Rapid Bridge Condition Screening by Falling Weight Deflectometer

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ABSTRACT

There are several full-scale testing methods that can be used to characterize and evaluate the global performance and condition of bridges. These global methods mainly consist of static load tests and dynamic testing methods using either controlled or uncontrolled dynamic excitation. Each approach has advantages and disadvantages with respect to experimental and logistical considerations, data analysis requirements, and the scope and utility of the characterization results obtained. This report presents a global dynamic characterization program based on controlled impact dynamic testing that was applied to a truss bridge. The impact testing was performed using a hand-held impact hammer and a falling weight deflectometer (FWD) as dynamic excitation sources. The objective of the project was to evaluate if the FWD, which can produce a broadband dynamic force, can be effectively used as a tool for quantitatively characterizing the performance and condition of bridges. Many transportation agencies already use FWD devices for their pavement evaluation programs and it follows that if the device is suitable for impact dynamic testing of bridges, these agencies could also use their FWDs to quantitatively evaluate their bridges. The report discusses different dynamic testing approaches and presents an impact dynamic testing program executed for the truss bridge. The results obtained using the two dynamic excitation devices are presented and compared with each other, and with the results from an analytical model of the bridge. Finally, several observations and conclusions related to the efficacy of FWD devices for impact dynamic testing of bridges are presented and discussed.

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INTRODUCTION

Current bridge evaluation and management approaches rely substantially on visual inspection data that is considered qualitative in nature. Quantitative evaluation methods, though less frequently employed in practice, have the potential to significantly improve the effectiveness and reliability of bridge operational and maintenance management decisions since these methods provide objective and measureable descriptions of in-situ performance and condition. The available methods for evaluating bridges are described in The Manual for Bridge Evaluation published by AASHTO (*1*). The quantitative characterization methods described in the manual include various nondestructive testing/evaluation techniques that provide localized descriptions of condition, and global methods including load testing and dynamic (vibration) testing, which can characterize a structure as a complete system. The manual provides a very brief introduction to vibration testing methods. Many researchers continue to develop and refine the available dynamic testing methods to more reliably characterize and evaluate the global performance and condition of bridges, to calibrate analytical models so that they more closely simulate the actual structure, and to detect and quantify damage or deterioration. The widespread interest in dynamic testing methods stems from a number of advantages they have in terms of logistical and experimental considerations and because of the broader utility of the experimental results relative to other quantitative characterization methods.

Dynamic testing methods can be broadly classified into two distinct categories: (1) operational modal analysis (OMA) and (2) experimental modal analysis (EMA). The main distinction between the two categories is whether or not the dynamic excitation is uncontrolled or controlled. No matter which specific type of dynamic testing is used to characterize a bridge, the structure being evaluated is assumed to be linear and time invariant. In OMA, only the measured vibration responses are used to estimate the modal frequencies, modal vectors and damping ratios. The uncontrolled and unmeasured excitation in OMA is provided by operating loads and ambient environmental sources. The excitation is assumed to be stationary, broadband white noise that will excite any structural modes located within the frequency band of interest. Because of the stringent assumptions that are required related to the dynamic excitation and the inability to measure or control this excitation, OMA testing can be subject to more uncertainty than EMA testing.

In EMA, controlled and measureable excitation is created using various excitation devices, and both the measured excitation (input) and vibration responses (outputs) are used to identify the modal characteristics. Typical excitation devices include: (1) impact devices such as drop weights or impact hammers that produce broadband, impulsive dynamic forces, (2) eccentric mass shakers that produce a harmonic force at discrete frequencies or that is swept across a range of frequencies, and (3) linear mass shakers which can produce random or deterministic dynamic excitations of any type. Smaller-scale excitation devices can be suitable for EMA testing of short to medium span bridges.

The modal characteristics are identified from an EMA test using a variety of available algorithms in the time or frequency domains. The capability to characterize and control the dynamic excitation represents a significant advantage over OMA and can lead to reduced testing times and less uncertainty in the results. Also, since both the inputs and the corresponding outputs are measured, modal scaling factors can be determined from the measurements. Modal scaling factors are used to scale the identified modal vectors and are essential for evaluating the modal flexibility of a structure. Modal scaling factors cannot be obtained directly from an OMA test, although some alternative approaches, such as adding large masses to different locations on the structure and re-testing, have been proposed to identify them.

Objective and Scope

The primary objective of this study was to evaluate the efficacy of using a falling weight deflectometer (FWD) as a controlled excitation device for impact dynamic testing of short to medium span bridges. Impact dynamic testing is one of several available experimental modal analysis approaches that can be used to provide a quantitative global characterization of the structure in terms of its modal characteristics (natural frequencies, modal vectors, damping, and modal scaling). The flexibility matrix for a bridge can also be developed using the same modal characteristics. Flexibility derived in this manner is referred to as modal flexibility.

The individual elements of the modal flexibility matrix for a structure can be expressed in terms of its modal characteristics as outlined by Zhang and Aktan (*2*) using Equation 1.

$$f_{ij} = \sum_{r=1}^{N} \frac{\phi^r(i)\phi^r(j)}{\omega_r^2} \tag{1}$$

where

f_{ij} = modal flexibility at location i due to a unit load at location j,

$\phi^r(i)\phi^r(j)$ = product of the modal vector coefficients for measurement locations *i* and *j* of the mass-unit-normalized modal vector for mode *r*,

ω_r^2 = square of the natural frequency (rad/sec) for mode *r*, and

N = total number of modes identified from the measurements.

Most structures have distributed mass and stiffness properties, and their dynamics and modal flexibilities are completely described by the summation of an infinite number of modes. From a practical perspective, EMA identifies only a finite number of modes. This leads to truncation errors in the modal flexibility determined by Equation 1, and modal flexibility must be considered to be an approximation to a structure's static flexibility. Fortunately, Equation 1 is clearly dominated by the lower order structural modes since the denominator term is the natural frequency squared, and these modes can usually be identified by EMA. It has also been demonstrated in a number of earlier studies that the modal flexibility estimated from the lower order modes of a bridge can yield a very close approximation to the actual static flexibility (*3*, *4*). Therefore, modal flexibility represents a quantitative measure of a structure's static flexibility, and it automatically includes the effects of any known or unknown defects, deterioration and damage on a bridge's global performance. This is a significantly more meaningful description of the in-situ bridge than what can be obtained from qualitative condition indices.

The FWD was specifically evaluated as an excitation device for dynamic testing of bridges since many departments of transportation already employ these devices for their pavement evaluation programs, and they already have access to and substantial experience in operating them. In addition to the economic advantages associated with extending the use of this device for evaluating bridge performance and condition, FWD devices are very portable and can produce larger and more repeatable impulsive loads than is possible from hand-held impact hammers that are commonly used for impact dynamic testing of civil structures.

A review of the literature reveals limited examples of using a FWD for assessing bridges. Hoadly and Gomez (*5*) describe the use of a FWD to determine the stiffness for both a prestressed concrete bridge and a steel girder bridge. They used the FWD and its onboard geophones in a static testing sense to determine the point the stiffness on the bridges by recording average load and displacement from several impacts, and dividing the load by the displacement. The authors recommended further tests for calibration with their approach. Weidner et al. (*6*) describe different modifications that can be made to improve the efficacy of using FWD for impact dynamic testing of bridges. They note that the FWD's onboard sensors (force and velocity) and data acquisition are not directly suitable for dynamic testing of bridges due to their pre-configuration for pavement testing applications. The authors describe their attempts to modify the impact characteristics of the FWD to reduce the multiple impacts (rebound) of the drop mass. They reported some success in this regard by replacing the stock rubber material on the FWD strike plate with a harder plastic material. Dynamic testing was performed on four bridges in West Virginia using a FWD, a hand-held impact hammer, and a custom developed drop hammer. They note in the paper that their data analysis from these tests was underway.

In this study, the FWD was evaluated solely as a potential excitation device for impact dynamic testing. The onboard FWD sensors and data acquisition were not used as part of the bridge evaluation. The utility of the FWD was evaluated by performing a multiple-reference impact test (MRIT) of a simply-supported truss bridge. MRIT testing was also performed using hand-held impact hammer to provide a baseline for comparison. The modal characteristics were identified using the input-output measurements from each test case using the same frequency domain modal parameter identification approach. Modal flexibility matrices were computed using the modal characteristics identified from each test, and these were compared with the static flexibility extracted for the measurement locations from a finite element model of the bridge. The test bridge, experimental program, data analysis procedures and comparisons of the results are presented and discussed in following sections of the report.

BRIDGE DESCRIPTION

The bridge tested for this study is a Parker pony truss bridge located in Fayetteville, AR that was constructed in 1930. The bridge crossing consists of three 100 feet-long, simply-supported truss spans and carries two lanes of traffic across the West Fork of the White River. The superstructure details of each truss span are identical and each truss span consists of ten panels (Figure 1). The width of the bridge measured from center-to-center of the trusses is 22.3 feet. The depth of the trusses varies along the span length and has a maximum value of 14 feet at the midpoint of the span. The 8-inch thick reinforced concrete deck is supported directly by I-shaped rolled steel floorbeams that span between the bottom chords of the upstream and downstream trusses. The truss members consist of rolled and riveted built-up sections as follows: (1) top chords: two channels, a top cover plate, and lacing; (2) bottom chords: two channels with batten plates; (3) verticals: I-beams; and (4) diagonals: I-beams and two angles with batten plates. No plans could be located for the bridge and field measurements were used to determine the geometric characteristics of the truss and its members. The interior span of the bridge is supported on concrete piers and the end spans are supported by the interior piers and concrete abutments. The foundation characteristics are unknown. The dynamic testing program described in this report was applied to the middle span of the bridge crossing.

EXPERIMENT DESIGN AND EXECUTION

Excitation Devices

The MRIT approach was used to characterize the dynamic properties of the bridge. Impact dynamic testing was accomplished using both a falling weight deflectometer (FWD) and a hand-held impact hammer (instrumented sledge). When the FWD is used in pavement testing, an 11-inch diameter strike plate with a steel rod connected to it is first lowered onto the pavement surface. The steel rod extends into the FWD trailer and a predetermined number of weights are dropped onto the rod producing a short duration impact force. A Dynatest FWD was used for this study (Figure 2) and can generate force outputs ranging from 1,500 to 27,000 lbf. A nominal 12,000 lbf impact force was used for the bridge testing.

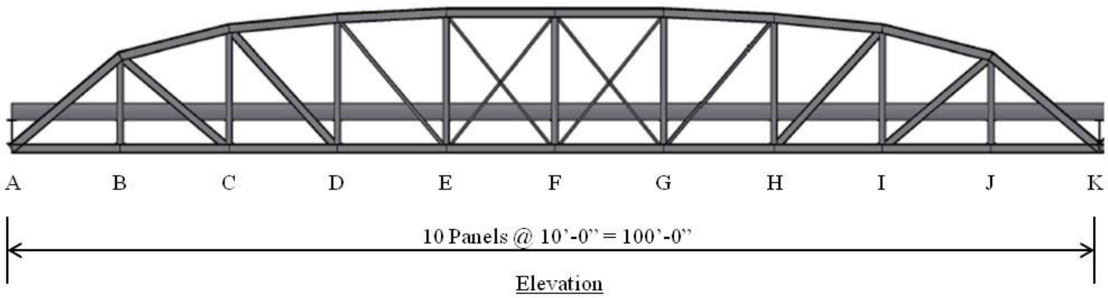
In the MRIT approach, the impact force should ideally consist of only a single pulse. The FWD device was observed to produce multiple pulses in the force measurement due to rebound of the drop weights. The authors constructed a low-cost absorber plate that was placed between the bridge deck and the FWD strike plate in an attempt to prevent or minimize the rebound of the drop masses and mitigate the multiple force pulses. The absorber plate used elastomer bumpers obtained from EFDYN, Inc. that were placed at the four corners between a pair of 12 inch x 12 inch aluminum plates. A dynamic load cell was placed in the center location between the two plates (Figure 2) to measure the impact force produced by the FWD.

A Model 086D50 impact hammer (Figure 2) from PCB Piezotronics was also used as an excitation device. This is a hand-held sledgehammer (12 lbf) with an integrated dynamic load cell that has a sensitivity of 1 mV/lbf and peak measurement range of ±5000 lbf. The contact time between hammer and the structure determines the frequency range of the impulsive load produced. A shorter contact time produces a wider excitation band in the frequency domain. Hammer tips of various stiffnesses can be used to adjust the contact time and force magnitude produced by each hammer strike. A soft tip was used on the impact hammer in this study. The device can be labor intensive to use for dynamic testing civil structures.

Instrumentation and Data Acquisition

A total of 25 accelerometers were used to measure the bridge vibrations during the impact testing. The accelerometers included Model 393B05 and Model 393C accelerometers from PCB Piezotronics. The 393B05 accelerometers have a nominal sensitivity of 10 V/g and a peak measurement range of 0.5 g. The 393B05 accelerometers were located where the vibration amplitudes were smallest. The 393C accelerometers have a nominal sensitivity of 1 V/g and a peak measurement range of 2.5 g. The accelerometer locations on the bridge structure are shown schematically in Figure 3. The accelerometers were attached to the bottom flange of the floorbeams and to the truss joints using magnets.

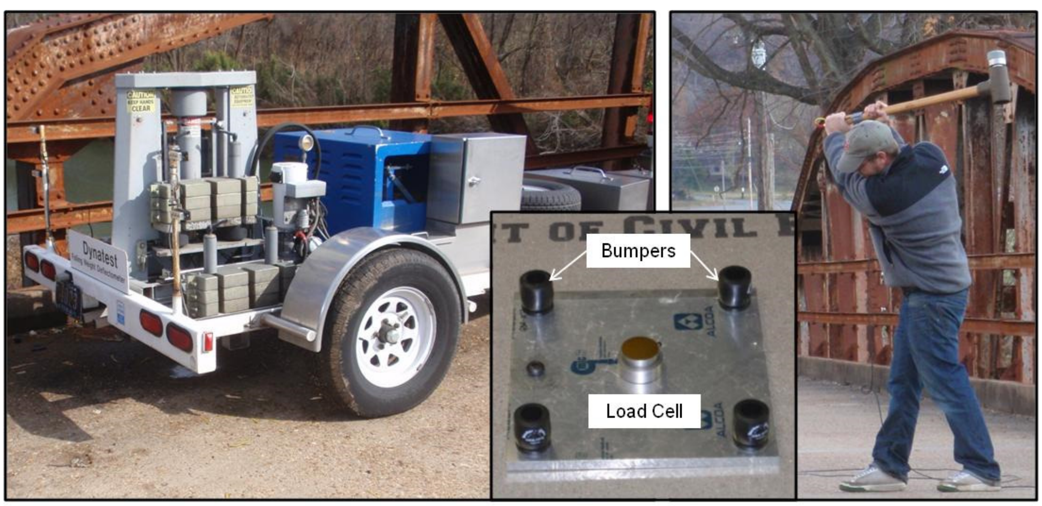
The accelerations and impact forces were recorded with a National Instruments PXI mainframe containing dynamic data acquisition input modules. The modules have 24-bit resolution and 110 dB of dynamic range. The measurement data were sampled at 1024 Hz for 16 seconds for each impact produced by the FWD and the hand-held impact hammer.



(a)

(b)

FIGURE 1 (a) Elevation view and (b) photograph of the Parker pony truss bridge.



(a) (b) (c)

FIGURE 2 (a) Falling weight deflectometer (FWD), (b) bottom portion of absorber plate and (c) hand-held impact hammer.

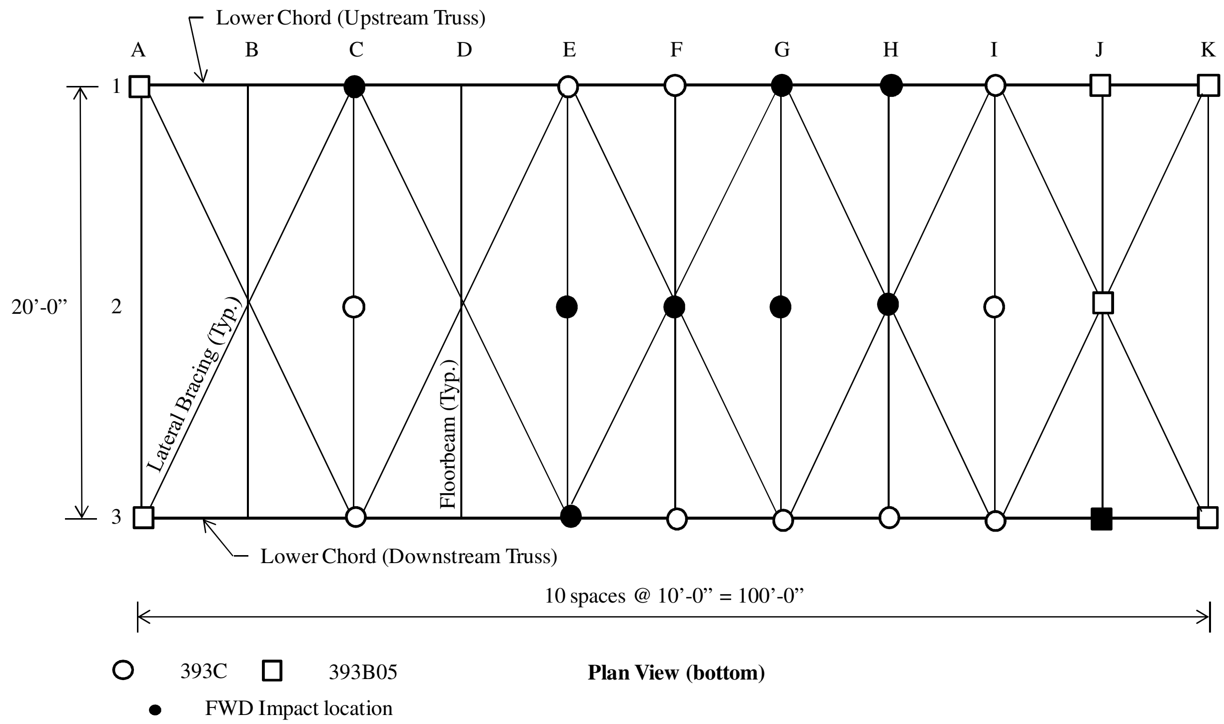


FIGURE 3 Accelerometer and FWD impact locations for the bridge.

Impact Testing

The FWD was used to impact nine different locations on the bridge deck as shown in Figure 3. Three input-output data records were recorded at each impact location and averaged. The width of the FWD trailer prevented the impacts produced along the edge of the bridge from being located directly above the accelerometers, and this adds some uncertainty to the modal scaling estimates. A nominal impact force of 12,000 lb was selected by the FWD operator, and the actual impact force was recorded by a load cell placed between the strike plate and the deck surface. The FWD is capable of producing approximately one hit per minute, and several minutes were required to reposition the device at the different locations on the bridge. Three input-output data records were also recorded at all of the accelerometer locations on the bridge with the hand-held impact hammer. The bridge was closed to traffic during the impact tests.

DATA ANALYSIS

The data processing procedure consisted of the following steps: (1) construct frequency response functions from the measured input-output records, (2) identify the natural frequencies, modal vectors, modal damping and modal scaling, and (3) formulate the modal flexibility matrix from the identified modal characteristics. All data processing was performed in MATLAB starting with the raw time measurements collected from both the FWD and hammer impact tests.

The measured forces and accelerations were subsampled to 512 Hz from the original 1024 Hz records. The time domain data were transformed to the frequency domain by fast Fourier transform and averaged. The H_1 algorithm (*7*) was used to construct frequency response functions (FRFs) from the measurements. The rows of the FRF correspond to the output locations and the columns correspond to the different input locations. The acceleration records were synthetically integrated in the frequency domain to produce FRFs in terms of displacement divided by force.

The complex mode indicator function (CMIF) and enhanced FRF approach was used to identify the modal characteristics from the FRFs (*8*). CMIF uses singular value decomposition (SVD) to estimate the contribution of assumed modal vectors to each frequency line of the FRF matrix (Equation 2). The singular values are proportional to the strength of the response caused by each mode while the left singular vectors are estimates of the mode shapes corresponding to each singular value. Performing the decomposition on only the imaginary part of the FRF returns real valued singular vectors.

$$[H(\omega)] = [U(\omega)][\Sigma(\omega)][V(\omega)]^H \tag{2}$$

where

$[H(\omega)]$ = frequency response function,

$[U(\omega)]$ = left singular vector matrix,

$[V(\omega)]$ = right singular vector matrix,

$[\Sigma(\omega)]$ = a diagonal matrix of singular values sorted in ascending order, and

$\bullet^H$ = Hermitian transpose.

Natural frequencies are selected from peaks in a plot of the singular values (CMIF plot) and the modal vectors are estimated as the left singular vectors corresponding to the selected peaks. Enhanced frequency response functions (eFRFs) were then formulated according to the methods outlined by Allemang and Brown (*8*) and Catbas et al. (*9*). The eFRF procedure decomposes the multiple degree of freedom FRFs into a system of equivalent single degree of freedom FRFs using the left singular vectors as modal filters. The modal frequencies, damping ratios and modal scaling are estimated by curve fitting methods applied to eFRFs.

Modal flexibility can also be expressed in terms of a FRF. This can be illustrated by considering the differential equations of motion for an MDOF system as shown in Equation 3.

$$[M]\{\ddot{x}(t)\}+[C]\{\dot{x}(t)\}+[K]\{x(t)\}=\{F(t)\} \tag{3}$$

where

$[M]$ = system mass matrix,

$[C]$ = system damping matrix,

$[K]$ = stiffness matrix,

$\{F(t)\}$ = external force vector, and

$\{\ddot{x}(t)\}$, $\{\dot{x}(t)\}$, and $\{x(t)\}$ are acceleration, velocity and displacement vectors, respectively.

A system of algebraic equations is obtained by taking the Laplace transform of Equation 3, and assuming zero initial conditions. The transfer function of the system in Laplace (s or complex frequency) domain is the system outputs divided by the system inputs as shown in Equation 4.

$$[H(s)]=\frac{\{X(s)\}}{\{F(s)\}}=\frac{1}{s^2[M]+s[C]+[K]} \tag{4}$$

where [H(s)] = the transfer function of the system. If the transfer function matrix is evaluated along the frequency axis (s = $j\omega$) with $j = \sqrt{-1}$, the FRF matrix can be obtained as shown in Equation 5.

$$[H(\omega)]=\frac{1}{-\omega^2[M]+j\omega[C]+[K]} \tag{5}$$

If the frequency response function given in Equation 5 is evaluated at $\omega = 0$, the flexibility matrix, $[K]^{-1}$, of the structure is obtained. The frequency response function can also be synthesized from the identified modal characteristics based on modal expansion theory (*7*) as shown in Equation 6. Evaluating this equation for $\omega = 0$ gives the modal flexibility matrix for the system.

$$[H(\omega)]=\sum_{r=1}^{N}\frac{\{\psi\}_r\{\psi\}_r^T}{M_{A_r}\left(j\omega-\lambda_r\right)}+\frac{\{\psi\}_r^*\{\psi\}_r^{*T}}{M_{A_r}\left(j\omega-\lambda_r^*\right)} \tag{6}$$

where

$\{\psi\}_r$ = modal vector for mode r,

λ_r = system pole at mode r, $\lambda_r = \sigma_r + j\omega_r$,

σ_r = damping coefficient for mode r,

ω_r = damped natural frequency (rad/s) for mode r,

M_{A_r} = modal scaling factor for mode r of a system with general damping,

j = $\sqrt{-1}$,

ω = frequency associated with each spectral line (rad/sec),

N = number of modes included in the formulation,

$\bullet^*$ = complex conjugate, and

$\bullet^T$ = transpose operator.

FINITE ELEMENT MODEL

A 3D finite element (FE) model of a single truss span was developed in SAP2000 to provide an additional baseline for comparing the dynamic test results. The model is based on nominal properties and dimensions of the bridge and its members, and it was not calibrated to match the experimental data given the significant uncertainty associated with many details of the structure. Different extremes for the boundaries (supports) were considered with the model to illustrate their effects on the natural frequencies and the analytically derived flexibilities. Other sources of uncertainty associated with the FE model included: (1) properties of all members had to be established by field measurements due to a lack of original plans, (2) the material properties of the concrete and steel elements were unknown, and (3) the foundations and substructure details were unknown. The truss members were represented by 3D frame elements with rigid joints to reflect the large riveted gusset plates at each connection on the actual structure. The concrete deck was modeled using shell elements. The top flanges of the transverse floor beams are embedded in the deck and their ends are stoutly connected to the truss verticals and bottom chord. The floorbeam flanges were modeled with frame elements and the web was modeled using shell elements to more realistically simulate the actual connection geometry with the truss members. The bridge supports were modeled as pin-roller, pin-pin and fixed-fixed in separate analysis cases. The modulus of elasticity for the steel and concrete were assumed to be of 29,000 ksi and 3,600 ksi, respectively.

DISCUSSION

The objective of the project was to evaluate the utility of performing experimental modal analysis of bridges using a FWD as an excitation device. To make this evaluation, the modal characteristics of the bridge identified from the FWD and hammer impact tests were compared. Finite element model output is also considered. The modal characteristics that are compared in this report include the modal frequencies, modal vectors and modal flexibility.

The impact hammer test results are used as the primary baseline for the comparison since this device is known to provide accurate results. The impulse forces produced by the hammer had relatively clean, sharp spikes with little evidence of rebound or noise in the measurements.

In contrast, the FWD impulses were influenced by the rebound of the drop weights, and a double impact was observed in most of the force measurements. The double impact can lead to drop-offs in the frequency spectrum of the force which can produce errors in the results. FWD has a major advantage in that it could produce a 12,000 lbf magnitude impact compared to the 2,500 lbf that was produced by the hand-held impact hammer. Figure 4 shows typical time records of the impact forces produced by the FWD and impact hammer. Several different measures were tried to reduce or eliminate the double impact observed with the FWD during the field testing. These included adjusting the impact force magnitude, placing an absorber plate between the bridge deck and FWD strike plate, and placing clay between the deck and FWD strike plate. The stiffness and number of bumpers used with the absorber plate was also varied during the testing. None of these approaches proved very successful in preventing the double impacts from occurring with the FWD, and in some instances reduced the impact force recorded by the dynamic load cell. The peak force measured by the load cell was scaled to the force measured by the FWD sensor in these instances.

Figure 5 shows the frequency spectra for typical FWD and hammer impact measurements. As expected, the sharp, clean impulse produced by the hand-held hammer leads to a fairly uniform force spectrum over a wide frequency band. The larger force produced by the FWD impact introduces ten times more energy into the system at low frequencies; however, the double impact creates a significant drop in the force spectrum at certain frequencies. The drop consistently occurred at about 54 Hz for this bridge, but the magnitude of the drop varied from impact to impact. The total force output from the FWD also varied by up to 10% from one impact to another.

The natural frequencies for both testing methods were found using the CMIF/eFRF approach discussed previously. The peaks in the CMIF plots constructed from the FWD and hammer test data were almost exactly aligned. Of the nine modes listed in Table 1, six occurred at identical frequency lines for both tests. The other three modes vary from each other by less than 1% in measurements with a frequency resolution of 0.06 Hz. It should be noted that the hammer testing was performed immediately following the FWD testing to minimize the potential for changes in the frequency due to a changing environment/structure. The mass of the relatively light FWD trailer and its tow vehicle (a pickup truck) did not noticeably alter the natural frequencies identified for the bridge. The measured frequencies do diverge from those found from the un-calibrated FE model, especially for the lower bending modes of the truss. The difference is likely driven by a combination of uncertainties reflected in the model and is not just related to the idealization of the bearings.

The left singular vectors produced from singular value decomposition of the measured frequency response function matrix are used as estimates of the modal vectors and are considered to be close approximations to the real mode shapes (*8*). The modal vectors identified from the FWD and hammer tests were compared with each other using the modal assurance criterion (MAC), which indicates the level of correlation between two modal vectors (*10*). A MAC value of 1.0 indicates that two modal vectors are identical and a value of zero indicates that two modal vectors are orthogonal. Figure 6 shows the MAC values computed by comparing the modal vectors from the FWD and impact hammer tests. The figure illustrates a high degree of correlation between these modal vectors. Although not shown here due to space limitations, the mode shapes from the FE model and FWD test also showed very good agreement. The high correlation between mode shapes also indicates that the added mass from the FWD trailer and its towing vehicle did not have a significant influence on the dynamic characteristics of the bridge.

Modal flexibility was computed by Equation 6 for the modal characteristics found from both impact tests. A total of nine modes were included in the modal flexibility formulation. The modal flexibility matrix was created from 25 output locations on the bridge and has 625 elements. A unit load vector was applied to the modal flexibility matrix (*4*) to generate vertical displacement curves for the structure. The same displacement curves were also extracted from the FE models by applying unit loads to the model nodes that corresponded to the accelerometer locations. A separate displacement curve was computed for the three longitudinal sensor lines on the bridge (Figure 7). The displacement curves indicate a reasonably good agreement between the modal flexibility from the two different impact tests and the FE model with the pin-roller supports. The FWD results indicate on average about 8% less flexibility than was found by hammer testing. The deflection profiles extracted from the FE model (static flexibility) with pin-roller supports were approximately midway between the two experimental test results. The static flexibilities from the FE models with pin-pin and fixed-fixed supports indicate a much greater stiffness than any of the experimental results. The results also show that the first bending mode had the most influence on the modal flexibility of the truss span. Including all of the identified modes in the modal flexibility computation altered the result by less than 5% for both test methods.

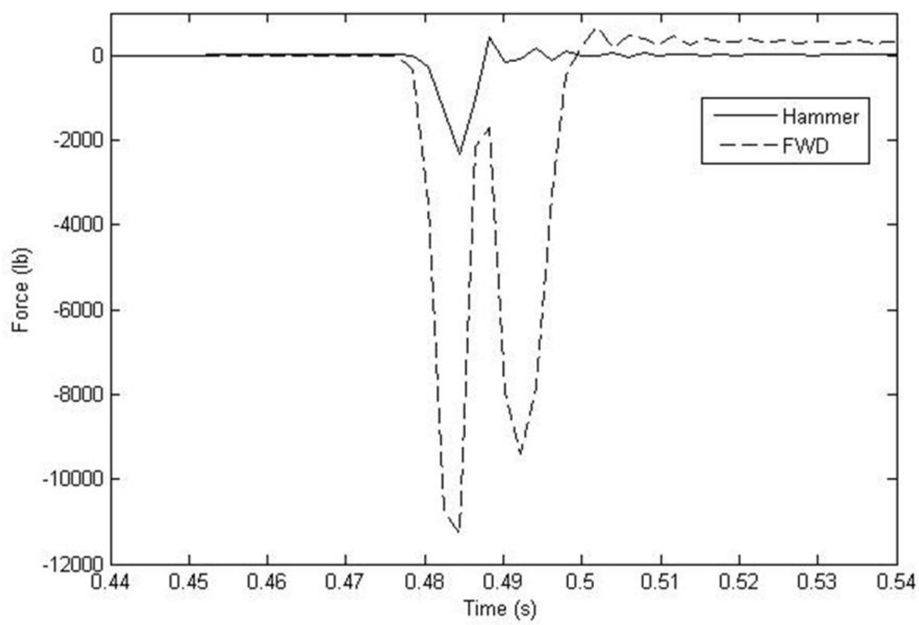


FIGURE 4 Time records of typical FWD and hammer impact forces.

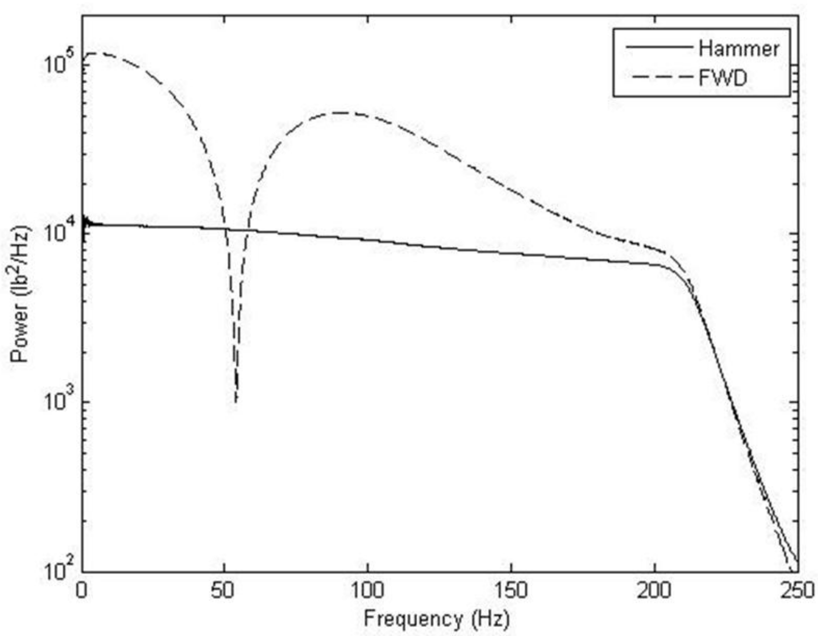


FIGURE 5 Frequency spectra for typical FWD and hammer impact forces.

TABLE 1 Undamped Natural Frequencies (Hz) from Impact Tests and FE Model Cases

Mode	Description	FWD	Hammer	FE Pin-Roller	FE Pin-Pin	FE Fix-Fix
1	Bending 1	4.13	4.13	3.73	4.84	4.85
2	Torsion 1	6.81	6.81	6.83	7.33	7.34
3	Bending 2	9.63	9.63	8.40	8.65	9.03
4	Bending 3	13.19	13.19	14.24	13.87	13.90
5	Torsion 2	17.19	17.13	17.38	17.75	17.86
6	Bending 4	17.63	17.63	17.78	18.87	19.20
7	Bending 5	20.06	20.13	20.25	20.33	20.36
8	Torsion 3	23.94	23.94	24.22	24.76	24.90
9	Torsion 4	32.50	32.81	32.71	32.93	32.90

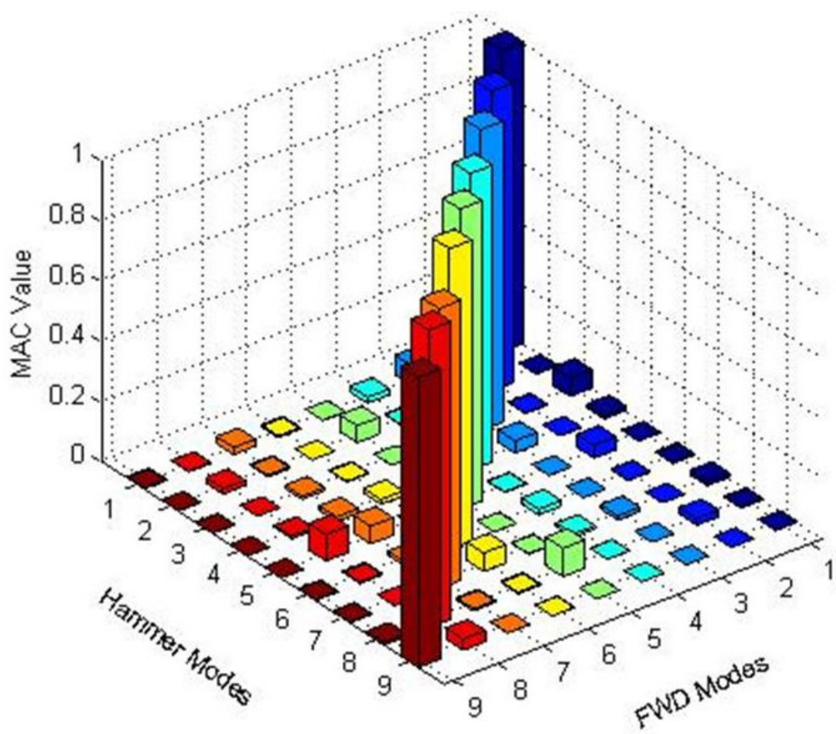


FIGURE 6 Modal assurance criterion (MAC) values computed for the identified modal vectors.

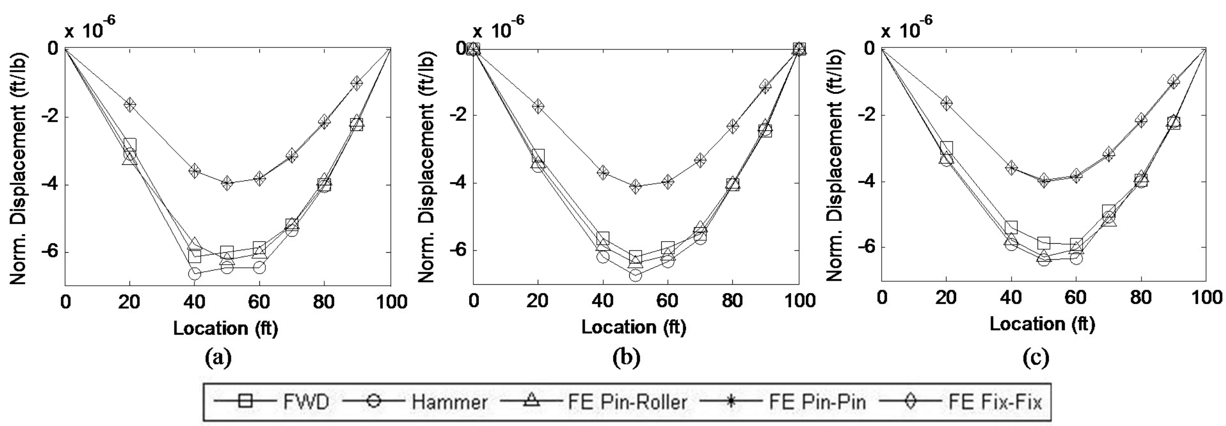


FIGURE 7 Deflection profiles from modal flexibility matrices and FE model (a) upstream truss, (b) longitudinal centerline of bridge, and (c) downstream truss.

CONCLUSIONS AND FUTURE WORK

The experimental characterization results indicated that the FWD can be an effective device for impact dynamic testing of highway bridges, even though the impulse forces produced by the device did not completely conform to ideal impact force characteristics. When compared to typical hammer testing, the FWD enabled identification of practically identical natural frequencies and mode shapes. Also, the modal flexibility extracted showed good agreement with the modal flexibility extracted from the hammer test and the static flexibility from the FE model.

The FWD has some positive qualities that make it attractive for impact dynamic testing of bridges. First, many departments of transportation already have these devices. Second, the FWD produces a significantly larger excitation forces than what can be produced by a standard hand-held impact hammer. The larger force leads to better signal-to-noise ratios in the measurements and represents a closer approximation to the service loads on the bridge. Finally, the FWD provides a fairly consistent impact force from hit to hit and from day to day. This is more difficult and labor intensive to accomplish using a hand-held impact hammer.

The FWD also has some shortcomings as an impact testing device. Most importantly, it produces a double impact that is difficult to mitigate. The device is designed for a use where the double impact is not of great importance, and there does not seem to be a very simple method for eliminating the second hit. It is noted that for longer bridges with lower frequencies, the double impact is of less importance since the effect in the frequency domain is minor below 20 Hz. Although not presented in this report, the damping ratios identified for some of the modes were also affected by the character of the FWD impact force. The FWD also has accessibility issues and traffic control requirements that are not necessarily applicable to hammer testing. The use of the FWD requires a lane closure at a minimum, and keeping traffic off of the bridge during the measurements is obviously preferred. The width of the trailer also prevents the FWD from being used very close to curbs, barriers, etc. Thus, some additional planning may be required to co-locate impact and response measurement locations. Finally, impact testing with the FWD was generally slower than with the hand-held impact hammer. The FWD required more time to reset between the hits at one location, and required considerably more time to relocate to the next impact location. The slower testing speed and the benefits of larger and more repeatable impact forces need to be considered when using an FWD for impact dynamic testing of in-service bridges.

In general, the FWD was found to be an effective excitation device for impact dynamic testing of bridges and did permit same modal characteristics of the bridge provided by other EMA methods to be identified. The main benefits of this device include the ability to create large impact forces and the consistency of the generated forces. The FWD device could definitely be useful for screening bridges to detect significant changes in their performance or condition as reflected by their global stiffness. Additional research and enhancements to the experimental approach using the FWD (to mitigate the double impacts) would be required if the objective is to reliably detect and quantify the locations of damage or deterioration.

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