (\Delta T)
\]

(29)

b-7) Obtain the total moment.

Finally, summarizing all of the effects discussed above, the total moment from the simulation model is defined as:

\[
M_{\text{Total}}^{\text{SM}} = M_{\text{Static}}^{\text{SM}} + M_{\text{PM}}^{\text{DY}} (LV) + M_{\text{PM}}^{\text{DY}} (TV) + M_{\text{PM}}^{\text{DY}} (\Delta W) + M_{\text{PM}}^{\text{DY}} (\text{Friction}) + M_{\text{PM}}^{\text{DY}} (\text{Time})
\]

(30)

Assuming the WIM testing moment is equal to the moment from the model, and substituting Equations 15, 18, 21, 24, 26 and 29 into 30, we obtain:

\[
M_{\text{WIM}} = M_{\text{Total}}^{\text{SM}}
= M_{\text{Static}}^{\text{SM}} + P_{i,j} (\Delta X_{\text{PM}}^{\text{DY}} (LV) + \Delta X_{\text{PM}}^{\text{DY}} (TV) + \Delta X_{\text{PM}}^{\text{DY}} (\Delta W) + \Delta X_{\text{PM}}^{\text{DY}} (\text{Friction}) + \Delta X_{\text{PM}}^{\text{DY}} (\text{Time}))
\]

(31)

Defining the filtered static moment \( M_{\text{WIM}}^{\text{Filtered}} \) equal to \( M_{\text{Static}}^{\text{SM}} \), then:

\[
M_{\text{SM}}^{\text{Filtered}} = M_{\text{WIM}} - P_{i,j} (\Delta X_{\text{PM}}^{\text{DY}} (LV) + \Delta X_{\text{PM}}^{\text{DY}} (TV) + \Delta X_{\text{PM}}^{\text{DY}} (\Delta W) + \Delta X_{\text{PM}}^{\text{DY}} (\text{Friction}) + \Delta X_{\text{PM}}^{\text{DY}} (\text{Time}))
\]
From Equation 32, the WIM moment is filtered from longitudinal vibration, transverse vibration, dynamic load, friction load, and time delay.

b-8) Obtain the vehicle weight.

The filtered static moment $M_{\text{WIM}}^{\text{Filtered}}$ can be defined as the sum of products of the individual axle weights and corresponding influence line.

$$[P_{i,j}][X_{ji}(i, j)] = [M_{\text{WIM}}^{\text{Filtered}}]$$

(33)

Where $p_{i,j}$ is the $i^{th}$ wheel axle load, $i$ is the scan number of WIM testing, $j$ is the wheel axle number.

Substituting Equation 32 into Equation 33, we obtain:

$$[P_{i,j}][\hat{X}(i, j)] = [M_{\text{WIM}}]$$

(34)

Where $\hat{X}(i, j) = X_{ji}(i, j) + \Delta X_{PM}^{DY}(LV) + \Delta X_{PM}^{DY}(LV) + \Delta X_{PM}^{DY}(\Delta W) + \Delta X_{PM}^{DY}(\text{Friction}) + \Delta X_{PM}^{DY}(\text{Time})$

So the vehicle weight is:

$$[P_{i,j}] = [\hat{X}(i, j)]^{-1}[M_{\text{WIM}}]$$

(35)

By applying the matrix Singular Value Decomposition (SVD) method [18], the vehicle weight can be obtained. This method is labeled as “Algorithm #3”. Table 1 summarizes the three alternative B-WIM algorithms.

Example Application

Initial WIM Calibration
The bridge on the US-78 highway is located in Graysville, Alabama, USA. It has three single-spans with each span of 12.8 m (42 ft) [16]. The Alabama Department of Transportation (ALDOT) conducted WIM testing in 2008 (Figures 6 and 7) [19]. The initial calibration test was calibrated under the test condition (R1-I) based on the European specifications for WIM [20]. The representative vehicles on highway US-78 were two semi-trailers with capacity of 355.84 KN (80,000 lbs, five-axle trailer trucks). These vehicles came from ALDOT as pre-weighed trucks. ALDOT tested the identified 10 runs for both vehicles at each lane. Table 2 provides details of the calibrated vehicles.

**Influence Line for Algorithm #1, #2 and #3**

Based on the pre-weighed truck calibration and simulation model for effects of moment strain (companion paper Part I), the influence lines are plotted in Figure 8. The influence line of algorithm #1 is ideally triangular at testing span only; the influence line of algorithm #2 is located at testing span only; and the influence line of algorithm #3 is for whole frame structure.

**Moment for Algorithm #1, #2 and #3**

Figure 9 shows the bending moment at sensor location for algorithms #1, #2 and #3. The WIM testing moment applies to algorithm #1. For algorithm #2, the adjusted WIM moments ordinated in testing span are returned to around zero at the starting and ending location of testing span. For algorithm #3, the figure presents the filtered moments for the entire bridge from WIM testing based on the proposed simulation model.
Results

The COST 323 draft standard defines accuracy classes based on the width of the confidence intervals within which error falls for GVW, single axle and group of axles. Acceptable percent error on GVW at highway speeds is ±5% [16]. Details of the class results are provided in Table 3.

1) Figure 10 shows the accuracies of predictions for 20 individual tests on Highway US-78 under normal traffic conditions.
   a. For each axle weight, algorithm #1 has as high as 60% error, algorithm #2 has as high as 45% error, but algorithm #3 only has within 10% error.
   b. For the GVW, algorithm #1 has about 20% error, algorithm #2 has about 10% error, and algorithm only has within 5% error.

2) Figure 11 gives a classification graphical representation of variability of results.
   a. For single axle weight, the class is E(55) and E(60) for algorithms #1 and #2, respectively. For algorithm #3, the class is A(5).
   b. For group axle weight, the class is E(60) and D+(20) for algorithms #1 and #2, respectively. For algorithm #3, the class is B(10).
   c. For GVW, the class is D+(20) and B(10) for algorithms #1 and #2, respectively. For algorithm #3, the class is A(5).
   d. For the entire bridge, the class is E(60), E(60) and B(10) for algorithms #1, 2, and 3, respectively.
   e. Comparison the results with above, for single axle weight, algorithms #1 and #2 have significant error. For group axle and GVW, algorithm #2 has greater accuracy than algorithm #1. Algorithm #3 has significant improvements in
accuracy for individual axle weight and GVW. Where it can be seen that accuracy classes of B (10) and A (5) are returned for GVW of Algorithm #2 and #3 from D+ (20) of Algorithm #1, and classes A (5) can be obtained for each axle weight using Algorithm #3.

3) The alternative algorithm #2 has excellent improvement in accuracy for group axle weight and GVW; algorithm #3 has significant improvement in accuracy for single axle weight, group axle weight, and GVW.

Conclusions

Based on the field WIM data, two alternative algorithms were developed for vehicle weight identification. Algorithm #2 has improved accuracy for the group of axles and GVW, and algorithm #3 has significantly accurate predictions for all single axle weight, group of axles, and GVW. For group axle weight, the acceptable class of D+(20) and B(10) are obtained for algorithms #2 and #3, respectively. For GVW, the acceptable class of B(10) and A(5) are obtained for algorithms #2 and #3, respectively. Additionally, algorithm #3 obtains acceptable class of B(10) for the entire WIM testing. Algorithms #2 and #3 both demonstrate significant improvements in the accuracy for vehicle weight identification.

Acknowledgements

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References


motion by two-span continuous bridge with skew and heavy-truck flow in Fukuoka area, 2009, Japan.


Table 1. Description of the algorithms

<table>
<thead>
<tr>
<th>Algorithm</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>#1</td>
<td>Moses’s algorithm: Applied the measured total responses with ideal triangular influence line.</td>
</tr>
<tr>
<td>#2</td>
<td>Adjusted WIM moment method: Measured total response, adjust the measured moment based on end moment of member, applied it to influence line from proposed simulation model</td>
</tr>
<tr>
<td>#3</td>
<td>Filtered Measured Moment Method: Measured total response, filtered the measured moment based on proposed simulation model, and applied it to influence line of proposed simulation model</td>
</tr>
</tbody>
</table>

Table 2. Initial vehicle calibration information of bridge on highway US-78

<table>
<thead>
<tr>
<th>Vehicle Number</th>
<th>GtW 1st axle</th>
<th>2nd axle</th>
<th>3rd axle</th>
<th>4th axle</th>
<th>5th axle</th>
<th>W1-W2</th>
<th>W2-W3</th>
<th>W3-W4</th>
<th>W4-W5</th>
<th>Traffic Speed Range (km/h)</th>
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<tr>
<td>1</td>
<td>351.4</td>
<td>49.15</td>
<td>69.61</td>
<td>71.61</td>
<td>80.95</td>
<td>80.06</td>
<td>4.32</td>
<td>1.35</td>
<td>11.2</td>
<td>1.3</td>
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<td>2</td>
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<td>44.70</td>
<td>71.17</td>
<td>70.28</td>
<td>81.40</td>
<td>80.28</td>
<td>4.34</td>
<td>1.35</td>
<td>11.1</td>
<td>1.29</td>
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</table>

Table 3. Algorithm Errors and Classification

<table>
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<tr>
<th></th>
<th>Single Axle</th>
<th>Group Axle</th>
<th>GtW</th>
<th>Class</th>
</tr>
</thead>
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<tr>
<td>µ-Alg.#1</td>
<td>2.3</td>
<td>-10.5</td>
<td>9.4</td>
<td>-7.9</td>
</tr>
<tr>
<td>σ-Alg.#1</td>
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<td>7.3</td>
<td>8.9</td>
<td>6.3</td>
</tr>
<tr>
<td>µ-Alg.#2</td>
<td>15.0</td>
<td>-25.0</td>
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<td>-18.6</td>
</tr>
<tr>
<td>σ-Alg.#2</td>
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<td>11.6</td>
<td>13.5</td>
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</tr>
<tr>
<td>µ-Alg.#3</td>
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<td>-1.7</td>
<td>0.0</td>
<td>-0.4</td>
</tr>
<tr>
<td>σ-Alg.#3</td>
<td>2.9</td>
<td>2.4</td>
<td>2.8</td>
<td>2.9</td>
</tr>
</tbody>
</table>
Figure 1. Simplified support model

Figure 2. Moment influence line for algorithm #1
Figure 3. Adjusted WIM moments for multi-span bridge

(a) Initial testing WIM moment, (b) Adjusted WIM moment
Figure 4. Fixed boundary conditions of bridge US-78

Figure 5. Proposed simulation model for highway US-78
Figure 6. Bridge WIM testing of highway US-78

(b) Testing Bridge of Highway US-78, (b) Field WIM Testing
Figure 8. Influence line for algorithms #1, #2, and #3
Figure 9. Moment comparisons for algorithms #1, #2, and #3.
Figure 10. Error scatter of algorithm results for single axle weight
Figure 11. Classification scatter of algorithm results
FIELD CALIBRATED SIMULATION MODEL TO PERFORM BRIDGE SAFETY ANALYSES AGAINST EXTREME EVENTS

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Format adapted for dissertation
Abstract

A bridge may be functionally obsolete or have a deficient structure because of older design codes or features. Such bridges are not unsafe for normal vehicle traffic, but they can be vulnerable to unusual traffic conditions, such as heavy trucks braking in emergency situations or during a disaster [1]. Continued ignorance of deficiencies or lack of sufficient maintenance can cause a bridge to be subjected to hazards and damage, disrupting its reliable and efficient function. For example, in 2008, the eight-lane Interstate 35W bridge, designed in 1961, suddenly collapsed into the Mississippi River, killing 13 people. Since a traditional visual inspection cannot effectively explore potential problems, we propose a simulation approach based on bridge weigh-in-motion (B-WIM) data to predict bridge safety and integrity if heavy trucks experience emergency braking on the bridge. A bridge on US-78 was used in a case study. The results provided an improved understanding of the deficiencies of this bridge and led to suggestions for appropriate maintenance.

Research Highlights

• A model for an emergency situation was simulated by use of B-WIM data.

• Potential damage to the bridge was identified using the simulation model.

• A retrofit was recommended to protect the bridge in an emergency braking situation.

Keywords: Bridge, Moment Strain, Weigh-in-motion, WIM, Bridge Safety, Truck Brake, Friction Load
Introduction

Much of the bridge stock in North America and Europe, constructed in the 1950’s and 1960’s, has significantly deteriorated. Substandard load ratings and posting of bridges due to heavy truck weights in combination with deterioration problems have led to more than plenty of U.S. bridges being classified as structurally deficient or functionally obsolete. In their 2009 Report Card for America’s Infrastructure, the American Society of Civil Engineers (ASCE) assigned the U.S. bridge infrastructure a grade of “C” [2]. Moreover, the Safe and Efficient Transportation Act of 2010, which increased allowable truck weights from 355.8 kN (80 kip) to 431.5 kN (97 kips), allows states to authorize heavy trucks to operate on the Interstate system. Thus, already dangerous situations are expected to become worse, and bridges will need more maintenance, rehabilitation, or replacement.

In normal traffic situations, bridges generally function effectively. If a bridge faces an emergency or disaster situation, however, it may be in danger of collapsing. Herein, the situations of “emergency” and “disaster” are the particular conditions that are outside of control by current rules, codes, regulations, requirements, and experiences in the past. This situation can be beyond the code limit of the current design. It is essential to recognize two issues: (i) hazard characterization and (ii) design of protective systems. This allows us to identify gaps in knowledge, predict hazard issues, and maintain bridge stability in a situation outside normal traffic situations.

In the last two decades, there has been extensive analysis of the performance of highway bridges. Worldwide, many researchers have studied bridges analytically and
experimentally and have documented the performance of bridges under various
conditions.

The present report proposes a method to predict potential safety issues. This method
involves use of B-WIM experimental data to characterize an extreme event involving
emergency braking of a five-axle semi-trailer truck on a bridge, and a retrofit is
recommended for the bridge. Section 3 describes a simulation model for the static
moment from moving vehicles; section 4 describes the simulation of dynamic moments
resulting from the vehicle suspension, the road profile, and emergency braking; and
section 5 describes validation of the simulation model based on B-WIM experimental
data. These data show that the simulation model is valid, based on 20 trials of B-WIM
testing data. Section 6 provides results of the analysis, and section 7 gives a
recommendation for an appropriate retrofit.

US-78 Bridge Description

The highway bridge on US-78 at Graysville, Alabama, USA, has three single-spans
of 12.8 m (42 ft) each and is supported by two square columns of 945 mm x 945 mm (3
feet x 3 feet) at each bent [3]. The B-WIM testing was conducted by the Alabama
Department of Transportation (ALDOT) in 2008 [4]. The sensors were mounted
underneath at the mid-spans of each girder. The bridge information and field testing are
shown in Figures 1 and 2. The initial calibration test was performed under test condition
(R1-I) according to the European specifications for B-WIM [5]. The two representative
vehicles for testing were semi-trailers with a loading capacity of 355.8 KN (80000 lbs,
emergency loaded, five-axle trailer truck) as pre-weighed trucks from the ALDOT. The
effective signals of 10 runs for each lane were evaluated for both vehicles. Table 1 provides detailed information for these vehicles.

Simulation Model for a Moving Load

In practice, the simplified structural model of bridge US-78 did not reflect the behavior of the bridge. In order to identify the true moment strain, the simulation model is presented in Figure 3. For this model, the semi-rigid rotational joint, boundary condition, and flexural stiffness are discussed as follows.

Semi-rigid Rotational Joint

In reality, the influence line of a simply supported bridge is not triangular and may be between the simply supported and fixed cases [6]. The reason is that the supported joints are not in “ideal” single support, with the capacity to transfer the forces and moments. For the connection of a simply-supported bridge, traditional designs neglect the real behavior of connections. Thus, the idealization of a pinned connection was used in the design. However, the predicted response of a bridge frame may not be realistic. In practice, most connections have the rotational capacity contributing to structure displacements. As shown in Figure 4, for the fixed joint of the bridge US-78, the vertical rebar, bridge girders, and the support columns work together to restrain the joint. However, this connection cannot emergency transfer all forces. Therefore, these types of joints can be represented as semi-rigid rotational joints. The semi-rigid connection can be simulated in the analysis of beam-column connections. This flexible connection behavior affects the internal force distribution of the frame structure, and a more reliable prediction
of frame behavior can be obtained by use of a semi-rigid rotational spring.

Therefore, the multi-span of bridge US-78 behaves similarly to a frame. A sophisticated method of frame analysis was adopted in this research. A linear representation of the spring was developed to analyze the bridge frame with a semi-rigid connection for each beam element, as shown in Figure 5. The effects of the connection flexibility are modeled as rotational spring constants \( S_j \) and \( S_k \), where \( j \) and \( k \) are the ends of a frame element, and \( \Phi_j \) and \( \Phi_k \) are rotations incurred by rotational springs.

According to first-order analysis, the stiffness matrix of a member with semi-restraint at the ends can be represented by the correction stiffness matrix based on the rigid connections [7-9].

\[
k = \begin{bmatrix}
\frac{EA}{L} & 0 & 0 & \frac{EA}{L} & 0 & 0 \\
\frac{12EI}{L^3} (\alpha_1 + \alpha_2 + \alpha_1 \alpha_2) & \frac{6EI}{L^3} (2\alpha_1 + \alpha_1 \alpha_2) & 0 & \frac{12EI}{L^3} (\alpha_1 + \alpha_2 + \alpha_1 \alpha_2) & \frac{6EI}{L^3} (2\alpha_2 + \alpha_1 \alpha_2) & 0 \\
\frac{12EI}{L^3} (4\alpha_2 - \alpha_1 \alpha_2) & \frac{6EI}{L^3} (2\alpha_2 - 4\alpha_1 \alpha_2) & 0 & \frac{12EI}{L^3} (4\alpha_2 - \alpha_1 \alpha_2) & \frac{6EI}{L^3} (2\alpha_2 - 4\alpha_1 \alpha_2) & 0 \\
\frac{4EI}{L^3} \alpha_1 & \frac{3EI}{L^3} \alpha_1 \alpha_2 & 0 & \frac{4EI}{L^3} \alpha_1 & \frac{3EI}{L^3} \alpha_1 \alpha_2 & 0 \\
\frac{6EI}{L^3} (2\alpha_2 - 4\alpha_1 \alpha_2) & \frac{6EI}{L^3} (2\alpha_2 - 4\alpha_1 \alpha_2) & 0 & \frac{6EI}{L^3} (2\alpha_2 - 4\alpha_1 \alpha_2) & \frac{6EI}{L^3} (2\alpha_2 - 4\alpha_1 \alpha_2) & 0 \\
\frac{4EI}{L^3} \alpha_1 & \frac{3EI}{L^3} \alpha_1 \alpha_2 & 0 & \frac{4EI}{L^3} \alpha_1 & \frac{3EI}{L^3} \alpha_1 \alpha_2 & 0 \\
\end{bmatrix}
\]

\( k^* \)

If the axle and shear deformations are neglected, the stiffness matrix can be modified as Equation 2.

\[
k^* = \begin{bmatrix}
\frac{4EI}{L} \alpha_1 & \frac{2EI}{L} \alpha_1 \alpha_2 \\
\frac{2EI}{L} \alpha_1 \alpha_2 & \frac{4EI}{L} \alpha_1 \\
\frac{4EI}{L} \alpha_1 \alpha_2 & \frac{2EI}{L} \alpha_1 \alpha_2 \\
\end{bmatrix}
\]

where \( EI \) is the flexural stiffness, and \( L \) is the length. In equation 3, the parameters \( \alpha_1 \) and \( \alpha_2 \) are the fixity factors at each end of the member, and both factors are related with rotational spring stiffness, \( S_j \) and \( S_k \).
The fixity factor \( \alpha \) determines the stiffness of the connection relative to the attached beam and to the rotational capacity of moments. Following the conventional matrix procedures of displacement for a rigid-jointed frame, the adjusted end-moments, \( M_j \) and \( M_k \), for the semi-rigid connection member are defined as follows:

\[
M_j = \frac{3\alpha_1(2-\alpha_2)}{4-\alpha_1\alpha_2} M_j'; 
M_k = \frac{3\alpha_2(2-\alpha_1)}{4-\alpha_1\alpha_2} M_k';
\]

where \( M_j' \) and \( M_k' \) are the end moment for rigid connection.

The adjusted end moment of the member is defined as

\[
\begin{bmatrix} M_j' \\ M_k' \end{bmatrix} = k \begin{bmatrix} \theta_j \\ \theta_k \end{bmatrix} + \begin{bmatrix} M_j \\ M_k \end{bmatrix}
\]

where \( \theta_j \) and \( \theta_k \) is the rotation at end elements \( j \) and \( k \).

In linear elastic analyses, if the connections are assumed to have a linear force-displacement relationship, and the axial and shear deformation of members are ignored, the solution of member moment can be achieved by the set of linear equations

\[
K \mu = M
\]

here \( K \) is the bridge stiffness matrix, \( \mu \) is the deformation of bridge, and \( M \) is the external moment.

**Boundary Condition**

During vehicle moving, the bridge horizontal movements exists with the sensitive parameters influencing the testing strains. Figure 6 shows the boundary conditions of the bridge on US-78 at an expansion joint. The connection was designed as the smooth steel
plate lay on the smooth steel pad without vertical ties or horizontal restraint at the expansion joint, so that the bridge has the potential to move in the horizontal direction. A horizontal spring is provided to simulate the horizontal movement at joint “A” (Figure 4), and the horizontal spring constant, $K_H$, is defined as

$$K_H = 0.5 \mu_b W_b,$$  (7)

where $\mu_b$ is the friction coefficient between the smooth steel plate and the smooth steel pad [10], and $W_b$ is the bridge weight of the related span.

**Formula of Simulation Model**

According to the assumption of the boundary condition and a semi-rigid connection, the structural system of the bridge on US-78 is modeled as a frame structure (Figure 3), subjected to a random wheel $P_{i,j}$ at location $x_{i,j}$, where $i$ is the scan number of the B-WIM system, and $j$ is the wheel number. By applying the slope-deflection method, the end moments of each member are defined as

$$M_{AB}(i, j) = \frac{2(EI)_1}{L_1} \theta_B(i, j) K_{AB}^1 + M^L_1 \beta_A^1$$  (8)

$$M_{BA}(i, j) = \frac{2(EI)_1}{L_1} 2\theta_B(i, j) K_B^1 + M^R_1 \beta_B^1$$  (9)

$$M_{BE}(i, j) = \frac{2(EI)_4}{L_4} (2\theta_B(i, j) - \frac{3\Delta_H(i, j)}{L_4})$$  (10)

$$M_{EB}(i, j) = \frac{2(EI)_4}{L_4} (\theta_B(i, j) - \frac{3\Delta_H(i, j)}{L_4})$$  (11)

$$M_{BC}(i, j) = \frac{2(EI)_2}{L_2} (2\theta_B(i, j) K_B^2 + \theta_C(i, j) K_{BC}^2) + M^L_2 \beta_B^2$$  (12)
\[ M_{CB}(i, j) = \frac{2(EI)_{ji}}{L_2} \left( \theta_{B}(i, j)K_{CB}^2 + 2\theta_{C}(i, j)K_{C}^2 \right) + M_{2}^R \beta_{C}^2 \] (13)

\[ M_{CF}(i, j) = \frac{2(EI)_{ji}}{L_5} \left( 2\theta_{C}(i, j) - \frac{3\Delta_{H}(i, j)}{L_5} \right) \] (14)

\[ M_{FC}(i, j) = \frac{2(EI)_{ji}}{L_5} \left( \theta_{C}(i, j) - \frac{3\Delta_{H}(i, j)}{L_5} \right) \] (15)

\[ M_{CD}(i, j) = \frac{2(EI)_{ji}}{L_3} 2\theta_{C}(i, j)K_{C}^3 + M_{3}^L \beta_{C}^3 \] (16)

\[ M_{DC}(i, j) = \frac{2(EI)_{ji}}{L_3} \theta_{C}(i, j)K_{CD}^3 + M_{3}^R \beta_{D}^3 \] (17)

where A, B, C, D, E, and F indicate joint numbers; \( \theta_{B}(i, j) \) and \( \theta_{C}(i, j) \) denote the rotation at joints B and C for load \( P_{i,j} \), respectively; \( \Delta_{H}(i, j) \) indicates the horizontal movement for load \( P_{i,j} \).

The fixity factor, \( \alpha \), and the rotational spring stiffness, \( S \), for each member are defined as follows:

For member 1: \( \alpha_{A}^1 = \frac{1}{1 + 3(EI)_{A}/S_{AB}L_{A}} \); \( \alpha_{B}^1 = \frac{1}{1 + 3(EI)_{1}/S_{BA}L_{A}} \)

For member 2: \( \alpha_{B}^2 = \frac{1}{1 + 3(EI)_{2}/S_{BC}L_{2}} \); \( \alpha_{C}^2 = \frac{1}{1 + 3(EI)_{2}/S_{CB}L_{2}} \)

For member 3: \( \alpha_{C}^3 = \frac{1}{1 + 3(EI)_{3}/S_{CD}L_{3}} \); \( \alpha_{D}^3 = \frac{1}{1 + 3(EI)_{3}/S_{DC}L_{3}} \)

The modified factors of stiffness matrix, \( K \), and adjusted moment factor, \( \beta \), for each member are defined as following

For member 1: \( K_{A}^1 = \frac{3\alpha_{A}^1}{4 - \alpha_{A}^1 \alpha_{B}^1} \); \( K_{AB}^1 = K_{BA}^1 = \frac{3\alpha_{A}^1 \alpha_{B}^1}{4 - \alpha_{A}^1 \alpha_{B}^1} \); \( K_{A}^1 = \frac{3\alpha_{B}^1}{4 - \alpha_{A}^1 \alpha_{B}^1} \);
\[ \beta_B^1 = \frac{3\alpha_B^1 (2 - \alpha_A^1)}{4 - \alpha_A^1 \alpha_B^1} \]

For member 2: \[ K_B^2 = \frac{3\alpha_B^2}{4 - \alpha_B^2 \alpha_C^2}; \quad K_{BA}^2 = \frac{3\alpha_C^2}{4 - \alpha_B^2 \alpha_C^2}; \quad K_C^2 = \frac{3\alpha_C^2}{4 - \alpha_B^2 \alpha_C^2}; \]
\[ \beta_B^2 = \frac{3\alpha_B^2 (2 - \alpha_C^2)}{4 - \alpha_B^2 \alpha_C^2}; \quad \beta_C^2 = \frac{3\alpha_C^2 (2 - \alpha_B^2)}{4 - \alpha_B^2 \alpha_C^2} \]

For member 3: \[ K_C^3 = \frac{3\alpha_C^3}{4 - \alpha_C^3 \alpha_D^3}; \quad K_{CD}^3 = \frac{3\alpha_C^3 \alpha_D^3}{4 - \alpha_C^3 \alpha_D^3}; \quad K_C^3 = \frac{3\alpha_D^3}{4 - \alpha_C^3 \alpha_D^3}; \]
\[ \beta_C^3 = \frac{3\alpha_C^3 (2 - \alpha_D^3)}{4 - \alpha_C^3 \alpha_D^3} \]

The fixed end moment of member for span \( s \) (\( s \) denotes the span No. 1, 2 and 3) is defined as

\[
M_s^L = \begin{cases} 
\frac{P_{i,j} x_{i,j} (L_s - x_{i,j})}{(L_s)^2} & \text{When } P_{i,j} \text{ locates at span } s \\
0 & \text{others span}
\end{cases}
\]

\[
M_s^R = \begin{cases} 
\frac{P_{i,j} x_{i,j} (L_s - x_{i,j})}{(L_s)^2} & \text{When } P_{i,j} \text{ locates at span } s \\
0 & \text{others span}
\end{cases}
\]

From Figure 3, the equations of each joint and force balance are given

\[ M_{BA} + M_{BE} + M_{BC} = 0 \quad (18) \]
\[ M_{CB} + M_{CF} + M_{CD} = 0 \quad (19) \]
\[ \frac{M_{BE} + M_{EB} + M_{CF} + M_{FC}}{L_4} = K_{H} \Delta_H + \mu_i P_{i,j} \quad (20) \]

Substituting Equations 8 – 17 into Equations 18 – 20, we obtain

\[
\left( \frac{4(EL)}{L_1} K_A^1 + \frac{4(EL)}{L_2} K_B^2 + \frac{4(EL)}{L_4} \right) \theta_A^1 + \frac{2(EL)}{L_2} K_{bc}^2 \theta_c^1 - \frac{6(EL)}{L_4} \Delta_H = -M^R \beta_B^1 - M^R \beta_B^2 \quad (21)
\]
The assembly stiffness matrix $K$ for frame structure is

$$
K = \begin{bmatrix}
\frac{2(EI)}{L_2} K_{bc}^2 \theta_B + \frac{4(EL)}{L_2} K_{c}^2 + \frac{4(EL)}{L_3} K_{c}^3 + \frac{4(EL)}{L_5} \theta_C - \frac{6(EL)}{L_3^2} \Delta_H = -M_B^R \beta_B^2 - M_B^L \beta_B^L \\
\frac{6(EL)}{L_4} \theta_B + \frac{6(EL)}{L_5} \theta_C - (\frac{12(EL)}{L_4^2} + \frac{12(EL)}{L_5^2} + K_{bc}^H) \Delta_H = \mu_{j,i}
\end{bmatrix}
$$

Where $EI$ is the effective flexure stiffness, $L$ is the length and the joint external moment $M(i,j)$ for frame structure is

$$
M(i,j) = \begin{bmatrix}
-M_B^R \beta_B^1 - M_B^L \beta_B^2 \\
-M_C^R \beta_C^2 - M_C^L \beta_C^3 \\
\mu_{j,i}
\end{bmatrix}
$$

Thus, following linear Equation 6, the $M-u$ relationship is defined as

$$
[K] \begin{bmatrix}
\theta_B (i,j) \\
\theta_C (i,j) \\
\Delta_H (i,j)
\end{bmatrix} = [M(i,j)]
$$

Solving Equation 26, each parameter can be obtained

$$
\begin{bmatrix}
\theta_B (i,j) \\
\theta_C (i,j) \\
\Delta_H (i,j)
\end{bmatrix} = [K]^{-1}[M(i,j)]
$$
Substituting the results of equation 27 into equations 8-17, the end moment of each beam can be obtained. Finally, the moment at sensor location of span 3, $M_{PM}^{ST}(i, j)$, for any random wheel load $P_{i,j}$ equal to

$$M_{PM}^{ST}(i, j) = \begin{cases} \frac{a}{L_3} (P_{i,j}(x_{i,j} - \sum_{x=1}^{2} L_x) + M_{DC}(i, j) - M_{CD}(i, j)) & a \leq \sum_{x=1}^{3} L_x - x_{i,j} \\ \frac{a}{L_3} (P_{i,j}(\sum_{x=1}^{3} L_x - x_{i,j}) + M_{DC}(i, j) - M_{CD}(i, j)) & a > \sum_{x=1}^{3} L_x - x_{i,j} \end{cases} \quad (28)$$

And for the multi-wheel vehicle is moving on the bridge, the final moving moment at the sensor, $M_{PM}^{ST}$, is

$$M_{PM}^{ST} = \begin{cases} \sum_{i=1}^{a} \sum_{j=1}^{a} \frac{a}{L_3} \left[ P_{i,j}(x_{i,j} - \sum_{x=1}^{2} L_x) + M_{DC}(i, j) - M_{CD}(i, j) \right] & a \leq \sum_{x=1}^{3} L_x - x_{i,j} \\ \sum_{i=1}^{a} \sum_{j=1}^{a} \frac{a}{L_3} \left[ P_{i,j}(\sum_{x=1}^{3} L_x - x_{i,j}) + M_{DC}(i, j) - M_{CD}(i, j) \right] & a > \sum_{x=1}^{3} L_x - x_{i,j} \end{cases} \quad (29)$$

Where $SM$ indicates simulation model, $M_{CD}(i, j)$ and $M_{DC}(i, j)$ are the end moment of element from Equations 16 and 17, respectively.

Hazard Characterization of an Extreme event Using the Simulation Model

Effect of Interaction of the Road Profile and the Vehicle Suspension System

The stiffness of a bridge is usually greater than the stiffness of vehicle suspension systems. For example, the first foundational frequency for the bridge on highway US-78 is 13.65 Hz (Appendix A), while the frequencies of vehicle suspension systems range from 2 to 5 Hz [11-12]. To simplify the simulation, the vibration interaction between the bridge and a vehicle can be considered as a vehicle interacting with a rigid road surface (Figure 7). This figure shows a simplified model that includes the interaction of the bridge road roughness and the vehicle suspension system. Equations 30, 31, and 32
present the vertical dynamic loads for the interaction between the road profile and the vehicle suspension system

\[ U_g = \sum_{r=1}^{\infty} (U_{go,r} \sin(\omega_r + \phi_r)) \]  

(30)

\[ F_g^D(i,j) = -W(i,j) + F_k(i,j) + F_c(i,j) \]  

(31)

\[ F_g^D(i,j) = -W(i,j) + \sum_{r=1}^{\infty} [U_{st,r}(i,j)R_d(i,j)K_V \sin(\omega_r(t-\phi_r)) + C_V \omega_r \cos(\omega_r(t-\phi_r))] \]  

(32)

where \( U_{go} \) is the displacement amplitude of harmonic ground motion; \( U_{st} \) is the wheel displacement; \( R_d \) is the deformation response factor; \( r \) is the \( r \)-th of the road profile; \( \omega \) and \( \phi \) are the frequency and phase angle of road profile, respectively; \( K_V \) is the combined stiffness of vehicle suspension system; and \( C_V \) is the combined damping of the vehicle suspension system.

Therefore, the effects from interaction of the road profile and the suspension system are simulated in Equations 28 and 29 by evaluating \( M_{PM}(i,j) \), which can be obtained by substituting \( P_{i,j} \) of Equation 25 with \( F_g^D(i,j) \).

**Horizontal Friction Load Due to Rolling Friction and Emergency Braking**

Vehicle tires act above the bridge surface with the friction load (Figure 8), which relies on the friction coefficient. This friction includes two statuses: tire rolling friction and static friction (emergency brake) [10]. Under normal traffic conditions, the tire rolling friction coefficient, \( \mu_r \), is about 0.01- 0.015 [13], and at emergency braking, the static friction coefficient \( \mu_s \) can have a maximum value of 0.8 for a heavy truck [10]. The friction load \( F_f \), a function of the tire reacting force, is defined in Equation 25. It replaces the static friction load of Equation 33.
\[ F^D_{gr}(i, j) = \mu_r F^D_{g}(i, j) \]  

(33)

where \( F^D_{g}(i, j) \) is the dynamic load in Equation 32.

Therefore, the effects of a horizontal friction load are simulated in Equation 29 by evaluating \( M_{pm}(i, j) \), which can be obtained by substituting \( \mu_r P_{i,j} \) of Equation 25 with \( \mu_r F^D_{gr}(i, j) \).

**Simulation Model Subjected to a Friction Load of Emergency Braking**

When a truck moves on a highway under normal traffic conditions, the tire friction force is a low percentage of the truck weight, and it does not have a significant horizontal impact on the bridge. However, if the vehicle, moving with high speed on the bridge and suddenly stops, the static friction coefficient is as high as 0.8 for a heavy truck. This massive horizontal load will test the bridge’s capacity and may damage the bridge and cause it to lose function. To recognize such a possible condition, the simulation model for effects of full braking was developed and analyzed (Figure 9).

The simulation assumptions are applied as followings: 1) identify bridge properties and relate the traffic information to the bridge simulation model, 2) simulate the braking condition of a heavy truck on the bridge during emergencies. The information, such as speed and road roughness, were kept the same as for B-WIM testing, except for replacing the tire rolling friction coefficient of 0.015 with the tire static friction coefficient of 0.8. In this simulation, three cases were considered: 1) one vehicle emergency brakes at lane 1; 2) one vehicle emergency brakes at lane 2, and 3) each lane has one vehicle braking in an emergency. The brake distance, \( S = \frac{V^2}{2a} \), can be determined by the testing
information, where V is the velocity of vehicle before braking. The vehicle stops acceleration according to \( a = \mu g \).

Validation of Simulation Model using the B-WIM Testing Moment

**B-WIM Testing Moment**

The bridge on US-78 was subjected to B-WIM testing, and the strain signals were collected and analyzed. Based on the relationship between the moment strain \( \varepsilon = \frac{\sigma}{E} \) and stress \( \sigma = \frac{Mr}{I} \), the B-WIM testing moment at each girder \( M_{WIM}(k) \), and the total moment \( M_{WIM} \) are defined as

\[
M_{WIM}(k) = \sum_{i=1}^{n} \sum_{j=1}^{m} \frac{\varepsilon_k(i, j)(EI)_k}{r_k} = \sum_{i=1}^{n} \sum_{j=1}^{m} \frac{\varepsilon_k(i, j) EI D_k}{r_k} \quad (34)
\]

\[
M_{WIM} = \sum_{k=1}^{w} \sum_{i=1}^{n} \sum_{j=1}^{m} \frac{\varepsilon_k(i, j) EI D_k}{r_k} \quad (35)
\]

where \( \varepsilon_k(i, j) \) is the moment strain for the wheel \( i \) at location \( j \) of girder \( k \), \( r_k \) is the bottom arm based on the effective girder cross section, \( EI \) is the effective flexural stiffness, \( D_k \) is the girder distribution factor, \( w \) is the total girder number, \( n \) is total scan number of the B-WIM system, and \( m \) is the total wheel load number.

**Bridge Parameters for the Simulation Model**

Based on the vibration signals of the field B-WIM testing, the bridge damping ratio, \( \zeta = 0.04 \) (Appendix B), and the bridge natural frequency of first bending, \( f_1=13.65 \) Hz (Appendix A), were identified. The bridge flexural stiffness was identified as shown in Table 2. A wheel rolling friction coefficient of \( \mu=0.015 \) was applied in this simulation.
To determine the fixity factor of rotational spring, the analysis steps were as follows:

1. Select one B-WIM testing result and plot the testing moment.

2. In the simulation program trial, the fixity factor ranged from 0 to 1.0 for each member one at a time, the other parameters were automatically revised with the program.

3. Determine if the moment from the simulation model matches the field-testing moment from the B-WIM system, find the convergence of the fixity factor, and finalize the fixity factor of each member.

4. Perform the next run.

Upon applying the initial rotational stiffness for each connection joint, and, by trial-and-error using 20 testing results, the rotational spring constant and the fixity factor are shown in table 3, and the horizontal spring constants were derived as 1.46E+7 kn/m.

**Verification of Frame Simulation Model**

For the highway bridge on US-78, 20 B-WIM tests were conducted in 2008, and the data were analyzed. Figure 10 shows the dynamic moment of the simulation model (SMDY) and the moment of B-WIM for one case; the results of the other 19 test were similar (not shown). The dynamic moment from the simulation model is close to the B-WIM testing moment curve. When the vehicles traffic at spans 1 and 2 (negative X coordinates), a negative moment exists in the simulation model and in B-WIM testing. At spans 2 and 3, the static moment and dynamic moment are similar to results for B-WIM testing. The results demonstrate that the connection joints are not ideally simple.
connections but have moment transfer and that each part of bridge does not respond individually but interact by each span.

Based on figure 10, the simulation model represents the bridge behavior for moving vehicles, and the model can be applied to simulate the bridge response for emergency braking of vehicles.

Case Study of the Bridge on US-78

Results

Figure 11 shows the results of bridge horizontal movement, moments of column ends, and moments of the testing location under normal traffic conditions and under emergency braking situations with cases 1 and 2. The results are summarized as following:

1. At normal traffic conditions for one vehicle at lanes 1 and 2, the maximum horizontal movement is small (Figure 11(a)). At the sensor location on the testing span, the maximum moment is 480 kN-m (Figure 11(d)), and, at the bottoms of column ends E and F, the maximum moment is about 16kN-m (Figure 11(b), 11(c)).

2. For emergency braking in the simulation model, for either lane 1 or lane 2, the maximum horizontal movement increases significantly, but the horizontal deflection of 2.5 mm is still in the control (Figure 11(a)). At the sensor location of the testing span, the maximum moment is about 570 kN-m (Figure 11(d)), and at bottom of column ends E and F, the maximum moment is about 500 kN-m (Figure 11(b), 11(c)).

3. For simulation case 3 in the emergency braking model, when vehicles are in both lanes 1 and lane 2, the maximum horizontal movements and related member moments are double those for cases 1 and 2.
4. Figure 12 shows comparisons of the moment responses between normal traffic and emergency braking. In this figure, the solid line is the WIM experimental moment at normal traffic conditions. And the hidden line is the simulated response moment when a semi-trailer truck brakes in an emergency. From this figure, after the vehicle brakes, the moment curve has significant moment variation, the maximum moment varies from 400 KN-m (before brake) to 500 KN-m (after brake), and the moment is increases by about 25%. Thus, emergency braking causes a significant moment change for the bridge, which presents a safety consideration.

Discussion

1. Based on field measurements of gap and design information, the gap of the horizontal joint ranges from 10 mm to 25.4 mm (3/8 in. to 1 in.). According to the simulation results, the deformation is about 5 mm when two vehicles emergency brake on the bridge. From the above analysis, the bridge is safe in regard to the horizontal deformation.

2. At the mid-span sensor location, the maximum moment was about 1.25 times higher at emergency braking than at the normal traffic situation (Figure 11(d)). This is an essential moment increase for beams or girders. Since, with economic growth, freight transportation will increase on the highway, and since the 2010 ACT allows increased vehicle capacity, there is a concern about the capacity of the existing bridge.

3. Also, at the bottom of column ends “E” and “F”, the maximum moment is almost zero under normal conditions but increases to as much as 500 kN-m for one semi-
trailer truck and to 1,000 (kN-m) for two semi-trailer trucks. This increase of the moment has a significant influence on the column capacity of the bridge and also the as-built foundations.

From above the analysis, when a vehicle brakes in an emergency situation on the bridge, the bridge loading significantly increases and is a potential risk to the bridge capacity. Under normal conditions, there is a low possibility of multi-vehicle braking at the same time. The risk prediction, however, shows that this bridge has a safety issue about its capacity under conditions of emergency braking.

**Retrofit Recommendation**

Although the existing bridge has operated for many years, and no problems in operation have been observed, there may be problems in the future, even though they may not be determined by visual inspection. In order to avoid the possibility of overstressing when vehicles brake in emergency situations, the following options are available: (1) routine visual inspection to confirm that the bridge is safe; (2) traffic control to assure safety, and (3) retrofitting. These options are only for future consideration; no action is required due to the current situation for the bridge.

1. To confirm that the bridge is in good condition, the first selection is routine visual inspection, specifically checking for cracks at the bottom of the girders and at the column ends. If any cracks occur at these locations, the next two options can be considered.

2. In order to reduce horizontal movement that may significantly influence the bridge capacity when vehicles brake in emergency situations, traffic control can reduce
damage. This can be accomplished by control of the traffic volume, thus reducing the possibility that multiple vehicles are present on bridge at the same time. Further, a decrease in vehicle speed would reduce the braking distances on the bridge. In practice, however, such traffic controls, are not always desirable or affordable.

3. If cracking is a serious issue for bridge capacity, retrofitting could be a better alternative to avoid further damage. A retrofit can be conducted for this bridge, as shown in Figure 13. To restrain horizontal movements, a horizontal support could be provided at one end, with no changes for the other end. This retrofit can effectively reduce the moment increase. The simulation results of the retrofit are shown in Figures 11 and 12. Figure 11 shows that horizontal movements and moments for emergency braking return to the normal level; Figure 12 shows the moment (dotted line) for emergency braking has only about a 3% increase for maximum moment.

4. After retrofitting, this bridge has no potential safety issue about moment capacity when vehicles brake in an emergency. This retrofit would protect the bridge from damage and extend its operational lifetime.

Conclusion

A simulation model of the frame structure for the bridge has been developed and validated based on B-WIM experimental data. For the validated model, simulations of vehicle emergency braking were conducted, and the bridge responses were predicted. The simulation results show that the bridge has no problem related to horizontal movement, but it has a potential safety issue about moment capacity. Therefore, a retrofit is
recommended to improve the performance of the bridge and to reduce the possibility of over-reaction by the bridge.

Reference


Table 1. Initial vehicle calibration information of the bridge on highway US-78

<table>
<thead>
<tr>
<th>Vehicle Number</th>
<th>Axle Weight (kN)</th>
<th>Wheel Space (m)</th>
<th>Traffic Speed Range (km/h)</th>
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<td></td>
<td>GVW 1st axle</td>
<td>2nd axle</td>
<td>3rd axle</td>
</tr>
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<td>351.4</td>
<td>49.15</td>
<td>69.61</td>
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<tr>
<td>2</td>
<td>347.8</td>
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</tr>
</tbody>
</table>

Table 2. Bridge properties

<table>
<thead>
<tr>
<th>Dimension (m)</th>
<th>Bridge Elastic Stiffness (kN-m²)</th>
<th>Natural Frequency (Hz)</th>
<th>Tire Rolling Friction μ</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>L₁= 12.8</td>
<td>EI₁= 1.07E+07</td>
<td>f₁=13.65</td>
<td>~ 0.015</td>
</tr>
<tr>
<td>L₂= 12.8</td>
<td>EI₂= 1.07E+07</td>
<td>f₂=18.20</td>
<td></td>
</tr>
<tr>
<td>L₃= 12.8</td>
<td>EI₃= 1.07E+07</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Column</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>L₄= 9.14</td>
<td>EI₄= 2.5E+06</td>
<td></td>
<td></td>
</tr>
<tr>
<td>L₅= 9.14</td>
<td>EI₅= 2.5E+06</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 3. Semi-rigid rotational spring constant in the frame model

<table>
<thead>
<tr>
<th>Semi-rigid connection joint</th>
<th>Kₛ,AB</th>
<th>Kₛ,BA</th>
<th>Kₛ,BC</th>
<th>Kₛ,CB</th>
<th>Kₛ,CD</th>
<th>Kₛ,DC</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spring constant (MN-m/Rad)</td>
<td>0~85</td>
<td>765~1146</td>
<td>0</td>
<td>255~412</td>
<td>765~1146</td>
<td>0~85</td>
</tr>
<tr>
<td>Fixity factor α</td>
<td>0~0.1</td>
<td>0.5~0.6</td>
<td>0</td>
<td>0.25~0.35</td>
<td>0.5~0.6</td>
<td>0~0.1</td>
</tr>
</tbody>
</table>
Figure 1. Sensor layout of the bridge on highway US-78

Figure 2. Field B-WIM testing for bridge US-78

(c) Bridge view, (b) Field WIM testing
Figure 3. Simulation model for the bridge on highway US-78

Figure 4. Fixed boundary conditions of the bridge on US-78
Figure 5. Displacements of the semi-rigid frame element

Figure 6. Expansion boundary conditions of the bridge on US-78
Figure 7. Model for vehicle and road interaction

Figure 8. Horizontal and vertical force model
Figure 9. Simulation model for vehicle emergency braking

Figure 10. Bridge moment comparisons
(a) Horizontal Movement

(b) Column End Moment $M_E$
Figure 11. Comparison of results

(a) Horizontal movement; (b) Column end moment at joint “E”; (c) Column end moment at joint “F”; (d) Beam moment at testing sensor location
Figure 12. Moment compressions between normal traffic and emergency braking

Figure 13. Retrofitted boundary condition
APPENDIX A

Natural Fundamental Frequency

Data for the B-WIM testing strain were exported to signal processing software, DADiSP (DADiSP, 2000) for post-processing and analysis. The steps were as follows:

4. Eliminated forced vibration signals and kept free vibration signals only.
5. For output signal, eliminated data after the free vibration died down.
6. Converted signals into frequency domain by using fast Fourier transformation (FFT).

The natural frequency could be determined by DADiSP software, as shown in Figure A1. The natural frequency of the first mode was 13.65 hz, and that for the second natural frequency was 18.2 hz.

Figure A.1. Natural frequency analysis of B-WIM for free vibration (Double click, enlarge)
Appendix B

Bridge Damping Ratio

By applying the free vibration of B-WIM data for the bridge on highway US-78 (Figure B1), the damping ratio of this bridge can be determined. Since the decay of motion is slow for this case of a lightly damped system, multi cycles, $j$, of the motion were used to determine the damping ratio instead of the successive amplitude. The damping ratio is given by

$$\delta = \frac{1}{j} \ln\left(\frac{\mu_i}{\mu_i + j}\right) \equiv 2\pi\xi$$

(B.1)

From Figure 1, $\mu_i = 1.77$ and $\mu_i + j = 0.86$ for $j=4$, and central coordinate line is 0.32. The damping ratio can be determined by

$$\delta = \frac{1}{4} \ln\left(\frac{1.77 - 0.32}{0.86 - 0.32}\right) \equiv 2\pi\xi$$

$$\xi = 0.0393$$

Similar results were achieved for analyses of the other 19 cases. Thus, for this bridge, a damping ratio of 4% is applied to the program.

Figure B.1. Damping ratio.
DETERMINATION OF DYNAMIC AMPLIFICATION FACTORS USING SITE-SPECIFIC B-WIM DATA FOR A BRIDGE

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Abstract

The Dynamic Amplification Factor (DAF) is a significant parameter for new bridge design and characterization of existing bridges. The AASHTO code provides a high conservative value relative to field-testing results. Recently, the Safe and Efficient Transportation Act (2010) increased the traffic truck weight from 355.84 KN (80 kips) to 431.5 KN (97 kips). Thus, improved methods are needed to determine accurate DAF values for new bridge designs and to evaluate the site-specific DAFs of existing bridges. The present report proposes a simulation method to evaluate the DAFs of existing bridges using bridge weigh-in-motion (B-WIM) data. This approach includes the static moment obtained from the simulation model and the dynamic moment obtained from the B-WIM experimental data. A model for multiple vehicles is developed to simulate the traffic intensity and to predict the DAF of an existing bridge. Presented are the experimental findings and determination of the site-specific DAF value.

Keywords: Bridge, Moment Strain, Bridge Weigh-in-motion, B-WIM, Dynamic Amplification Factor, DAF, DIF

Introduction

When a vehicle is moving on a bridge, the oscillations between the bridge and the vehicle cause dynamic reactions of the wheel loads in response to variations in the road profile as a function of the vehicle suspension system and the stiffness of bridge. In some cases, the forces of the wheel reactions are higher or lower than the vehicle’s static load. The dynamic amplification factor (DAF) value varies with environmental factors and
bridge configurations, and it can be determined based on the dynamic behavior of the bridge, which is a function of the vehicle’s dynamic properties, the bridge’s structural properties, and road roughness (Kwon et al. 2010).

The dynamic impact factor (DIF) is defined as the dynamic load exceeding the static load divided by the static load. Thus, the DIF resulting from passage of a vehicle on a bridge is

\[
DIF = \frac{R_d - R_s}{R_s}
\]

where \( R_d \) is the maximum dynamic response of the bridge, and \( R_s \) is the maximum static response. Therefore, the DAF is

\[
DAF = (1 + DIF) = \frac{R_d}{R_s}
\]

Before 1992, application of the DIF was different in most nations, but the disagreement of DIF applied by different countries was not included in various codes (AASHTO, 1982 and 1989; OHBDC; Euro code). Now, however, DAFs in terms of the AASHTO standard and the Ontario Highway Bridge Design Code (OHDBC) have been adopted by many counties, such as the DIF of the AASHTO standard (2002), which reflects the parameter of bridge span length, and the DIF of OHDBC which is based on the natural frequency of the bridge. Neither code presents the accurate dynamic behavior; they only attempt to interpret the sufficient, reasonable, and conservative need for bridge design. Over 20-30 years, numerous experimental procedures have been applied. These techniques, such as impact testing, vehicle testing, static testing, and other dynamic testing, are somewhat appropriate for evaluating the dynamic vibration (Hwang et, al 1991; Kirkedaaad et, al 1997). But, as determined by the research results, it is best to
determine the DAF value in a normal traffic environment, by use of as B-WIM (or simulated-WIM) testing (Bakht et al, 1989; Brady, 2006; DIVINE, 1997; Herris et al, 2006; Heywood, 2001; Jacob, 1996; oBrien et al, 2009). As mentioned previously, traditional methods tend to be conservative to account for uncertainty levels in the DAF applied to a structure and the structure’s response to those loads. In many cases, these methods also neglect potential sources of reserve capacity (i.e., additional strength resulting from the slab/girder composite action in bridges designed as non-composite, rigid, or semi-rigid connections designed as flexible), resulting in an underestimation of the structural safety. Safety assessments often are a part of the permit review process for oversize/overweight (OS/OW) movements to verify the integrity of bridges along the permitted vehicle’s intended route. The noted improvements to theoretical assumptions supporting load DIFs afforded through B-WIM systems - combined with the added certainty in actual stress characteristics - enhance the accuracy of conclusions regarding the safety of an existing bridge under the intended vehicle loading. The present report investigates the DAF value that can be used to identify and quantify such sources of additional capacity present in the structure but not accounted for in theoretical models. As a less costly, less intrusive method, bridge strain data can support DAF determinations using normal traffic loading and the observable structural behavior of the bridge. As such, DAF can be performed with little to no advance preparation or cost, particularly if the structure of interest is already instrumented with a B-WIM system. Based on the testing signals, the DAF can be determined in the following assumptions. The dynamic amplification of bridges subjected to moving loads was found to be a function of several factors, such as the weight of the loads, the dimensions of the bridge, the deck’s surface
roughness, the natural frequency of bridge vibration, and the dynamic properties of the moving vehicle.

There are several available methods to estimate the DAFs for typical bridge configurations. These methods vary in their levels of sophistication and specifications and produce remarkably different results for the same bridge. For example, there are differences among the DAF values of the AASHTO LRFD specifications (2003, 2004), the impact factor equation of the AASHTO Standard Specifications, and the natural frequency-related equation of the Ontario Highway Bridge Design Code (OHBDC). DAF values were obtained based on a limited number of field measurements and simulation studies. There is a need to supplement these investigations by testing the dynamic behavior of a large range of bridge configurations.

In this study, a site-specific evaluation of the DAF is simulated to include evaluation of the dynamic moment based on experimental B-WIM data and static moment based on a sophistication frame structure. Also, a simulation of multiple vehicles is developed to predict DAF values.

US-78 Bridge Description

The highway bridge on US-78 at Graysville, Alabama, USA, has three single-spans of 12.8 m (42 ft) each and is supported by two square columns of 945 mm x 945 mm (3 feet x 3 feet) at each bent (ALDOT, 1958). The B-WIM testing was conducted by the Alabama Department of Transportation (ALDOT) in 2008 (Zhao, 2010). The sensors were mounted underneath at the mid-spans of each girder. The bridge information and field testing are shown in Figures 1 and 2. The initial calibration test was performed
under test condition (R1-I) according to the European specifications for B-WIM (Cost 323, 2002). The two representative vehicles for testing were semi-trailers with a loading capacity of 355.8 KN (80000 lbs, fully loaded, five-axle trailer truck) as pre-weighed trucks from the ALDOT. The effective signals of 10 runs for each lane were evaluated for both vehicles. Table 1 provides detailed information for these vehicles.

Simulation Model for Static Moment

In practice, the simplified structural model of bridge US-78 did not reflect the behavior of the bridge. In order to identify the true moment, the simulated model is presented in Figure 3. For this model, the semi-rigid rotational joint, boundary condition, and flexural stiffness are discussed as follows.

Semi-rigid Rotational Joint

In reality, the influence line of a simply supported bridge is not triangular and may be between the simply supported and fixed cases (Žnidarič & Baumgartner, 1998). The reason is that the supported joints are not in “ideal” single support, with the capacity to transfer the forces and moments. For the connection of a simply-supported bridge, traditional designs neglect the real behavior of connections. Thus, the idealization of a pinned connection was used in the design. However, the predicted response of a bridge may not be realistic. In practice, most connections have the rotational capacity contributing to structure displacements. As shown in Figure 4, for the fixed joint of the bridge US-78, the vertical rebar, bridge girders, and the support columns work together to restrain the joint. However, this connection cannot fully transfer all forces. Therefore,
these types of joints can be represented as semi-rigid rotational joints. The semi-rigid connection can be simulated in the analysis of beam-column connections. This flexible connection behavior affects the internal force distribution of the frame structure, and a more reliable prediction of frame behavior can be obtained by use of a semi-rigid rotational spring.

Therefore, the multi-span of bridge US-78 behaves similarly to a frame. A sophisticated method of frame analysis was adopted in this research. A linear representation of the spring was developed to analyze the bridge frame with a semi-rigid connection for each beam element, as shown in Figure 5. The effects of the connection flexibility are modeled as rotational spring constants $S_j$ and $S_k$, where $j$ and $k$ are the ends of a frame element, and $\Phi_j$ and $\Phi_k$ are rotations incurred by rotational springs.

According to first-order analysis, the stiffness matrix of a member with semi-restraint at the ends can be represented by the correction stiffness matrix based on the rigid connections (Monforton & Wu, 1963; Simoes, 1995; Ali & Hikmet, 2005).

\[
\begin{bmatrix}
\frac{EA}{L} & 0 & 0 & \frac{EA}{L} & 0 & 0 \\
\frac{12EI}{L^3} (\alpha_1 + \alpha_2 + \alpha_3) & \frac{6EI}{L^2} (2\alpha_1 + \alpha_2) & \frac{6EI}{4L} (\alpha_1 + \alpha_2) & \frac{6EI}{L^3} (2\alpha_2 + \alpha_3) & \frac{6EI}{4L} (2\alpha_2 + \alpha_3) & \frac{6EI}{L^3} (2\alpha_2 + \alpha_3) \\
0 & \frac{6EI}{L^3} (2\alpha_1 + \alpha_2) & \frac{6EI}{L^2} (2\alpha_2 + \alpha_3) & \frac{6EI}{4L} (\alpha_1 + \alpha_2) & \frac{6EI}{L^3} (2\alpha_2 + \alpha_3) & \frac{6EI}{4L} (2\alpha_2 + \alpha_3) \\
0 & \frac{6EI}{L^3} (2\alpha_1 + \alpha_2) & \frac{6EI}{L^2} (2\alpha_2 + \alpha_3) & \frac{6EI}{4L} (\alpha_1 + \alpha_2) & \frac{6EI}{L^3} (2\alpha_2 + \alpha_3) & \frac{6EI}{4L} (2\alpha_2 + \alpha_3) \\
0 & \frac{6EI}{L^3} (2\alpha_1 + \alpha_2) & \frac{6EI}{L^2} (2\alpha_2 + \alpha_3) & \frac{6EI}{4L} (\alpha_1 + \alpha_2) & \frac{6EI}{L^3} (2\alpha_2 + \alpha_3) & \frac{6EI}{4L} (2\alpha_2 + \alpha_3) \\
0 & 0 & 0 & 0 & 0 & 0 \\
\end{bmatrix}
\]

(3)

If the axle and shear deformations are neglected, the stiffness matrix can be modified as in Equation 3.
where \( EI \) is the flexural stiffness, and \( L \) is the length. In equation 5, the parameters \( \alpha_1 \) and \( \alpha_2 \) are the fixity factors at each end of the member, and both factors are related with rotational spring stiffness, \( S_j \) and \( S_k \).

\[
\alpha_j = \frac{1}{1 + 3EI/S_j L} ; \quad \alpha_k = \frac{1}{1 + 3EI/S_k L} \quad (5)
\]

The fixity factor \( \alpha \) determines the stiffness of the connection relative to the attached beam and to the rotational capacity of moments. Following the conventional matrix procedures of displacement for a rigid-jointed frame, the adjusted end-moments, \( M_j \) and \( M_k \), for the semi-rigid connection member are defined as follows:

\[
M_j = \frac{3\alpha_k(2 - \alpha_2)}{4 - \alpha_1\alpha_2} M_j' ; \quad M_k = \frac{3\alpha_j(2 - \alpha_1)}{4 - \alpha_1\alpha_2} M_k' \quad (6)
\]

where \( M_j' \) and \( M_k' \) are the end moments for a rigid connection.

The adjusted end moment of the member is defined as

\[
\begin{bmatrix} M_k \\ M_j \end{bmatrix} = k^* \begin{bmatrix} \theta_j \\ \theta_k \end{bmatrix} + \begin{bmatrix} M_j \\ M_k \end{bmatrix} \quad (7)
\]

where \( \theta_j \) and \( \theta_k \) are the rotations at end elements \( j \) and \( k \).

In linear elastic analyses, if the connections are assumed to have a linear force-displacement relationship, and the axial and shear deformation of members are ignored, the solution of member moment can be achieved by the set of linear equations

\[
K \mu = M \quad (8)
\]

where \( K \) is the bridge stiffness matrix, \( \mu \) is the deformation of bridge, and \( M \) is the external
moment.

**Boundary Condition**

During vehicle moving, the bridge horizontal movements exist with the sensitive parameters influencing the testing strains. Figure 6 shows the boundary conditions of the bridge on US-78 at an expansion joint. The connection was designed as the smooth steel plate lay on the smooth steel pad without vertical ties or horizontal restraint at the expansion joint, so that the bridge has the potential to move in the horizontal direction. A horizontal spring is provided to simulate the horizontal movement at joint “A” (Figure 3), and the horizontal spring constant, $K_H$, is defined as

$$K_H = 0.5\mu_b W_b,$$  \hspace{1cm} (9)

where $\mu_b$ is the friction coefficient between the smooth steel plate and the smooth steel pad (Static and Kinetic Friction, 2012), and $W_b$ is the bridge weight of the related span.

**Formula for Simulation Model**

According to the assumption of the boundary condition and a semi-rigid connection, the structural system of the bridge on US-78 is modeled as a frame structure (Figure 3), subjected to a random wheel $P_{i,j}$ at location $x_{i,j}$, where $i$ is the scan number of the B-WIM system, and $j$ is the wheel number. By applying the slope-deflection method, the end moments of each member are defined as

$$M_{AB}(i, j) = \frac{2(EL)}{L_1} \theta_B(i, j) K_{AB}^1 + M_1^Z \beta_A^1$$  \hspace{1cm} (11)

$$M_{BA}(i, j) = \frac{2(EL)}{L_1} 2\theta_B(i, j) K_B^1 + M_1^R \beta_B^1$$  \hspace{1cm} (12)
where $A$, $B$, $C$, $D$, $E$, and $F$ indicate joint numbers; $\theta_B(i, j)$ and $\theta_C(i, j)$ denote the rotation at joints $B$ and $C$ for load $P_{i,j}$, respectively; $\Delta_H(i, j)$ indicates the horizontal movement for load $P_{i,j}$.

The fixity factor, $\alpha$, and the rotational spring stiffness, $S$, for each member are defined as follows:

For member 1: 
\[
\alpha_A^1 = \frac{1}{1 + 3(El)_1 / S_{AB}L_1}; \quad \alpha_B^1 = \frac{1}{1 + 3(El)_1 / S_{BA}L_1}
\]

For member 2: 
\[
\alpha_B^2 = \frac{1}{1 + 3(El)_2 / S_{BC}L_2}; \quad \alpha_C^2 = \frac{1}{1 + 3(El)_2 / S_{CB}L_2}
\]

For member 3: 
\[
\alpha_C^3 = \frac{1}{1 + 3(El)_3 / S_{CD}L_3}; \quad \alpha_D^3 = \frac{1}{1 + 3(El)_3 / S_{DC}L_3}
\]
The modified factors of stiffness matrix, $K$, and adjusted moment factor, $\beta$, for each member are defined as follows:

For member 1:  
\[
K_A^1 = \frac{3\alpha_A^1}{4 - \alpha_A^1\alpha_B^1}; \quad K_{AB}^1 = K_{BA}^1 = \frac{3\alpha_A^1\alpha_B^1}{4 - \alpha_A^1\alpha_B^1}; \quad K_A^1 = \frac{3\alpha_B^1}{4 - \alpha_A^1\alpha_B^1}; \\
\beta_B^1 = \frac{3\alpha_B^1(2 - \alpha_A^1)}{4 - \alpha_A^1\alpha_B^1}
\]

For member 2:  
\[
K_B^2 = \frac{3\alpha_B^2}{4 - \alpha_B^2\alpha_C^2}; \quad K_{BA}^2 = K_{AB}^2 = \frac{3\alpha_B^2\alpha_C^2}{4 - \alpha_B^2\alpha_C^2}; \quad K_C^2 = \frac{3\alpha_C^2}{4 - \alpha_B^2\alpha_C^2}; \\
\beta_B^2 = \frac{3\alpha_B^2(2 - \alpha_C^2)}{4 - \alpha_B^2\alpha_C^2}; \quad \beta_C^2 = \frac{3\alpha_C^2(2 - \alpha_B^2)}{4 - \alpha_B^2\alpha_C^2}
\]

For member 3:  
\[
K_C^3 = \frac{3\alpha_C^3}{4 - \alpha_C^3\alpha_D^3}; \quad K_{CD}^3 = K_{DC}^3 = \frac{3\alpha_C^3\alpha_D^3}{4 - \alpha_C^3\alpha_D^3}; \quad K_C^3 = \frac{3\alpha_D^3}{4 - \alpha_C^3\alpha_D^3}; \\
\beta_C^3 = \frac{3\alpha_C^3(2 - \alpha_D^3)}{4 - \alpha_C^3\alpha_D^3}
\]

The fixed end moment of member for span $s$ ($s$ denotes the span No. 1, 2 and 3) is defined as

\[
M_s^L = \begin{cases} 
\frac{P_{i,j}x_{i,j}(L_s - x_{i,j})^2}{(L_s)^2} & \text{When } P_{i,j} \text{ locates at span } s \\
0 & \text{others span}
\end{cases}
\]

\[
M_s^R = \begin{cases} 
\frac{P_{i,j}x_{i,j}^2(L_s - x_{i,j})}{(L_s)^2} & \text{When } P_{i,j} \text{ locates at span } s \\
0 & \text{others span}
\end{cases}
\]

From Figure 3, the equations of each joint and force balance are given

\[
M_{BA} + M_{BE} + M_{BC} = 0 \quad (21)
\]

\[
M_{CB} + M_{CF} + M_{CD} = 0 \quad (22)
\]
\[
\frac{M_{BE} + M_{EB}}{L_4} + \frac{M_{CF} + M_{FC}}{L_5} = K_H \Delta_H
\]  

(23)

Substituting Equations 11 – 20 into Equations 21 – 23, we obtain

\[
\left(\frac{4(EI)}{L_4} K_B^1 + \frac{4(EI)}{L_2} K_B^2 + \frac{4(EI)}{L_4} K_C^4\right) \theta_B + \frac{2(EI)}{L_3} K_B^2 \theta_C - \frac{6(EI)}{L_4} \Delta_H = -M_B^R \beta_B^1 - M_B^L \beta_B^2
\]  

(24)

\[
\frac{2(EI)}{L_2} K_B^2 \theta_B + \frac{4(EI)}{L_2} K_C^2 + \frac{4(EI)}{L_3} K_C^3 + \frac{4(EI)}{L_4} K_C^4 \theta_C - \frac{6(EI)}{L_3} \Delta_H = -M_C^R \beta_C^2 - M_C^L \beta_C^3
\]  

(25)

\[
\frac{6(EI)}{L_4} \theta_B + \frac{6(EI)}{L_5} \theta_C - \left(\frac{12(EI)}{L_4} + \frac{12(EI)}{L_5} + K_S^H \right) \Delta_H = 0
\]  

(26)

The assembly stiffness matrix \( K \) for frame structure is

\[
K = 
\begin{bmatrix}
\frac{4(EI)}{L_4} K_B^1 + \frac{4(EI)}{L_2} K_B^2 + \frac{4(EI)}{L_4} K_C^4 & \frac{2(EI)}{L_2} K_B^2 & \frac{6(EI)}{L_4} \\
\frac{2(EI)}{L_2} K_B^2 & \frac{4(EI)}{L_2} K_C^2 + \frac{4(EI)}{L_3} K_C^3 + \frac{4(EI)}{L_4} K_C^4 & \frac{6(EI)}{L_3} \\
\frac{6(EI)}{L_4} & \frac{6(EI)}{L_5} & -\left(\frac{12(EI)}{L_4} + \frac{12(EI)}{L_5} + K_S^H \right)
\end{bmatrix}
\]  

(27)

Where \( EI \) is the effective flexure stiffness, \( L \) is the length.

The joint external moment \( M(i,j) \) for frame structure is

\[
M(i,j) = \begin{bmatrix}
-M_B^R \beta_B^1 - M_B^L \beta_B^2 \\
-M_C^R \beta_C^2 - M_C^L \beta_C^3 \\
0
\end{bmatrix}
\]  

(28)

Thus, following linear Equation 8, the \( M-u \) relationship is defined as

\[
[K] \begin{bmatrix}
\theta_B(i,j) \\
\theta_C(i,j) \\
\Delta_H(i,j)
\end{bmatrix} = [M(i,j)]
\]  

(29)

Solving Equation 29, each parameter can be obtained
\[
\begin{bmatrix}
\theta_B(i, j) \\
\theta_C(i, j) \\
\Delta_R(i, j)
\end{bmatrix} = [K]^{-1}[M(i, j)]
\]  
(30)

Substituting the results of equation 30 into equations 11-20, the end moment of each beam can be obtained. Finally, the static moment at sensor location of span 3, 
\[ M_{SM}^{ST}(i, j), \]
for any random wheel load \[ P_{i,j} \]
equal to
\[ M_{SM}^{ST}(i, j) = \begin{cases} 
\frac{a}{L_3} (P_{i,j} (x_{i,j} - \sum_{s=1}^{2} L_s) + M_{DC}(i, j) - M_{CD}(i, j)) & a \leq \sum_{s=1}^{3} L_s - x_{i,j} \\
\frac{a}{L_3} (P_{i,j} (\sum_{s=1}^{3} L_s - x_{i,j}) (L_3 - a) + M_{DC}(i, j) - M_{CD}(i, j)) & a > \sum_{s=1}^{3} L_s - x_{i,j}
\end{cases}
\]  
(31)

And, for the multi-wheel vehicle is moving on the bridge, the final moving moment at the sensor, \[ M_{SM}^{ST} \], is
\[ M_{SM}^{ST} = \begin{cases} 
\sum_{i=1}^{n} \sum_{j=1}^{m} \left( \frac{a}{L_3} \left[ P_{i,j} (x_{i,j} - \sum_{s=1}^{2} L_s) + M_{DC}(i, j) - M_{CD}(i, j) \right] \right) & a \leq \sum_{s=1}^{3} L_s - x_{i,j} \\
\sum_{i=1}^{n} \sum_{j=1}^{m} \left( \frac{a}{L_3} \left[ P_{i,j} (\sum_{s=1}^{3} L_s - x_{i,j}) (L_3 - a) + M_{DC}(i, j) - M_{CD}(i, j) \right] \right) & a > \sum_{s=1}^{3} L_s - x_{i,j}
\end{cases}
\]  
(32)

where \[ SM \] indicates simulation model, \[ M_{CD}(i, j) \] and \[ M_{DC}(i, j) \] are the end moment of element from Equations 19 and 20, respectively.

Dynamic Moment from B-WIM Testing

The bridge on US-78 was subjected to B-WIM testing, and the strain signals were collected and analyzed. Based on the field testing data, the load distribution factor, \( D_r(i, j) \) and the average load distribution factor \( \overline{D_r} \) are defined as
\[ D_r(i, j) = \frac{\varepsilon_r(i, j)}{\sum_{r=1}^{n_r} \varepsilon_r(i, j)} \]  

\[ \bar{D}_r = \text{Mean} (D_r(i, j)) \]  

where \( \varepsilon_r(i, j) \) is the moment strain at the girder \( r \) from B-WIM testing; \( i \) is the scan number of the B-WIM system; and \( j \) is the wheel number.

Also, based on bridge geometries, the bridge stiffness distribution factor, \( D_r^s \) can be derived, and the elastic stiffness for each girder \( r \), \((EI)_r\), is defined as

\[ (EI)_r = (EI)D_r^s \]  

where \( EI \) is the flexural stiffness for the entire bridge.

The moment strain and moment stress relationships, \( \varepsilon = \frac{\sigma}{E} \) and \( \sigma = \frac{M_r}{I} \), the B-WIM testing moment at girder \( r \), \( M_r^{DY}(i, j) \), and the total moment, \( M^{DY} \), can be derived as

\[ M_r^{DY}(i, j) = \frac{\varepsilon_{rk}(i, j)(EI)_r}{R_r} = \frac{\varepsilon_r(i, j)(EI)D_r^s}{R_r} \]  

\[ M^{DY} = \sum_{i=1}^{n} \sum_{j=1}^{m} \sum_{r=1}^{n_r} \left( \frac{\varepsilon_r(i, j)(EI)}{R_r} \right) \]  

where, \( \varepsilon_r(i, j) \) is the moment strain at the scan number \( i \) for wheel \( j \) and girder \( k \); \( R_r \) is the force arm from bottom edge, \( n \) is the total scan number of the B-WIM system, \( m \) is the number of the total wheel axles, and \( n_r \) is the total girder numbers.
Dynamic Amplification Factor (DAF)

**DAF for an Individual Vehicle**

Bridge strain data were used to refine theoretical estimates of the DAF, - the ratio between maximum dynamic and static response - to estimate the true dynamic effects of live loads on a structure. Based on Equation 2, the DAF value can be defined as the ratio of maximum dynamic moment to maximum static moment, so the DAF value of individual run at girder \( r \), \( DAF_r^I \), is

\[
DAF_r^I = \frac{\text{Max}(M_r^{DY}(i,j))}{\text{Max}(M_r^{ST}(i,j)D_r)}
\]  

(37)

**Simulation for DAF for Group Vehicles**

Usually, for reasons of cost control, limited numbers of typical vehicles (i.e., five-axle semi-truck) are applied to B-WIM testing under normal traffic conditions. In a low-traffic environment, only one vehicle may be on the highway. More often, groups of vehicles are present in actual traffic intensity. It is essential that the group vehicles be present at one lane or multi-lanes at the same time. In this study, we propose the simulation runs of the group vehicles for one lane or multi-lanes, and conduct the analysis to identify the “true” DAF value. To use the current sources of B-WIM data effectively, each individual case of testing was considered as normal traffic. In selecting sufficient vehicles with possible traffic conditions based on traffic volume, all of possible group vehicles were simulated to predict the DAF value. The following simulation rules about the traffic conditions were considered: 1) Random vehicle spacing at one lane or
different lanes; 2) Random vehicle speeds for each individual vehicle; 3) Random numbers of the group vehicles run at an individual lane or multiple lanes (Figure 7).

With the above assumptions, the DAF for group vehicles, \( DA_F^G(N) \), can be defined as

\[
DA_F^G(N) = \frac{\text{Max} \left\{ \sum_{i=1}^{n} \sum_{j=1}^{m_i} \sum_{c=1}^{m_c} \left( M_{r,SM}^{ST} \left( i_{Ln}^c, \sum_{c=1}^{m_c} \left( L_{Ln}^c + j_{Ln}^c + S_{Ln}^c + LS_{Ln}^1 \right) \right) \right) \right\}}{\text{Max} \left\{ \sum_{i=1}^{n} \sum_{j=1}^{m_i} \sum_{c=1}^{m_c} \left( M_{r,SM}^{ST} \left( i_{Ln}^c, \sum_{c=1}^{m_c} \left( L_{Ln}^c + j_{Ln}^c + S_{Ln}^c + LS_{Ln}^1 \right) \right) \right) \right\}} \quad \text{for} \quad N = 1 \rightarrow \infty
\]  

(38)

where \( M_{r,SM}^{ST}(i, j) \) is static moment from the simulation model (Equation 31), \( M_{r,SM}^{DY}(i, j) \) is the dynamic moment from B-WIM experimental data (Equation 36), \( r \) is the bridge girder number, \( i \) is the scan number of the B-WIM system, \( j \) is the wheel number, \( c \) is the \( c^{th} \) vehicle, \( n \) is the scan numbers of the B-WIM system, \( m \) is the number of the vehicle wheel axles, \( m_c \) is the number of group vehicles, \( X \) is the lane number, \( L_{Ln}^c \) is the vehicle length for \( c^{th} \) vehicle at lane \( Ln \), \( S_{Ln}^c \) is the vehicle spacing between \( c^{th} \) vehicle and \((c+1)^{th}\) vehicle at lane \( Ln \), \( i_{Ln}^c \) is the location of the \( c^{th} \) vehicle at lane \( Ln \), \( LS_{Ln}^1 \) is the spacing between each first vehicle of different lanes, and \( N \) is the N\textsuperscript{th} simulation case.

Results

**Determination of Bridge Parameters**

Base on the vibration signals of the field B-WIM testing, the bridge flexural stiffness was identified as shown in Table 2.

To determine the fixity factor of rotational spring, the analysis steps are following:

1. Select one B-WIM testing result and plot the testing moment.

2. In the simulation program trial, the fixity factor ranged from 0 to 1.0 for each
member one at a time, the other parameters are automatically revised with the program.

3. Compare the moment from simulated model to match the field-testing moment from the B-WIM system and find the convergence of fixity factor, then finalize the fixity factor of each member.

4. Perform the next run.

Upon applying the initial rotational stiffness for each connection joint, and, by trial and error using 20 testing results, the rotational spring constant and the fixity factor are shown in table 3, and the horizontal spring constants was derived as 1.46E+7 kn/m.

Verification of the Simulation Model

For the highway bridge on US-78, 20 B-WIM tests were conducted in 2008, and the data were analyzed. Figure 8 shows that static moment of simulated model (SMST) and the moment of B-WIM for one case, the results of other 19 tests were similar (not shown). In this figure, the static moment from the simulated model is similar to the B-WIM testing moment curve. When the vehicles traffic at span 1 and span 2 (negative X coordinates), there is a negative moment in the simulated model and B-WIM testing. At spans 2 and 3, the static moment has similar results with B-WIM testing. At span 1, however, the large error is due to the effect from outside resource of activated loads (such as vehicle loads influence the retaining wall of bridge). The results demonstrate that the connection joints are not simple connections but have moment transfer and that parts of bridge do not work individually but interact by each span.
Parameters of DAF Application

In order to estimate the DAFs following the AASHTO design code of HS20 loading, the chosen vehicle for this B-WIM testing was the five-axle semi-truck with a weight of 354.84 KN (80 kips). In this line group simulation, we assumed the total random vehicle \( M_c = 10 \) to be one line group. To model the least favorable possibility, the vehicle spacing, \( S_{Ln} \), was considered at a range from 18.29 m (60 feet) to 45.72 m (150 feet), and the first vehicle spacing in a different lane, \( S^i_{Ln} \), was set at ranges from -15.24 m (-50 feet) to 15.24 m (50 feet). For group simulation, total cases of 2,000 were simulated for both lanes (\( L_n = 2 \)). Figure 6 presents three simulation cases as discussed previously.

Dynamic Amplification Factor

Following is a summary of DAF findings from this study.

a. For individual B-WIM testing, the DAF values of each girder and the whole bridge for 20 tests are presented in Figure 9. The DAF values at side girders 1 and 4 were generally <1.3, and the DAF values at the internal girders 2 and 3 were as high as 1.5. The DAF value of the entire bridge is < 1.2.

b. For group vehicle simulation runs, a total of 2,000 simulations were analyzed. Figures 10-12 show the DAF of each girder for a different case. For simulation of group vehicle at lane 1, the DAF values are shown in Figure 10. When trucks traffic at lane 1, girders 1 and 2 carry about 35% of the load at each girder, girder 3 carries about 25% of the load, and girder 4 carries about 5% of the load. From this figure (a), (b) and (c), most of the DAF values are less than 1.2, 1.5 and 1.2, respectively. Nevertheless, from figure (d), the DAF is scattered when it was over...
1.2, with some values as high as 1.55. As discussed above, girder 4 takes only about 5% of the load in this simulation; the high DAF value is due to transverse vibration existed. Also from figure (e), the DAF of the whole bridge is < 1.3.

c. For simulation of group vehicles at lane 2, The DAF values are shown in Figure 11. When a truck traffics at lane 2, girder 1 takes only about 5% of the load, girder 2 takes about 25% of the load, and girders 3 and 4 take about 35% of the load. From this figure (b), (c) and (d), the most of the DAF values were <1.38, 1.6, and 1.25, respectively. But from figure (a), the DAF was scattered when it was over 0.9. Also from figure (e), the DAF for the whole bridge was as high as 1.55.

d. For simulation the case at both lanes, the DAF values are shown in Figure 12. For girders 1 to 4, most of the DAF values are about 1.15, 1.5, 1.45, and 1.2, respectively, and the DAF for whole bridge is about 1.3. Also, side girders 1 and 4 have lower DAF values than internal girders 2 and 3. This is because the internal girders carry more load response.

e. A summary of the results of all three combination cases are in Table 4, which shows comparisons of DAF values among the AASHTO code, single testing running, and simulation group running. From this table, 79% of cases have > 1.3 of the DAF value at girder 2 of lane 1; 61% of cases have > 1.3 of the DAF value at girder 3 of lane 2. For simulation at both lanes, 27% and 12% cases have > 1.3 of the DAF values at girders 2 and 3, but the two side girders have lower DAF values. Thus, the DAF is higher than the AASHTO code requirement and is in disagreement with the design code for this bridge.
Conclusions

Based on B-WIM data, a model of the frame for a multi-span bridge was developed to simulate the static moment, and, by field testing, a simulation approach was developed to predict site-specific DAF values for group vehicles. The DAF can be predicted for this bridge based on the field B-WIM data. This approach predicts that as high as 79% simulation cases at traffic lane 1 are beyond the AASHTO requirements (DAF=1.3), and also that the DAF is as high as 1.6 in the inter girders. There are lower DAF values at the side girders, however, and these meet the code requirements. Based on the results, this newly developed simulation approach can be applied to predict the site-specific DAFs of existing bridges.

Acknowledgements

The authors gratefully acknowledge funding and support provided by National Science Foundation (NSF) for this research project (CMMI-1100742).

References


Table 1. Initial vehicle calibration information for the bridge on highway US-78

<table>
<thead>
<tr>
<th>Vehicle Number</th>
<th>Axle Weight (kN)</th>
<th>Wheel Space (m)</th>
<th>Traffic Speed Range (km/h)</th>
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<tr>
<td></td>
<td>1st axle</td>
<td>2nd axle</td>
<td>3rd axle</td>
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<td>1</td>
<td>351.4</td>
<td>49.15</td>
<td>69.61</td>
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<td>2</td>
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Table 2. Bridge properties

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<th>Dimension (m)</th>
<th>Bridge Elastic Stiffness (kN-m²)</th>
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<td>EI₁ = 1.07E+07</td>
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<tr>
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<tr>
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Table 3. Semi-rigid rotational spring constant

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<th>Semi-rigid connection joint</th>
<th>Kₛ,AB (MN-m/Rad)</th>
<th>Kₛ,BA (MN-m/Rad)</th>
<th>Kₛ,BC (MN-m/Rad)</th>
<th>Kₛ,CB (MN-m/Rad)</th>
<th>Kₛ,CD (MN-m/Rad)</th>
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<td>Spring constant</td>
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<td>255~412</td>
<td>765~1146</td>
<td>0~85</td>
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<tr>
<td>Fixity factor α</td>
<td>0~0.1</td>
<td>0.5~0.6</td>
<td>0</td>
<td>0.25~0.35</td>
<td>0.5~0.6</td>
<td>0~0.1</td>
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Table 4. DAF comparisons

<table>
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<th>Girder</th>
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<th>Girder 4</th>
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<td>1.30</td>
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<tr>
<td>Single Run Max value (10 cases)</td>
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<td>1.43</td>
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<td>1.52</td>
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<th>Girder 3</th>
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<td>1.26</td>
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<table>
<thead>
<tr>
<th>Girder</th>
<th>Girder 2</th>
<th>Girder 3</th>
<th>Girder 4</th>
<th>Total</th>
</tr>
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<td>1.30</td>
<td>1.30</td>
<td>1.30</td>
</tr>
<tr>
<td>Single Run Max value (20 cases)</td>
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<td>1.43</td>
<td>1.5</td>
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<td>DAF Max Value</td>
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<td>Possibility (over 1.3 case)</td>
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Figure 1. Sensor layout of the bridge on Highway US-78

Figure 2. Field B-WIM testing

(a) Highway US-78, (b) Field B-WIM Testing
Figure 3. Mathematical simulation model for the bridge on highway US-78

Figure 4. Boundary conditions of the US-78 bridge
Figure 5. Displacements of semi-rigid frame element

Figure 6. Expansion boundary conditions of the bridge on US-78
Figure 7. Simulation models for line group run

(a) Line group at lane 1, (2) Line group at lane 2, (3) Line group at both lanes
Figure 8. Bridge moment comparisons
Figure 9. Summary of DAF for B-WIM testing
Figure 10. DAF for line group simulation at Lane 1
(a) at girder 1; (b) at girder 2; (c) at girder 3; (d) at girder 4; (e) whole bridge
Figure 11. DAF for line group simulation at Lane 2
(a) at girder 1; (b) at girder 2; (c) at girder 3; (d) at girder 4; (e) whole bridge
Figure 12. DAF for line group simulation at both lanes

(a) at girder 1; (b) at girder 2; (c) at girder 3; (d) at girder 4; (e) whole bridge
CONCLUSIONS AND SUMMARY

For bridge weigh-in-motion (B-WIM) applications, new technologies and methodologies for the analysis of the data gathered are developed. This study presents applications for slab-on-girder bridges based on B-WIM exponential data. The proposed simulation model and developed algorithm have the potential to improve the current B-WIM system for evaluating bridge integrity and safety.

1. For the FAD B-WIM system, the combined application of slab bending sensors and girder compression sensors is the best selection to detect vehicle information for beam-slab bridges, and the girder shear sensor is a good back-up choice in field tests. The effective coverage of each type of sensor is demonstrated with the identified peak for axle detection, and an optimized location of the axle detector is recommended. For the application of multiple sensors, a priority list is suggested for selection of sensors and for determination of vehicle axles in the B-WIM system.

2. A simulation model for the effects of B-WIM testing is developed and verified based on the site-specific B-WIM data. Various factors, such as the vehicle suspension system, multi-span bridges, semi-rigid connections, road roughness, and boundary conditions were simulated in the model, which demonstrated that semi-rigid rotational joints, the road profile, and foundation displacement significantly influence the dynamic behavior of a bridge, and that time delay has less influence for the moment variation. Moreover, the proposed model
demonstrates the specific multi-span bridge vibration is similar to the frame structure and provides a good understanding about bridge moment effects.

3. Based on the field B-WIM data, two alternative algorithms were developed for vehicle weight measurements. Algorithm #2 has improved accuracy for groups of axles and for GVWs, and algorithm #3 has accurate predictions for all single-axle weights, groups of axles, and GVWs. For algorithm #2, the accuracy class improves from E(60) to D+(20) for group axle weight, and from D+(20) to B(10) for GVW. For algorithm #3, the accuracy class improves from E(55) to A(5) for single axles, from E(60) to B(10) for group axles, and from D+(20) to A(5) for GVW. Algorithms #2 and #3 demonstrate significant improvements in the accuracy for vehicle weight measurement.

4. With little knowledge of a five-axle semi-trailer truck braking in an emergency on a highway bridge, a simulation model based on the B-WIM experimental information of bridge US-78 was developed to simulate the bridge behavior when a truck emergency brakes on the bridge at high speed. The simulation demonstrates that the bridge has a potential safety issue about moment capacity under this condition. A prediction of future risk was made, and possible control options were recommended to improve the bridge performance and to reduce the possible over-reaction of the bridge.

5. A simulation approach was developed to predict site-specific DAF values for group vehicles. The DAF can be predicted for this bridge based on the field B-WIM data. This approach predicts that as high as 79% of simulation cases at traffic lane 1 are beyond the AASHTO requirements (DAF=1.3), and also that the
DAF is as high as 1.6 in the inner girders. There are lower DAF values at the side girders, however, and these meet the code requirements. Based on the results, this newly developed simulation can be applied to predict the site-specific DAFs of existing bridges.

Finally, from this study, a simulation methodology integrating B-WIM data to evaluate structural safety was investigated and the overall goals to improving the reliability and effectiveness of the B-WIM were achieved. The potential contributions are listing as follows.

1. We developed a vehicle axle detection strategy which could improve the performance and reliability of B-WIM system. This method will benefit all current commercial B-WIM systems.

2. We developed a simulation model, which was verified by field experiments (B-WIM), to obtain true flexural moments. Also, based on this simulation model, two alternative algorithms were developed for vehicle weight identification with acceptable accuracy (<15%). These two algorithms have significant improvement for vehicle weight identification. Using these algorithms, the confidence level (95%) of B-WIM system will get promised.

3. First time ever, based on B-WIM experimental data (rather than estimation based on visual inspection with conservative assumptions), the simulation methods were developed to evaluate bridge safety and estimate the critical DAFs for existing bridge. The above developed approaches and methods have the potential to be used by state DOTs.
Future Research

Throughout this study, a bridge simulation model and some applications were explored based on B-WIM experimental data. It is the author’s belief that B-WIM system has a large potential for development and application for bridge infrastructure. Future research will include the following areas.

1. More bridges need to be tested by the proposed simulation model and algorithms for safety during extreme events, and the site-specific DAF should be determined for existing bridges.

2. Further research of the proposed algorithm will be conducted to broaden its use, including its application to bridges with rough pavement or approach, its application to wide bridges with 3 or 4 lanes, and its application to flexible bridges.

3. The next step of research is to investigate application of the B-WIM system together with the proposed algorithm to other types, such as steel bridges, continuous support bridges, and pre-stressed concrete bridges.

4. Field testing with multiple trucks under different load combinations is needed to verify the proposed algorithm in the application of identifying axle loads of multiple heavy vehicles.

5. Recently, B-WIM using moving force identification theory has received substantial attention. Future research should be to develop accurate and easily implementable B-WIM for a wider range of bridges.
GENERAL LIST OF REFERENCES


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