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A New Methodology for Nondestructive Evaluation and Rating of Bridges

Timothy M. Whalen, Javier F. Gauthier, and Judy Liu

Abstract— **The growing number of highway bridges in poor condition requires the development of effective tools for inspecting and evaluating bridges. To address the limitations of current assessment practices, a new nondestructive evaluation methodology, incorporating global and local evaluation techniques to obtain a quantitative assessment of the structural condition, is proposed. In this study, a simple global damage identification and assessment method, called the differential damage factor (DDF) method, is presented. The results from preliminary analyses of the proposed DDF method performed on numerical and experimental models are discussed.**

I. Introduction

HIGHWAY bridges are susceptible to structural damage over their service lives due to factors such as excessive operating loads, fatigue, and corrosion. Structural inspection and assessment programs have been developed in order to detect damage in its early stages and thus avoid a catastrophic situation. Since the implementation of National Bridge Inspection Standards (NBIS), these programs have relied largely on visual inspection to provide critical information on the condition of bridges in the United States. It is expected that visual inspections should provide sufficient information about the bridge inventory to permit effective maintenance and repair planning. However, the subjective nature of current inspection practices does not allow for thorough assessment of the functional condition of the bridge. The visual inspection technique has a limited capability to detect damage, especially when damage is not visible and the portion of the structure being inspected is not readily accessible [1], [2]. Additionally, damage could go undetected at inspection, or cracks could grow to critical levels between inspection intervals [3]. This approach to bridge inspection is thus deemed insufficient for effective bridge management. Therefore, more detailed and quantitative information is required to effectively determine the condition of the bridge inventory and ultimately establish an effective health monitoring plan.

To address the limitations of current assessment practices, a new nondestructive evaluation (NDE) methodology based on the use of global and local evaluation techniques to obtain a quantitative assessment of the bridge's structural condition is proposed. Experimental information obtained by NDE techniques, in conjunction with visual inspection, will increase the reliability of the results of the condition assessment process, permitting more cost-effective bridge inventory management and rehabilitation. The overall goal of this study is to develop a simple NDE methodology that does not depend entirely on previous information of the structure under consideration or require the use of complex analytical models.

The proposed NDE methodology will consist of four stages (see Fig. 1). In the first stage, *visual inspections* would be performed according to the NBIS requirements. Visual inspections are intended to monitor previously detected damage or to identify the appearance of potential problem areas. This stage would provide the necessary information for identifying potential problems such as onset of cracks, loosening of connections, or appreciable deterioration. However, the findings of this stage may not be sufficient to assure that safe service will be provided. Information collected from visual inspection will be used to determine whether the current conditions of the bridge require a more thorough study. This assessment, if necessary, will be executed in the second stage.

In the second stage, *global experimental (GE) techniques* would be performed in order to identify any damage not readily detectable by visual inspections. GE techniques are generally based upon vibration techniques such as forced-vibration or ambient vibration surveys. Such techniques allow testing of a whole structure or a large component of a structure without any *a priori* knowledge of the location of the damage.

The basic concept behind GE techniques is that the vibration response is dependent upon structural parameters such as mass, damping, and stiffness. Therefore, changes

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T. M. Whalen is with the School of Civil Engineering, Purdue University, West Lafayette, IN 47907 USA (phone: 765-494-2225; fax: 765-496-1105; e-mail: whalen@ecn.purdue.edu).

J. F. Gauthier is with the School of Civil Engineering, Purdue University, West Lafayette, IN 47907 USA (e-mail: jfgauthi@ecn.purdue.edu).

J. Liu is with the School of Civil Engineering, Purdue University, West Lafayette, IN 47907 USA (e-mail: jliu@ecn.purdue.edu).

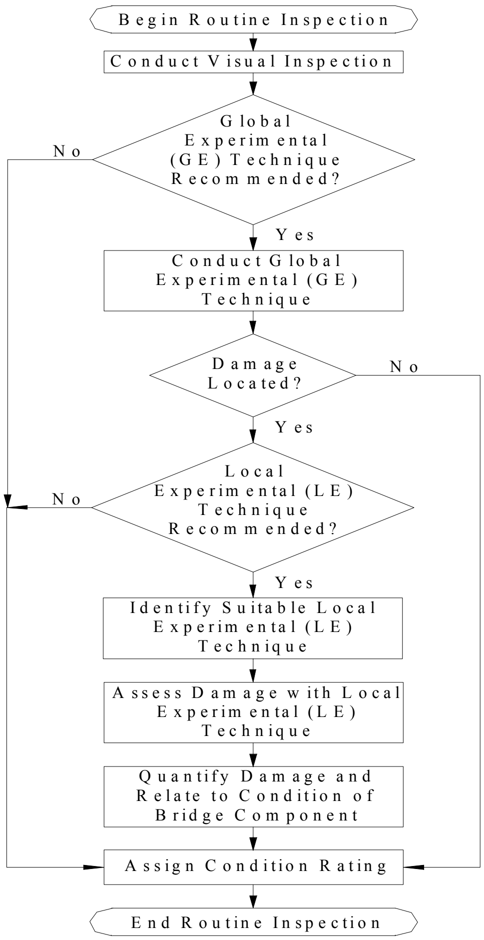


Fig. 1. Proposed bridge rating methodology.

in these parameters caused by damage or deterioration produce changes in the vibration response of the structure. Hence, measuring the vibration response and identifying anomalies in structural parameters and associated response quantities would allow the location (and possibly the severity) of a fault in a structure to be estimated. This process, when combined with appropriate structural identification methods, is called *damage identification.*

It is considered that the evaluation accomplished in this stage will be sufficient for reliably determining the location of any fault and estimating the structural condition of the bridge. Occasionally, however, a detailed assessment of the extent of the damage may be needed. For such purposes, *local experimental (LE) techniques* would be implemented in the third stage to examine portions of the structure in order to characterize specific types of damage and quantify their severity. Results obtained from the damage identification in the second stage would provide criteria for determining whether the use of LE techniques is recommended. Finally, a *condition rating* will be generated to quantify the condition of the bridge.

Among the possible information sets that may be utilized in interrogation for damage identification, modal parameters have been identified as excellent choices due to their relative ease of estimation and accuracy of results. Specifically, it has been shown in the literature that higher order derivatives of the mode shapes — e.g., modal rotations and modal curvatures – are more sensitive to the presence of structural damage than changes in the modal frequencies, modal displacements or modal damping [4]-[6]. Accordingly, a new damage identification method, called the "Differentiated Damage Factor" (DDF) method, is proposed herein based upon fourth derivatives of the mode shapes. An important advantage of the novel DDF method is that it does not require *a priori* data of the undamaged state of the structure. Moreover, few modes are necessary. This is convenient due to the limitations in system identification methods in which only the first few modes of the structure, typically the lowest-frequency modes, are measured. In this study, the differential damage factor method is presented, and results from preliminary analyses of the proposed method performed on numerical and experimental models are discussed.

II. The Differentiated Damage Factor Method

As indicated above, it has been found that higher order derivatives of the mode shapes enhance the ability to locate structural damage. The basis for the use of higher order derivatives is their sensitivity to small perturbations in the system that could overcome the errors introduced by the modal estimation process. Accordingly, the DDF method was developed using the fourth derivative of the mode shapes of an Euler-Bernoulli beam model for the bridge. Accordingly, the deflection $y(x,t)$ of the bridge is assumed to be influenced only by bending moment effects – shearing deformation is ignored. Also, it is assumed that the linear mass density ρA and stiffness EI of the bridge can vary with position x. This leads to the following governing equation of motion for the "beam":

$$\frac{\partial^2}{\partial x^2}\left[EI(x)\frac{\partial^2 y}{\partial x^2}\right]+\rho A(x)\frac{\partial^2 y}{\partial t^2}=0. \qquad (1)$$

Taking the deflection as a superposition of modes, i.e., $y(x,t)=\sum_n Y_n(x)q_n(t)$, gives the following equation for the mode shape $Y_n(x)$:

$$\frac{d^2}{dx^2}\left[EI(x)\frac{d^2Y_n(x)}{dx^2}\right]-\omega_n^2\rho A(x)Y_n(x)=0, \qquad (2)$$

where ω_n is the natural frequency of vibration for mode n. Solving (2) for the fourth derivative gives us

$$Y_n''''(x) = \frac{\omega_n^2 \rho A(x)}{EI(x)} Y_n(x) - \frac{2EI'(x)}{EI(x)} Y_n'''(x) - \frac{EI''(x)}{EI(x)} Y_n''(x). \quad (3)$$

From (3), it can be observed that if the stiffness of the beam is constant (*e.g.*, if no damage is present), the derivatives of *EI(x)* would vanish and thus no anomalies would be expected in the Y_n'''' curve – it would be a scaled version of the modal displacement curve. However, if damage causes a change in *EI(x)*, the coefficients of Y_n''' and Y_n'' will have large values due to the localized nature of this change. Therefore, if there is a fault in a structure, the magnitude of Y_n'''' will increase sharply at the location of the fault. Hence, Y_n'''' is suitable for locating damage.

Extending to discretized systems and using multiple modes of vibration, an averaged sum of derivatives can be used, as expressed by

$$\Delta_i = \frac{1}{N} \sum_{n=1}^{N} \left| Y_n''''(x_i) \right|, \quad (4)$$

where Δ_i represents the average value of *N* modal fourth derivatives at nodal point $x = x_i$. Finally, the DDF value corresponding to the location $x = x_i$ of the beam is normalized via

$$DDF_i = \frac{\Delta_i - \mu_\Delta}{\sigma_\Delta}, \quad (5)$$

where μ_Δ and σ_Δ represent the mean and the standard deviation of the Δ_i values over the beam, respectively.

This method takes advantage of the central difference approximation, in this case applied twice to obtain the fourth derivative. From previous investigations in which this approximation has been applied, it has been observed that erroneous estimates can be obtained when derivatives are being determined over support locations or point of discontinuity. In such cases, the second backward and forward differences are used to avoid differentiation over a discontinuity or support.

III. Preliminary Analysis of the DDF method

A. Numerical Bridge Model Studies

A preliminary analysis was undertaken to determine the degree of sensitivity of the DDF method to changes in stiffness parameters. The main focus of this investigation was the detection of localized damage, such as those found at bolted girder splices or welded connection details. This type of damage was selected because of its difficulty to detect by visual inspection methods [7]. Finite element (FE) models were used to simulate a simply supported steel girder, two-span continuous steel girder, and a two-span bridge structure composed of a concrete deck and six steel girders. Springs in the transversal (vertical) and rotational directions were used to simulate the behavior of a bolted or welded connection on the steel girder.

After benchmarking a FE model of a spliced steel girder in its undamaged condition against a FE model of a non-spliced steel girder, numerical studies were performed to evaluate the DDF method. Damage was introduced by reducing the stiffness of the spring used to simulate a splice connection. A two-span bridge model composed of an 8 in. concrete deck supported by six W36x232 steel girders spaced at 80 inches was developed (see Fig. 2).

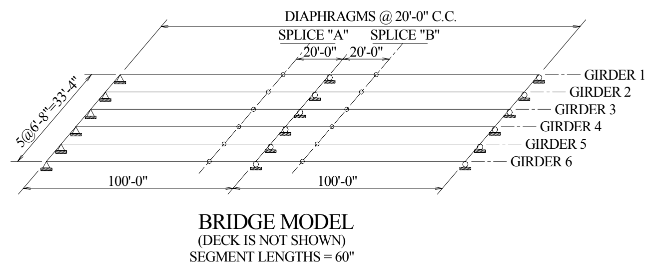


Fig. 2. FE bridge model.

The steel girders and the concrete deck were modeled, respectively, using two-node beam elements and four-node shell elements. Full composite action between the concrete slab and the steel girder was assumed and modeled using rigid links [8]. Each girder was divided into equally spaced segments, with a segment length of 60 inches. In this analysis, damage was simulated by reducing progressively the rotational stiffness K_θ at (i) Splice "A", located on Girder 1, and (ii) Splice "B", located on Girder 5, between 5 to 50 percent. The rotational spring stiffness was chosen because previous analysis results showed that changes in this spring stiffness are more difficult to detect than changes in the transversal spring stiffness. In other tests, damage was simulated by reducing the stiffness of some elements outside the splice zone. Combinations of damages (on elements and splices) were also examined.

Fig. 3 shows the maximum values of DDF for the six girders of the bridge model corresponding to the damage patterns previously indicated. Damage was successfully located from all damage scenarios. The maximum DDF values were located at the damage location for each damage case. It was observed that better results were found when only the first flexural and torsional modes are used. The method was an adequate indicator of damage severity since the maximum DDF increases as the level of damage increases. However, the DDF method produced a more

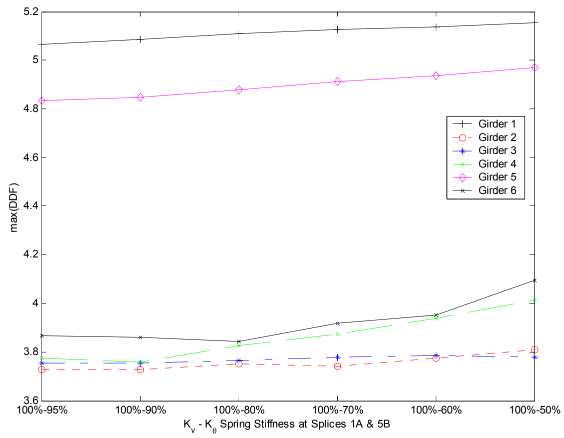


Fig. 3. Maximum DDF for each girder of numerical bridge model.

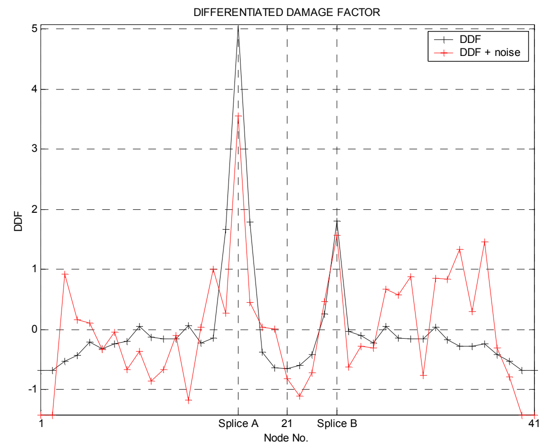
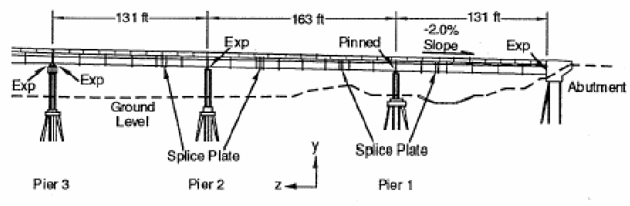
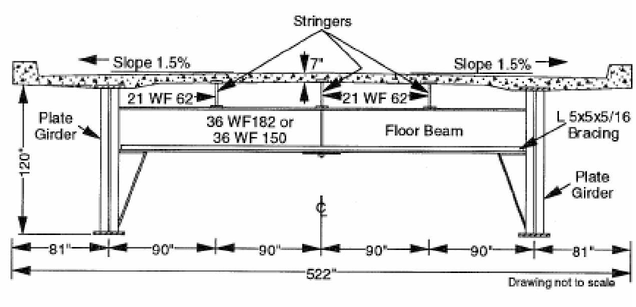


Fig. 4. Typical effect of noise on DDF.



(a)



(b)

Fig. 5. Bridge geometry (from [3]). (a) Elevation, (b) Cross-Section.

robust indication of damage for the less severe cases in comparison to other DI methods examined.

An additional study was performed to examine the robustness of the DDF method in the presence of noise. Using the FE bridge model discussed above, a 5% reduction in rotational stiffness was given to Splice "A" on Girder 1, with no stiffness changes applied to the other girders (including Splice "B" on Girder 5). The first bending and torsional modes were measured for Girder 1 under this damage scenario, and then normally distributed, zero mean white noise of various root mean square (rms) amplitudes was added to each mode. Fig. 4 shows a typical result for the effect that the added noise had on the DDF results. It can be seen that the damaged splice "A" was still identified by having the maximum DDF value, although the noise did cause a change in this peak. Splice "B" was also still identified as a location of high DDF value, although this peak is now less distinct when compared to other, noise-induced peaks. In the case shown, the signal-to-noise ratio (rms) for the mode shape was found to be 51.68 dB; tests indicated that higher noise amplitudes lead to less distinct peaks in the DDF values at Splice "A".

B. *Experimental Bridge Data*

The next study in this preliminary investigation was a comparison of the DDF method to a more established global experimental method called the curvature damage factor (CDF) method [5]. This method is based on averaging (over all modes) the absolute difference between modal curvatures measured in the "damaged" bridge and curvatures previously measured in the "undamaged" structure. This method was selected for testing due to its accurate performance in detecting damage in the numerical models discussed above.

Data from a full-scale plate girder bridge field experiment performed by Farrar and Jauregui [3] was used to assess the performance of the DDF and CDF methods in the presence of real-life noise and other compromising effects. This three-span bridge is composed of a concrete deck supported by two plate girders and three steel stringers. Loads from the stringers are transferred to the plate girders by floor beams spaced at approximately 20 ft. intervals. Each plate girder was spliced at approximately a quarter distance of each span from both sides of piers 1 and 2. Fig. 5(a) and Fig. 5(b) show the elevation and cross-section of the bridge, respectively.

Farrar and Jauregui performed a series of forced-vibration tests on the undamaged bridge and four different damage levels; two of these levels (corresponding to the least damaged and most damaged states) were used in our study. Damage was introduced by progressively cutting from the web to the bottom flange of the plate girder 2 at the center of the middle span, simulating fatigue-crack propagation in the girder. Accelerometers, spaced at about 30 ft., were mounted to the inside web of both girders at the

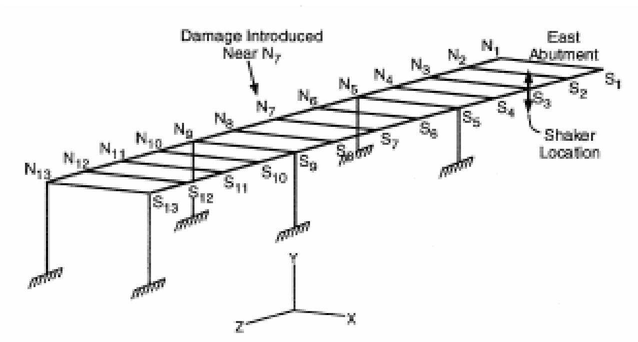
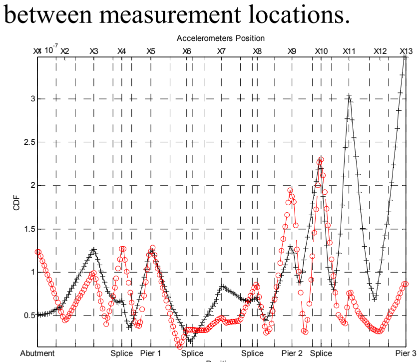
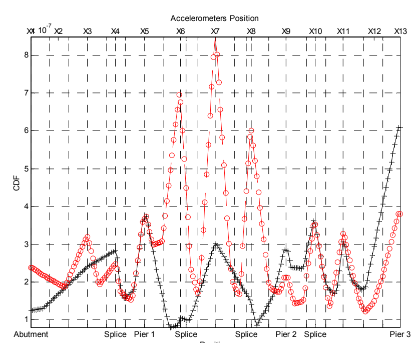


Fig. 6. Accelerometer and damage locations (from [3]).

mid-height to measure the response of the structure in the vertical direction. Fig. 6 shows the accelerometer locations and the damage location, which is near position 7 on the north girder. In order to enhance the damage detection procedure, a cubic-spline interpolation was used in the analysis to estimate the magnitude of the mode shapes at intermediate locations between sensors This type of interpolation was found on previous analyses in this study to be more effective in comparison to the cubic interpolation. Accordingly, the maximum segment length used was 20 in. after using a cubic-spline interpolation function between measurement locations.



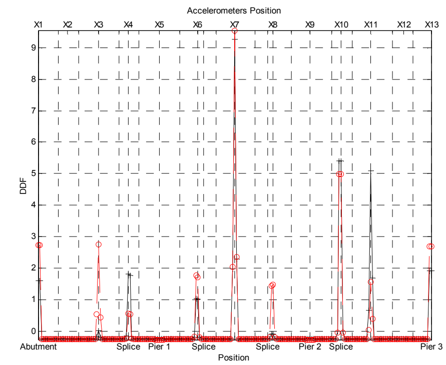
(a)



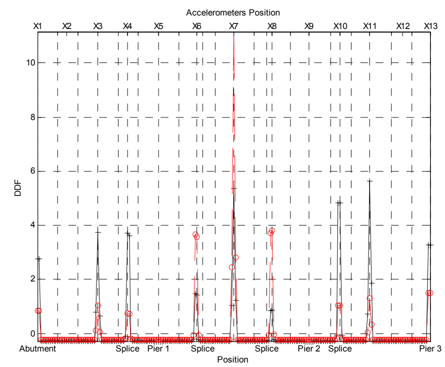
(b)

South Girder (Undamaged) North Girder (Damaged)

Fig. 7. CDF values for experimental bridge data. (a) Damage case 1. (b) Damage case 4.



(a)



(b)

South Girder (Undamaged) North Girder (Damaged)

Fig. 8. DDF values for experimental bridge data. (a) Damage case 1. (b) Damage case 4.

Fig. 7 and Fig. 8 show the results of applying the CDF and DDF methods to the experimental data. It can be seen that the CDF method did not locate the damage properly in the less severe case. Peaks were observed at locations not corresponding to the damage location. Only the more severe (fourth) damage state was properly detected and located by this method. Note that all six measured modes of the bridge were used to calculate the CDF values. Conversely, the DDF method located all of the damage cases accurately, using only the first flexural and torsional modes. It was found that both the CDF and DDF methods captured the location of the splices. Particularly, the DDF method was found to be the most sensitive to the changes in stiffness associated with splices. This is attributed to the sharp discontinuity that this type of connection produces, which would be associated with a reduction in the stiffness of the girder at the splice. Additionally, due to the geometric configuration of the bridge, sudden changes in stiffness are expected at the floor beam connection to the plate girders. Such locations were also captured by both DI methods, specifically at locations where the measurement sensors were located at or relatively near the floor beams. The DDF method was the most sensitive to these sudden changes in stiffness.

IV. Conclusion

A new methodology for conducting bridge rating, incorporating both global experimental and local experimental methods, has been proposed as a means for overcoming deficiencies in visual inspection practices. It is expected that this methodology would provide a more quantitative and less subjective means for estimating the condition of a bridge, thus allowing for more effective bridge management. As part of this methodology, a new global experimental method, the differentiated damage factor method, has been proposed for use in indicating the presence of damage and locating it within the structure. This method has the advantage of not relying upon knowledge of an "undamaged" state for the bridge nor requiring complex numerical models for application. Tests of this method show that it can robustly perform damage identification in numerical and experimental testing, demonstrating improved performance when compared to other GE techniques.

The analyses have shown that discontinuities inherent to the structure, such as girder splice connections, produce strong signals which may be interpreted as damage without additional processing. Since it is desired to have a methodology that relies as little as possible upon subjective judgments, it is necessary to provide users with a means of distinguishing such "natural" features from actual damage conditions. Additional analyses will be used to develop a procedure for properly distinguishing damage signals from signals due to connections, supports, and other bridge components which produce discontinuities even when undamaged. Also, choice of excitation, placement of sensors, and other variables in the data acquisition process will have significant implications for the quality of the results obtained by the DI method. Thus, in order to develop a practical NDE methodology, these data acquisition issues must be investigated so that guidelines can be established for the use of the NDE methodology in the field.

Finally, measured modal properties are prone to many uncertainties, caused mostly by variations resulting from environmental fluctuations during tests. Other sources of uncertainties are related to the data extraction process and the modal extraction process. Statistical modeling is necessary to distinguish the changes in features caused by damage from changes caused by the impact of these uncertainties – a process referred to as data normalization. The importance of applying a data normalization procedure is that false-positive indications of damage are minimized by establishment of a threshold. Exploration and selection of a statistical model must be performed as part of this work, and a threshold for distinguishing damage from operational uncertainties must be established.

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