2-2022

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Citation Details

Published as: Hafiz, A., Schumacher, T., & Raad, A. (2022). A self-referencing non-destructive test method to detect damage in reinforced concrete bridge decks using nonlinear vibration response characteristics. Construction and Building Materials, 318, 125924.

A Self-Referencing Non-Destructive Test Method to Detect Damage in Reinforced Concrete Bridge Decks Using Nonlinear Vibration Response Characteristics

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Abstract: Several non-destructive test (NDT) methods, namely visual inspection, hammer sounding, chain drag, impulse response testing, impact echo testing, ultrasonic (array) echo testing, and under certain conditions ground penetrating radar (GPR) are currently used to detect and estimate the extent of damage such as delaminations in reinforced concrete bridge decks. In this article, we present a self-referencing NDT method that builds on impulse response (IR) testing to detect damage using nonlinear vibration characteristics. The hypothesis is that for an undamaged

deck, varying the impact force applied to a specific test point does not affect the corresponding frequency response function (FRF) for frequencies that lie within the measurement system's linear operating range. On the other hand, the FRFs for a test point that contains damage changes when the impact force is increased, indicating a nonlinear vibration response. To demonstrate that the concept works theoretically, two 2D finite element (FE) models of a bridge deck, one containing a shallow delamination, were developed and their responses to impact forces of increasing amplitude compared. IR data from an in-service bridge deck was processed and analyzed. Visual inspection results and ultra-high-pressure hydro-blasting performed on the deck for rehabilitation purposes provided an opportunity to compare the obtained results with common inspection methods and actual damage extent. Based on the observations, a new damage index, referred to as nonlinear vibration index (NVI), is proposed and shown to be sensitive to damage, including shallow delaminations that were missed by means of visual inspection.

Keyword: Bridge deck; reinforced concrete; damage detection; delamination; condition assessment; non-destructive testing; impulse response testing; nonlinear vibration characteristics; frequency response function; finite element model.

1. Background and Motivation

Highway infrastructure in the United States and around the world experience degradation due to environmental conditions and increasing traffic volume. Additionally, damage is caused by degradation of structural materials due to aging. The corrosion of steel bars and resulting gradual degradation of the concrete are the most common causes of damage in reinforced concrete structures (NCHRP 2004). Accordingly, bridge engineers are typically concerned about four primary damage mechanisms: steel reinforcing bar (or rebar) corrosion, delamination, vertical cracks, and concrete degradation (Gucunski, Imani et al. 2013). Delaminations in concrete bridge decks, which are the focus of this article, are an advanced form of damage in reinforced concrete bridge decks resulting from advanced corrosion of the embedded steel rebar, and are initiated by the presence of cracks in the concrete and sufficient moisture. The rebars expand due to corrosion, leading to cracking and subsurface fracture planes within the concrete. With advancing corrosion, delaminations can progress to open spalls.

To date, many non-destructive test (NDT) methods have been developed to detect deterioration in concrete bridge decks such as delaminations (Scott, Rezaizadeh et al. 2003, Arndt, Schumacher et al. 2011, Zhang, Harichandran et al. 2012, Gucunski, Imani et al. 2013, Sun, Zhu et al. 2018, Garrett 2019). An ultrasonic stress pulse is used in techniques aiming to initiate high-frequency stress waves, which include impact echo (IE) and ultrasonic echo (UE) testing (Sansalone and Streett 1997, Kee, Oh et al. 2012, Zhang, Harichandran et al. 2012, Shokouhi, Wolf et al. 2014, Scherr and Grosse 2021). On the other hand, low-frequency dynamic response characteristics are used in impulse response (IR) testing (Davis 2003). In the latter method, specific characteristics of the dynamic response to a given hammer impact are evaluated to detect delaminations among other degradations. IR testing is based on a hammer impact resulting in a low strain stress wave and vibrations and it has been primarily used for pile integrity testing (Davis and Robertson 1975). While the methodology of this test has not changed since its popularization in the 1970s, application to other types of concrete members has increased notably (Davis 2003, Davis and Germann Petersen 2003, Sajid and Chouinard 2019). ASTM Standard C1740 provides guidance for evaluating the condition of concrete plates such as bridge decks using the IR method (ASTM 2016). In IR testing, an instrumented hammer is struck against the concrete surface to generate local vibrations, and the dynamic response is measured at a nearby location using a geophone or accelerometer. The frequency response function (FRF) is obtained by dividing the dynamic response by the impact force, where both signals are expressed in the frequency domain. The typical frequency range used to evaluate the condition of a concrete slab is 0 to 1 kHz (ASTM 2016). Several parameters are computed from the FRF, referred to as mobility plot, that are used as empirical indicators of damage. For concrete bridge deck condition assessment, all available NDT technologies have limitations to identify certain types of defects (Abdelkhalek and Zayed 2020). One of the limitations of the IR method is that it cannot detect defects with a large depth-to-size ratio (Lin, Azari et al. 2021). Moreover, limitations in detecting delaminations of a certain size appear to be related to the fixed frequency limit prescribed by the ASTM standard (Clem, Popovics et al. 2013). Finally, the method may not be sensitive to early stages of damage because it solely relies on linear response characteristics.

In structural dynamics, modal analysis is the most popular approach for performing linear-elastic structural system identification, where the modal parameters, i.e., natural vibration frequencies, mode shapes, and damping ratio, can be extracted and monitored over time (Kerschen, Worden et al. 2006, Farrar and Worden 2013). Since these parameters are a function of the structural and material properties, they can be related to the initiation and propagation of damage (Doebling, Farrar et al. 1998). Samman and Biswas (Samman and Biswas 1994, Samman and Biswas 1994) presented waveform-recognition techniques to detect damage in bridges and they applied these techniques under both laboratory and real-world conditions by detecting damage in a laboratorysized bridge and a highway bridge. These techniques depend on a comparison between two dynamic signatures: one from an intact (= reference) state and the other from a state with a certain level of damage. Zhou et al. (Zhou, Wegner et al. 2007) utilized vibration-based damage detection (VBDD) methods to detect and localize low levels of damage in the deck of a two-girder, simply-supported bridge. They conducted their study using laboratory-based experimental and finite element analysis. The methods evaluated included the mode shape curvature method, the change in flexibility method, the damage index method, the change in uniform flexibility curvature method, and the change in mode shape method. They concluded that VBDD methods have excellent potential as structural health-monitoring tools for bridge decks. However, these methods require extracting the mode shapes, a process requiring multiple sensors. Additionally, there is difficulty in extracting the mode shapes for bride decks in the field (Salawu and Williams 1995, Bien, Krzyzanowski et al. 2002) because the excitation forces are required to have sufficiently large amplitudes (Bien, Krzyzanowski et al. 2002). Kee et al. (Kee and Gucunski 2016) used impact-echo (IE) testing in order to improve the interpretation of local flexural vibration modes of delaminated areas in concrete bridge decks. This approach was more accurate than conventional binary images for detecting the areal sizes of shallow delaminations. On the other hand, for deep delaminations, the conventional IE approach was more accurate (Kee and Gucunski 2016). Finally, there are two challenges in using modal analysis methods for damage detection: first, it requires the dynamic response for the reference case, which is unavailable in most cases. Second, temperature variations can have a significant effect on the frequency response of the structure (Zhou, Ni et al. 2011), and there might be significant difficulty in distinguishing between the effects of temperature and damage.

Fundamentally, if a structural system fails to follow the principle of superposition, i.e., its response deviates from linearity, then it can be considered nonlinear (Ewins 1995), and traditional linear-elastic modal analysis cannot be used to analyze the dynamic response. In reality, most structural systems exhibit a certain level of nonlinear behavior (Lin 1990). The sources of nonlinearity can be summarized as (Farrar and Worden 2013): (1) Geometric nonlinearity, when the structure exhibits large displacements, (2) material nonlinearity, when a material exhibits a nonlinear stress-strain response, (3) nonlinear boundary conditions, where imperfect boundary conditions result in a nonlinear vibration response, (4) damage, for example cracking, and (5) energy dissipation due to damping. This last phenomenon is to date not fully understood. According to Samman and Biswas (Samman and Biswas 1994), the identification of nonlinear behavior of a structural system includes three steps. The first step is "Detection," where the existence of nonlinearity in structural behavior is determined. "Characterization" is the second step, where the source and location of the nonlinearity is investigated, and its behavior established. The final step is "Parameter estimation." In this step, the coefficients of the nonlinearity are estimated, and their uncertainty quantified. There are many identification methods that have been established in the preceding three decades, such as the restoring force surface method and nonlinear autoregressive moving average model with exogenous inputs (NARMAX) method (Kerschen, Worden et al. 2006, Noel and Kerschen 2017). Nonlinearity is important in damage detection for cases where damage changes the behavior of the structural form (initially) from linear to nonlinear (Lin 1990). Underwood et al. (Underwood, Meyer et al. 2015) investigated using nonlinear behavior for detecting and locating subsurface damage in composite materials by comparing the FRFs for different input force amplitudes. Idriss et al. (Idriss, El Mahi et al. 2015) found that nonlinear vibration parameters are much more sensitive to debonding damage in sandwich beams than linear vibration parameters. Zhao et al.

(Zhao, Lang et al. 2015) presented a new transmissibility analysis method for the detection and location of damage using the characteristics of nonlinear vibrations of structural multi-degree-of-freedom (MDF) systems.

This literature review reveals an opportunity to improve the sensitivity of the established IR test method to detect delaminations in concrete bridge decks early on. The objective of this study was thus to develop and evaluate a highly sensitive yet simple NDT test method to detect damage such as delaminations in reinforced concrete bridge decks. Unlike traditional vibration-based methods, the method discussed herein is self-referencing, i.e., it does not require a reference measurement of the undamaged state for comparison. Using the impulse response (IR) test procedure, the collected data is analyzed in a manner that enables us to detect damage based on deviation from linearity, following what was originally proposed by Ewins (Ewins 1995), i.e., by comparing the frequency response functions (FRF) due to impacts of varying amplitude. The significance of the proposed method lies in its availability, simplicity, cost-effectiveness, and that its application could be extended to other members.

2. Test Methodology

The proposed method is based on the vibration response of a reinforced concrete bridge deck and produces a nonlinear vibration index (*NVI*) for each test point on the member. The same instruments and general test procedure used for impulse response (IR) testing and vibration-based methods apply: An instrumented hammer is used to create an impact at a specific test point and the vibration response at a nearby location is measured with an accelerometer (see Fig. 1). The proposed method requires applying at least two impact forces with different amplitudes for each test point and measuring their vibration responses separately. The basic concept is that for a test point on an ideal undamaged linear-elastic structural system, varying the amplitude of the impact force does not result in a change in the FRF. On the other hand, a test point on a system that contains damage exhibits nonlinear characteristics, which result in different FRFs for impact forces of different amplitude. The frequency ranges of the FRFs need to be within the linear operating range of the measurement system.

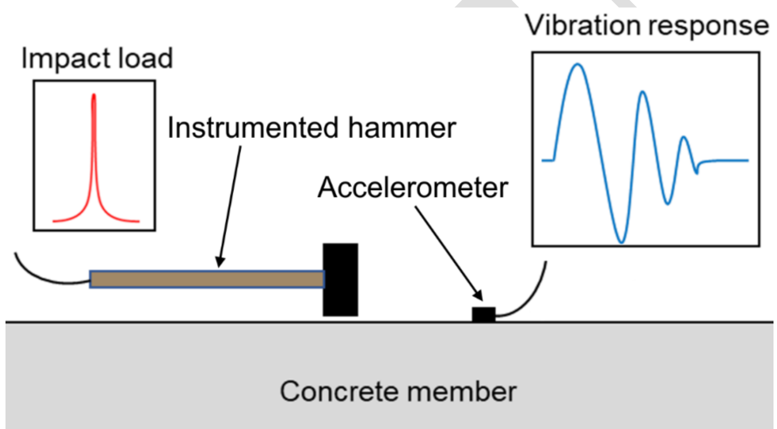

Fig. 1. Illustration of test setup used in this study.

A parameter describing the nonlinearity effect, or deviation from linearity, can be computed in multiple ways, see e.g., Idriss et al. (Idriss, El Mahi et al. 2015), Zhao et al. (Zhao, Lang et al. 2015) and Liu et al. (Liu, Todd et al. 2017). Typically, the correlation coefficient or root mean square are used. Assuming two different impact forces (e.g., soft and strong), our proposed *NVI* is computed for each test point as follows:

$$FRF = H(f) = \frac{Y(f)}{X(f)} = \frac{G_{xy}(f)}{G_{xx}(f)} \quad (1)$$

$$COV\ (H_0, H_i)(f) = \frac{1}{f-1}\sum_{J=1}^{f}\left(H_0 - \mu_{H_0}\right)\left(H_i - \mu_{H_i}\right) \quad (2)$$

$$\rho_{H_0,H_i}(f) = \rho(f) = \frac{COV\ (H_0,H_i)(f)}{\sigma_{H_0}\sigma_{H_i}} = \frac{\sigma_{H_0,H_i}}{\sigma_{H_0}\sigma_{H_i}} \quad (3)$$

$$R^2(f) = \rho\ (f)^2 \quad (4)$$

$$NVI = \frac{\sum_{f_1}^{f_2} R^2(f)}{f_2 - f_1} \quad (5)$$

where $Y(f)$ and $X(f)$ are the frequency domain representations of the measured vibration response and the impact force, respectively, and H_0 and H_i are the FRFs associated with two impact forces having different amplitudes. In this study, the FRF associated with the lowest force of a set of measurements from a particular test point was assigned to H_0, representing the reference case. $COV\ (H_0\ ,H_i)$ is the covariance between H_0 and H_i, and μ indicates mean values. ρ is the correlation coefficient, f_1 and f_2 are the lower and upper limits of a selected frequency range, respectively. G_{xy} is the cross spectrum between the measured vibration response and the impact force and G_{xx} is the auto spectrum of the measured impact force. *NVI* is the proposed nonlinear vibration index and a scalar between 0 and 1, indicating the level of nonlinearity in the structural system under evaluation. When $H_0 = H_i$, then $NVI = 1$, implying the structural system behaves linearly; otherwise, the FRFs are different, which implies that the structural system exhibits a certain level of nonlinearity.

In our proposed method, a test point on a bridge deck without damage is assumed to represent an ideal linear-elastic structural system, i.e., the FRF does not change with an increase in the amplitude of the impact force. This is illustrated in Fig. 2, where the FRFs of four impact forces with increasing amplitude [Fig. 2 (a)] are shown for test point A1 [see Fig. 9 (a)] on the tested bridge deck (introduced in Section 5.1). From Fig. 2 (b), it can be observed that increasing the applied impact force, even doubling it, does not result in significant visible differences between the FRFs, indicating system linearity, which in turn implies that no damage is present in the system. Our hypothesis is that if any area of a bridge deck deviates from linearity, some type and level damage can be assumed to be present. The observed nonlinearity is assumed to be caused by cracking and crack boundary interaction. Note that all other potential sources of nonlinearity must be controlled, i.e., minimized (see Section 5.3). Also, the selected frequency range (f_1 = 225 to f_2 = 500 Hz) was determined by trial and error and is application dependent. More details are provided in Section 5.2.

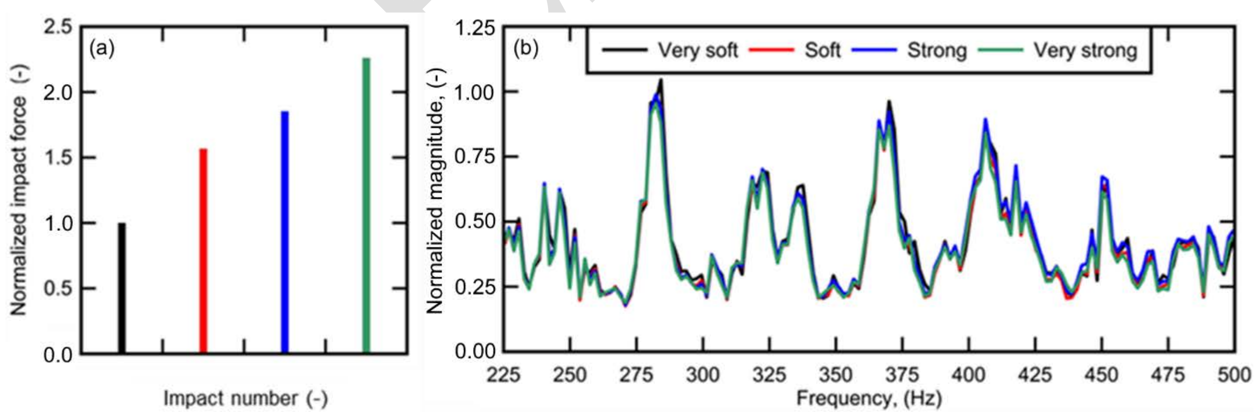

Fig. 2. Sample FRFs for test point A1 [see Fig. 9 (a)] on the selected bridge deck.

Fig. 3 shows a sample of the coefficient of determination, R^2 as a function of frequency. This coefficient was determined by comparing the FRFs of the very soft and very strong impact forces shown in Fig. 2 (b). Finally, the normalized area under the coefficient of determination-frequency curve represents the *NVI*, which for this case is 0.98. This value confirms that the system shows a very high degree of linearity at this test point, which was consistent with visual inspection results.

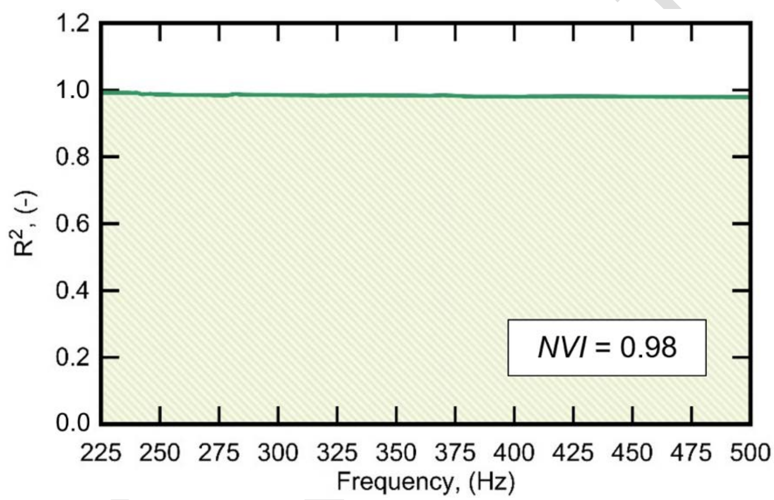

Fig. 3. Sample R^2 -frequency relationship for two FRFs [very soft and very strong from test point A1, see Fig. 9 (a)] vs. frequency and corresponding *NVI*.

Unless otherwise noted, computations were performed in MATLAB (Mathworks 2020) and plots generated in DPlot (Hyde 2014). Regressions and statistical metrics were computed using STATGRAPHICS (Centurion 2020).

3. Numerical Study

3.1. Modeling

A plane strain 2D finite element (FE) model was created to simulate the dynamic response of concrete bridge decks with and without delamination theoretically. The objective was to study the effect of a delamination on the dynamic response and whether it causes a nonlinear response. To that end, depth and width of the delamination were selected based on trial and error to prove that the idea works and not necessarily to represent an actual scenario. The bridge deck was modeled as a simply-supported 2D beam using ABAQUS (Systemes 2012) and guided by previous work reported in (Clem, Popovics et al. 2013). The model was created using quadrilateral elements, as shown in Fig. 4. The span length is 1.00 m (39.4 in) and the depth is 240 mm (9.45 in). The material properties assigned to the deck are normal-weight concrete with a modulus of elasticity, $E_{c,}$ = 23,520 MPa (3,410 ksi) and a mass density, ρ = 2400 kg/m^3 (150 lb/ft^3). An impact force modeled after a typical one observed in the field measurements (see Section 5) was applied as a distributed time-varying force over a length of 40 mm (1.58 in), which corresponds to the diameter of the hammer tip. The forcing function followed a sine (half of a complete cycle) with a duration of 1.8 ms. The acceleration response was measured at a point located 45 mm (1.77 in) from the applied impact force.

Two separate beam models were created: Model 1 refers to the concrete beam without delaminations, i.e., the intact (or reference) beam. Model 2 has the same geometry as Model 1 beam but with a delamination, which was modeled as a gap with the following dimensions: Width = 0.5 mm (0.02 in), length = 800 mm (31.5 in), located about the center of the beam at a depth of 15 mm (0.59 in) (see Fig. 4). To capture interactions of the delamination boundaries during vibration, these surfaces were modeled as contact elements. In both models, eight impact forces were applied to each of the beams where the peak value of the impact force varied from 0.5 to 15 kN (0.112 to 3.37 kip). This range was selected based on the actual forces employed in the field (see Section 5). Note that impact forces reported herein are total force and equivalent distributed forces as applied to the FE models can be calculated as force/0.04 m. A dynamic explicit step routine with a time step of 10 μs and a total simulation time of 1 s was used. The dynamic response of all 16 simulations was analyzed and is discussed in the following subsection.

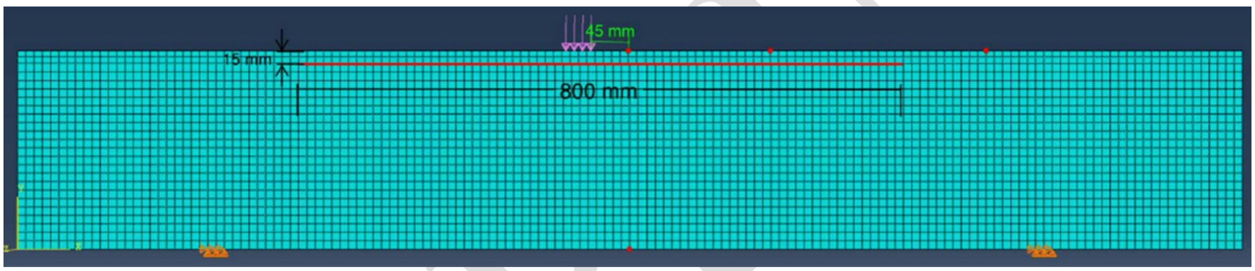

Fig. 4. Illustration of the 2D finite element (FE) model for Model 2. The red line indicates the delamination (gap). The red point at the surface indicates the acceleration measurement point. The purple arrows indicate the distributed force applied on the deck.

3.2. Results and Discussion

Fig. 5 shows the FRFs of the simulated beams with and without delamination, i.e., Model 2 and Model 1, respectively, due to an impact force with an amplitude of 4 kN (0.9 kip). While it is expected that the natural frequencies of the beam change because of the delamination, the interpretation of the results is not straight forward. As can be seen in Fig. 5, the FRFs look very different for the two models. Not only is there no consistent shift between individual peaks, they also do not have corresponding matches, and exhibit notable differences in their half-power bandwidths. The latter implies higher inherent damping in the system. In conclusion, a delamination has a significant effect on the measured vibration response. However, because the proposed method is self-referencing, this is not relevant.

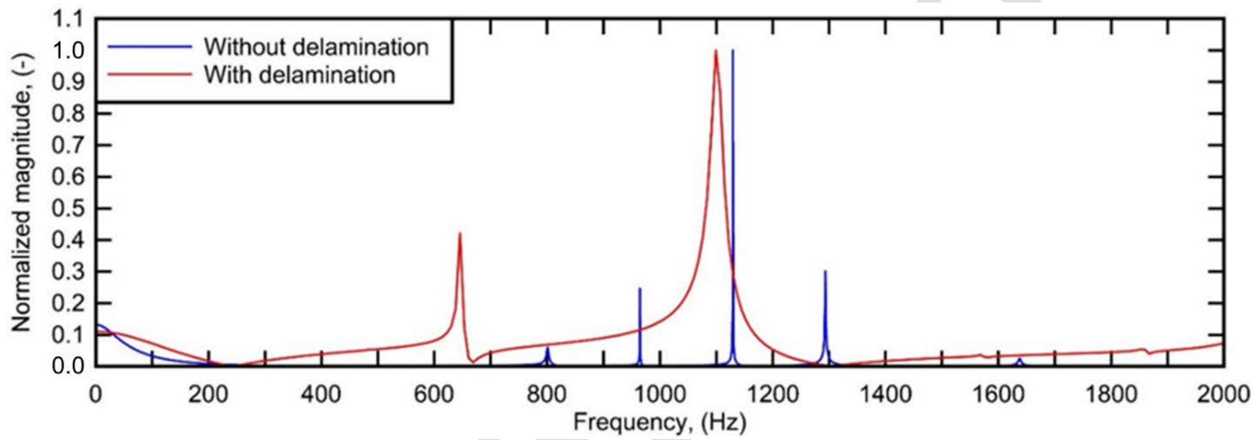

Fig. 5. Two sample FRFs for deck models with and without delamination; amplitude of impact force = 4 kN (0.9 kip).

As can be observed in Fig. 6 (a), the FRF response of the beam model without delamination for a select peak does not change due to an increasing impact force with amplitudes ranging from 0.5 to 15 kN (0.11 to 3.37 kip). On the other hand, increasing the value of the impact force does cause notable changes in the FRFs of the beam model with a delamination. This effect manifests as a change in the magnitude of the selected FRF peaks where the magnitude decreases with increasing impact force, as can be observed in Fig. 6 (b).

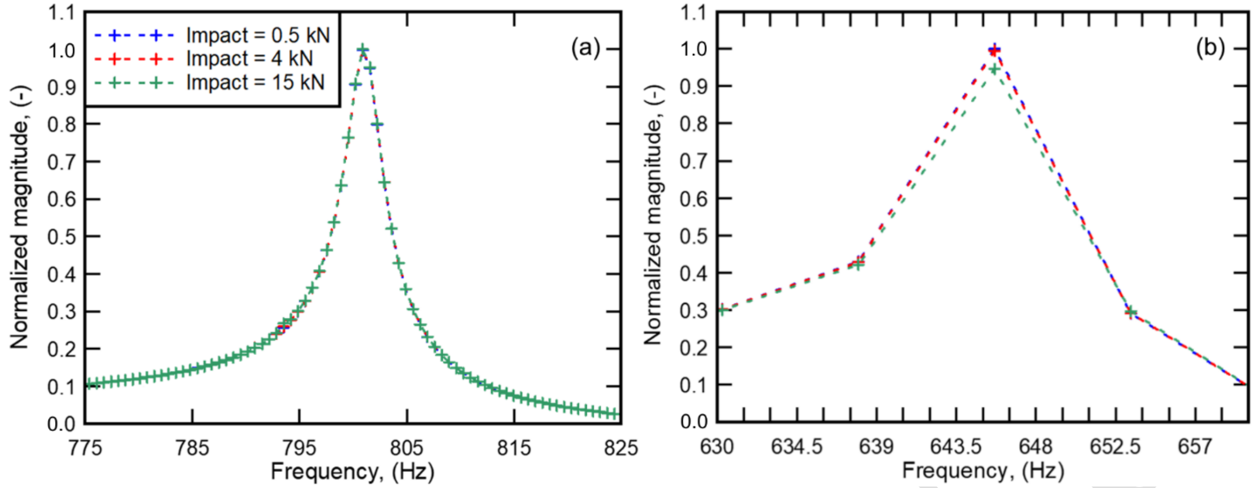

Fig. 6. Three sample FRF peaks (first peak) for impact forces with amplitudes, 0.5, 4 and 15 kN (0.112, 0.9, and 3.37 kip): (a) Model 1 (without delamination, reference) and (b) Model 2 (with delamination).

Fig. 7 shows a comparison of the FRF peak ratios, which corresponds to the FRF peak value normalized with the FRF peak value for the smallest impact force of 0.5 kN (0.112 kip), for both beam models. For Model 1 (reference case), it can be observed that there is a minute increase in the peak response, which can likely be attributed to the nonlinear material response of concrete (see Section 4.3 for further discussion). Model 2 (delamination case), however, shows a clear decrease in the peak response after the force exceeds approximately 3 kN (0.674 kip). This behavior can be associated with contact interaction of the lower and upper boundaries of the simulated delamination when the vibration amplitude of these boundaries exceeds the width of the delamination. Our numerical simulations show that changes of the FRF are sensitive to the presence of a delamination. It can be speculated that other types of damage and degradation have a similar but smaller effect.

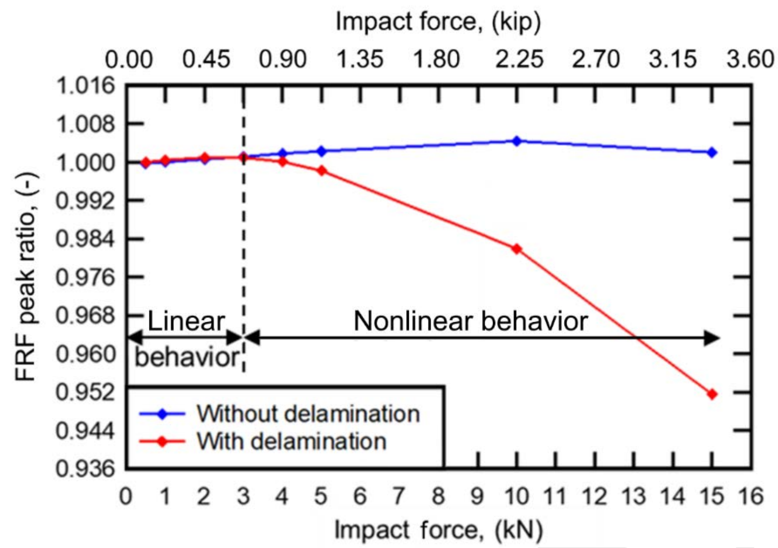

Fig. 7. FRF peak ratio vs. peak impact force value for the two FE models.

4. Experimental Study

4.1. Description of Structure Used for Evaluation

A steel-concrete composite bridge located in Branchport, NJ, USA was selected to evaluate the proposed method's ability to detect damaged areas in an in-service reinforced concrete bridge deck. The bridge, presented in Fig. 8, has a total length of 65.8 m (216 ft) and is 11.3 m (37 ft) wide. The superstructure consists of six 11.0 m (36 ft) long two-span sections with steel girders carrying a 216 to 305 mm (8.5 to 12 in) thick reinforced concrete deck, as shown in Fig. 8 (c). Due to the harsh environment combined with exposure to chlorides from seawater and deicers, the bridge exhibited severe distress when it was visually inspected in July 2011. Five of the six deck sections were found to have severe surface damage, showing visible signs of spalling, potholes, and in some locations the steel rebars were exposed. The deck selected for this study (#2, highlighted in Figs. 8 (a) and b) showed no visual distress and hammer sounding revealed only two small areas potentially having delaminations [see Fig. 9 (a)].

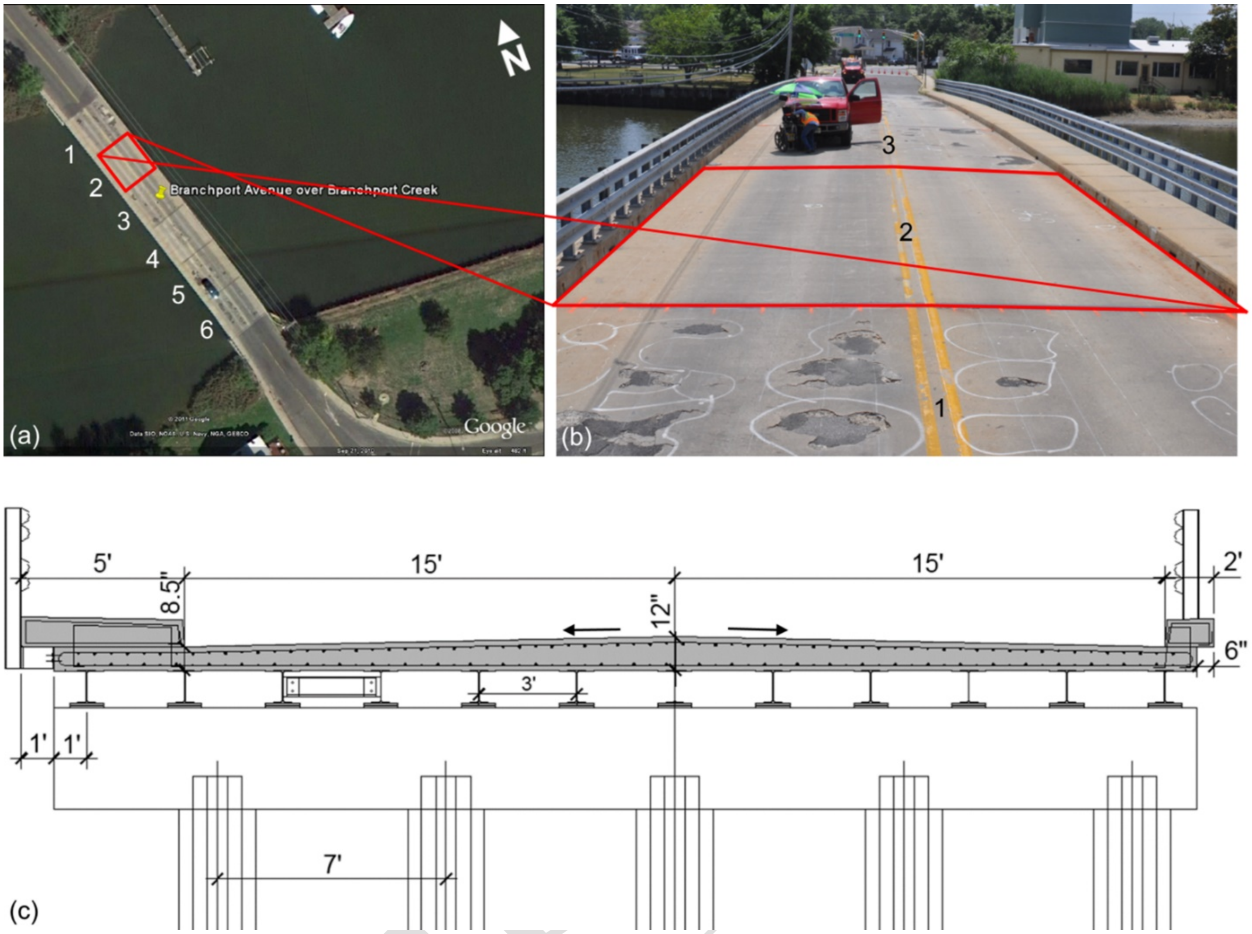

Fig. 8. Branchport Avenue Bridge in Long Branch, NJ: (a) Google map image showing plan view and selected deck (#2) used as part of this study, (b) photo of Deck #2 from a driver's perspective, and (c) bridge cross-section with dimensions in (in = ") and (ft = '). Unit conversion: 1 in = 25.4 mm, 1 ft = 0.305 m.

Since the responsible County had planned to rehabilitate the entire bridge deck, this represented an opportunity to evaluate a variety of NDT methods by comparing their results with the removed concrete. Findings are reported in Clem (Clem 2013) and Celaya et al. (Celaya, Schumacher et al. 2014). The NDT surveys, including the IR testing discussed in this article, were completed in July 2011; hydro-blasting to remove surface as well as damaged concrete was performed in March

2013. Before new concrete was placed in July 2013, the depth of the removed concrete was measured on a 610 x 610 mm (2 x 2 ft) grid. Depth measurements were established using traditional surveying equipment and made available by Cherry, Weber & Associates.

4.2. Test Setup and Procedure

A typical impulse response (IR) test setup was used, as illustrated in Fig. 1. The hammer (PCB, Model 086D20) weighs 0.67 kg (1.5 lb) and has a 51 mm (2 in) diameter hard-plastic hammer tip [±22.2 kN-peak (5000 lb-peak)]. It is equipped with a piezoelectric load cell connected to a signal amplifier/conditioner to measure the generated impact force. The vibration response was measured using a capacitive MEMS accelerometer (Silicon Designs-Model-2260-010) that has a flat frequency response within 3 dB over the range of 0 to 1 kHz. Both input (force) and output (acceleration) signals were recorded using a high-speed transient recorder (Elsys, Model TraNET 204s) with a sampling frequency of 500 kHz.

The two-lane traffic portion of Deck #2, measuring 9.14 x 11.0 m (30 x 36 ft), was divided into a 610 x 610 mm (2 x 2 ft) test grid, resulting in 270 individual test points, as shown in Fig. 9 (a). Two locations were selected for concrete coring and are highlighted by solid black circles. Four hammer impacts were applied at each test point manually, i.e., by a human operator, with increasing amplitude, referred to as “very soft”, “soft”, “strong”, and “very strong”, as illustrated in Fig. 9 (b). Sample results from the proposed test method are shown as colored circles and marked A1, A2, B1, B2, C1, C2 [see Fig. 9 (a)] and are discussed in detail in Section 6.1.

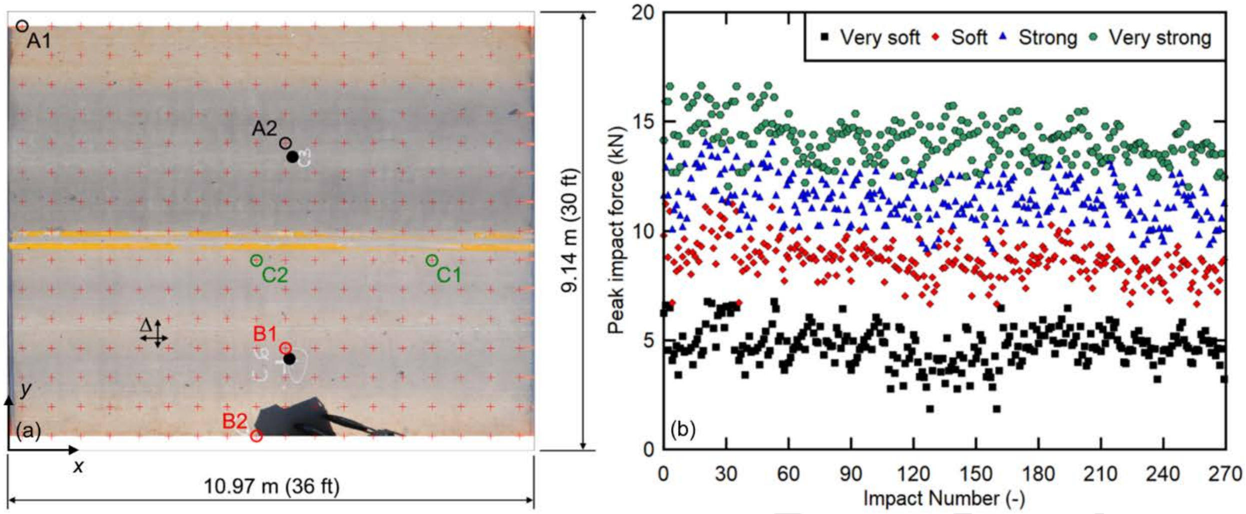

Fig. 9. (a) Plan view photo of Deck #2 with test grid (red '+'), Δ = 0.61 m (2 ft), locations of sample test points A1, A2, B1, B2, C1, and C2, and extracted concrete cores (full black circles). (b) Peak impact forces for all 270 test points. Unit conversion: 20 kN = 4.5 kip.

Fig. 10 shows sample time histories of four impact forces with different levels of amplitude and the corresponding acceleration responses for one select test point.

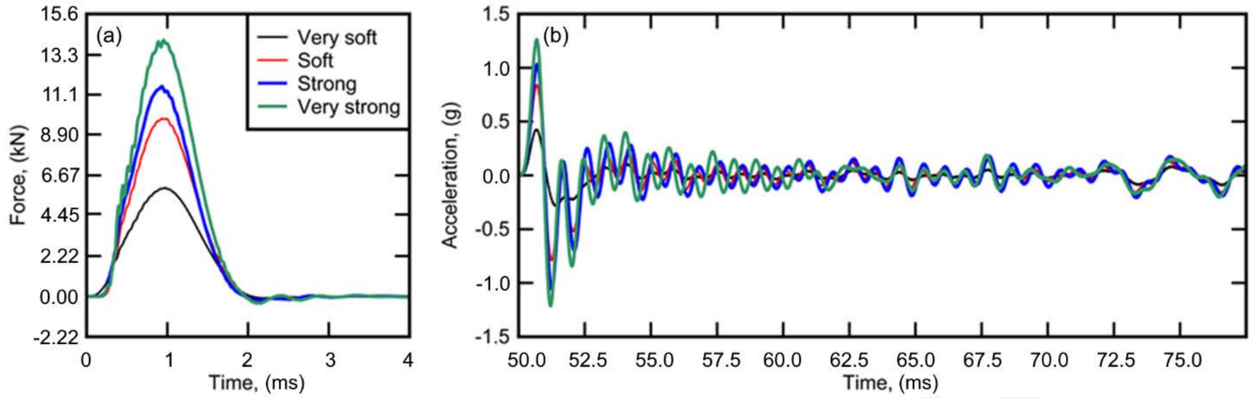

Fig. 10. Samples of (a) four impact forces and (b) corresponding acceleration responses for one select test point. Unit conversion: 15.6 kN = 3.5 kip.

4.3. Sources of Nonlinearity

Structural systems may exhibit nonlinear vibrations due to several factors (Farrar and Worden 2013). For the system investigated in this study, two factors are considered: Material nonlinearity and crack boundary interaction. The former is due to the nonlinear stress-strain relationship of concrete. To ensure that our proposed *NVI* is not affected by this nonlinearity, the stresses generated from the impact forces were calculated and compared with the theoretical concrete stress-strain relationship proposed by Carreira and Chu (Carreira and Chu 1985). Fig. 11 shows this stress-strain relationship, which assumes a conservative concrete compressive strength, f_c’ = 20.7 MPa (3,000 psi). It can be observed that all generated stresses lie within the suggested linear limit (*LL*) of 40% of f_c’ (shown as black dashed line) (fib 2010). The ranges of generated stresses for “very soft” and “very strong” impact forces spanning the means +/- two standard deviations taken from data shown in Fig. 9 (b) are provided for reference. To conclude, effects due to material nonlinearity can be assumed to have a negligible effect on the proposed nonlinearity parameter, *NVI*.

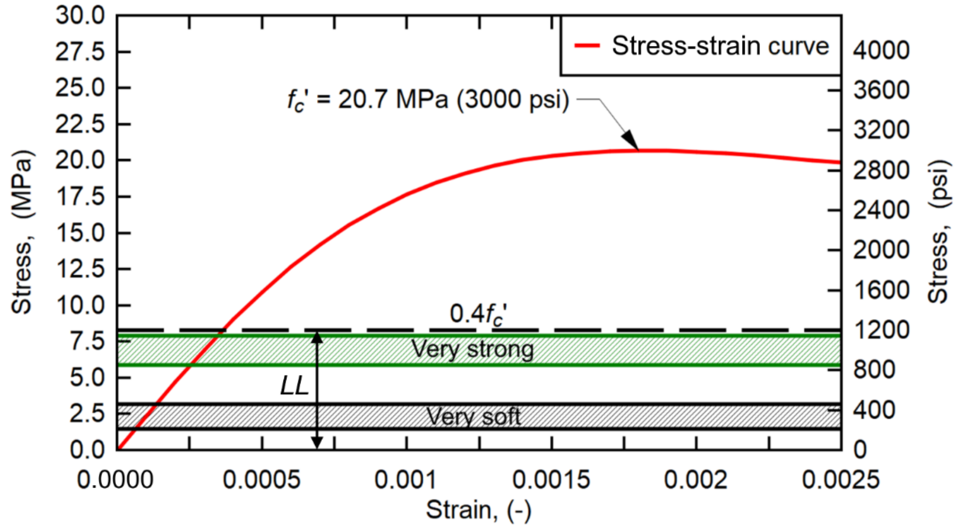

Fig. 11. Theoretical Stress-Strain relationship and actual generated stress ranges due to "very soft" (grey area) and "very strong" (green area) impact forces (ranges span mean +/- two standard deviations). The black dashed horizonal line represents the suggested linear limit (*LL*).

Cracks resulting from concrete degradation is the second factor leading to nonlinearity, and of interest to this study. Cracks open and close during vibrations, leading to a complex dynamic response when the crack boundaries interact, which has been referred to as crack breathing (Giannini, Casini et al. 2014). Although the cracks are initially small and distributed, they may grow and coalesce to eventually form a localized macro crack such as a shallow delamination in a concrete bridge deck. As demonstrated in Section 3, the proposed *NVI* should theoretically be able to detect this type of damage. While a delamination is distinctly different from distributed micro cracks, the crack breathing model still applies; in fact, it can be hypothesized that it is much more pronounced for this case.

To conclude, since material nonlinearity is deemed negligible, only cracking-related degradation should affect the *NVI*. Furthermore, it is assumed that the stress-strain relationship does not have a notable effect on detectability of concrete degradation such as a delamination.

5. Results and Discussion

5.1. Verification of Results from Individual Test Points

As has been reported, the crack boundary interaction of a delamination can cause nonlinear vibrations due to the effects of the crack breathing phenomenon (Giannini, Casini et al. 2014). In this section, results from six test points on Deck #2 were selected and are discussed in detail to evaluate the proposed test method. The six test points were divided into three groups (A, B, and C) according to the observed results from the proposed method, available cores, and visual inspection (see Fig. 9 (a) for test point locations). Each group consists of two test points (see Table 1). Note that "very soft" serves as the reference case for the three other impact forces, namely "soft", "strong", and "very strong". The level of nonlinearity of the tested locations, which is represented by the *NVI*, was computed over a frequency range of f_1 = 225 to f_2 = 500 Hz. The lower limit, f_1 of this subjective range was chosen to exclude low-frequency noise caused by traffic, wind, etc. The upper limit, f_2 was selected to minimize the effect of nonlinearity introduced by the used accelerometer. In a previous study the authors used the same instrumentation and found this type of nonlinear vibration response to start at approximately 600 Hz (Hafiz and Schumacher 2019). Therefore, the upper limit was conservatively set at 500 Hz.

Table 1. Six selected test point coordinates and their *NVI*; "very soft" = reference case.

Group	Location [see Fig. 9 (a)]	*x* [m (ft)]	*y* [m (ft)]	*NVI* (-)		
				Soft	Strong	Very strong
A	A1	0.305 (1.00)	8.84 (29.0)	0.99	0.98	0.98
	A2	5.79 (19.0)	6.40 (21.0)	0.98	0.98	0.97
B	B1	5.79 (19.0)	2.13 (7.00)	0.63	0.49	0.44
	B2	5.18 (17.0)	0.305 (1.00)	0.96	0.91	0.98
C	C1	8.84 (29.0)	3.96 (13.0)	0.38	0.25	0.05
	C2	5.18 (17.0)	3.96 (13.0)	0.89	0.88	0.86

Group A represents two test points that were not found to have any form of degradation by visual inspection. Fig. 2 shows the FRF for test point A1, as well as the peak impact forces. Recall from the discussion in Section 3, although the impact force was more than doubled, this only had a very minor effect on the FRF, which implies the system is linear. Fig. 3 shows the R^2 -frequency relationship for test point A1, which is close to 1, indicating near linear behavior. Similarly, the R^2-frequency relationship of test point A2 is also not significantly affected by the increase of the impact force, as can be observed in Fig. 12 (a). Since any structure will demonstrate a certain level of nonlinearity, 3% can be interpreted as the uncertainty in the *NVI* value for non-degraded concrete in this study. The concrete core taken near test point A2 is further proof that this location is in healthy condition, i.e., not showing any delamination, as can be seen in Fig. 13 (a). In conclusion, areas on the bridge deck that do not show signs of nonlinear vibration behavior can be considered healthy, i.e., free of degradation or delaminations, which supports the basic idea behind our proposed method.

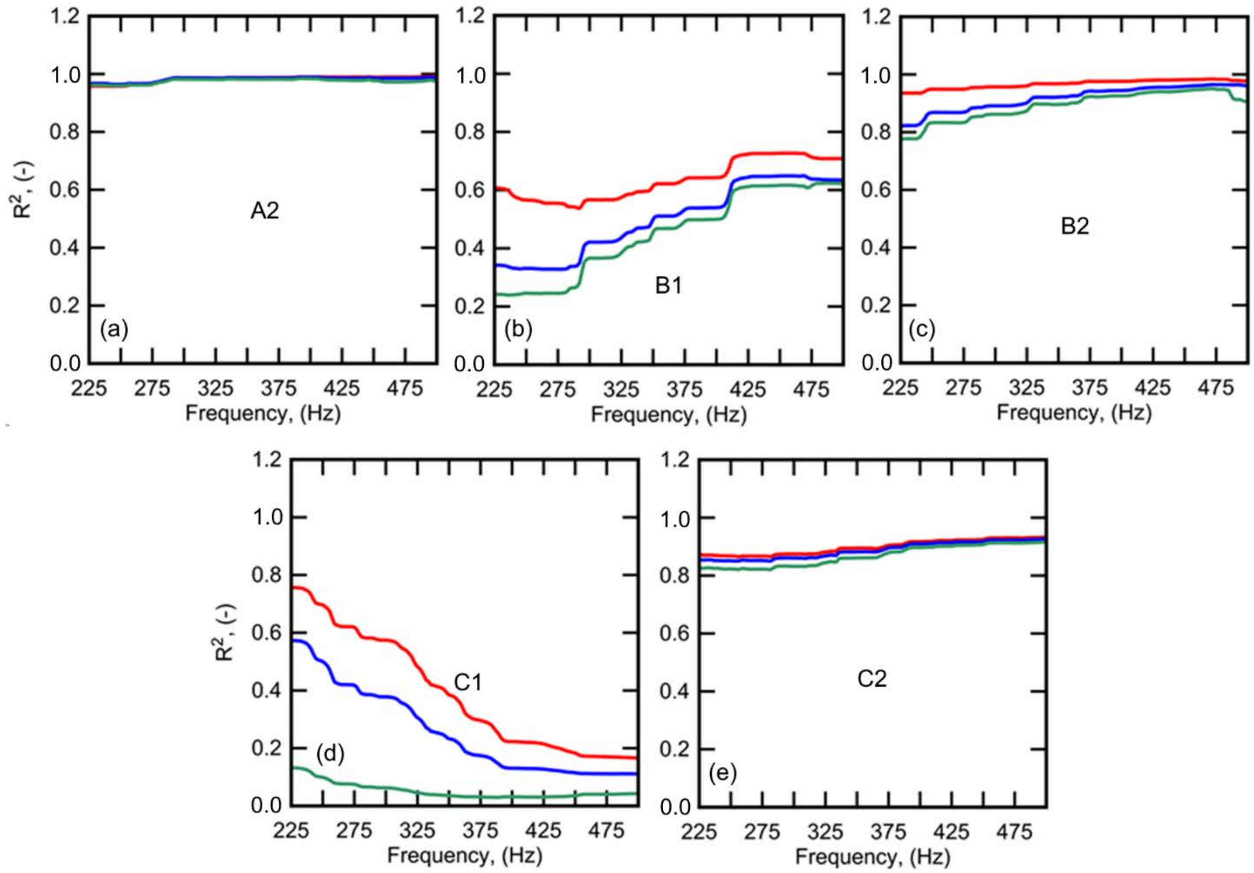

Fig. 12. R^2- frequency relationships for test point A2 (a), B1 (b), B2 (c), C1 (d), and C2 (e). Locations of these test points are shown in Fig. 9(a). Photos of extracted cores corresponding to (a) and (b) are shown in Fig. 13. Note that the R^2- frequency relationships for test point A1 is shown in Fig. 3.

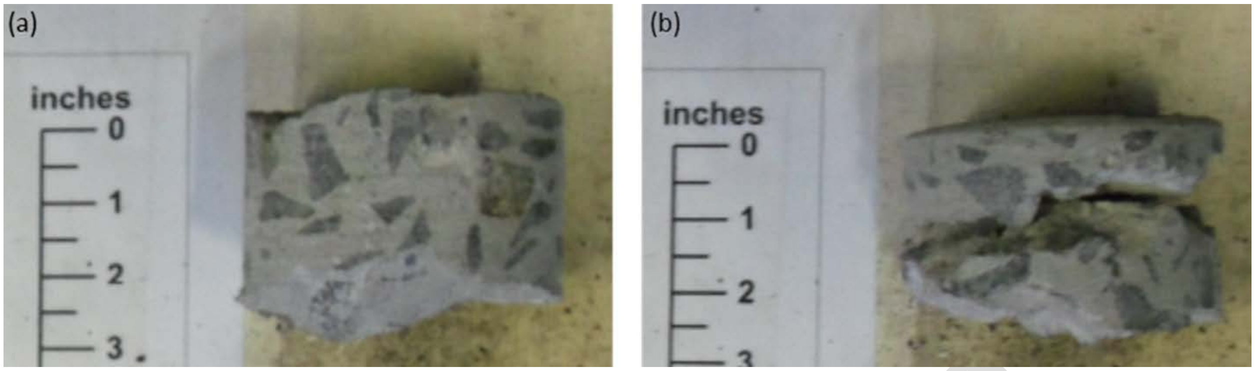

Fig. 13. Photos of extracted concrete cores: (a) Core 1 (near A2) and (b) Core 2 (near B1).

Group B represents two test points that were marked having delaminations by means of chain drag. The R^2- frequency relationships for the Group B test points are presented in Figs. 12 (b) and (c). It can be observed that the responses are significantly affected with increasing impact force, leading to low R^2 functions. Additionally, along with an increase in the amplitude of the impact force, the change in the R^2- frequency relationship increases, resulting in a decrease in the *NVI* value, as shown in Table 1. This response is distinctly different from the one found in Group A. These results also match the core taken near test point B1, which shows a horizontal crack at a depth of approximately 25 mm (1 in) [see Fig. 13 (b)]. In conclusion, the Group B results further confirm that our proposed method can detect delaminations. The *NVI* values for test point B2 are not as low as for B1, which is where visual inspection found a small, delaminated area [see Fig. 9 (a)].

Group C represents two test locations that exhibited nonlinear vibration behavior, but where the visual inspection did not find any degradation or delaminations. The two associated test points exhibited strong nonlinear vibration behavior, as shown in their FRFs [see Figs. 12 (d) and (e)]. Unfortunately, no cores were available for the Group C test locations. However, the *NVI* values could be compared with the depth of the removed concrete after hydro-blasting was performed, which is discussed in more detail in Section 6.2. The depth of removed concrete for these locations was approximately 40 mm (1.6 in) for C1 and C2, which can be considered relatively high. Assuming that hydro-jetting removes more depth when the concrete is degraded, i.e., having distributed micro cracks, the hypothesis that the *NVI* method can detect degradation is also supported.

Since only two cores were available for the entire deck, a comparison between the *NVI* results and the depth of removed concrete was the only way to evaluate the proposed method for all 270 test points, which is discussed in the subsequent section.

5.2. Comparison of Results with Removed Concrete

Fig. 14 shows contour plots of (a) *NVI* values and (b) depth of removed concrete by hydro-blasting across the entire Deck #2. *NVI* values were computed between the "very soft" (= reference) and "very strong" impact forces. Both *NVI* results and depth of removed concrete agree in that there is a large degraded or delaminated area along the centerline of the deck as highlighted by the black dashed box. Additionally, both figures show that the area highlighted by the gray dashed boxes are in good condition. On the other hand, the *NVI* method missed a literal hole in the deck found after hydro-blasting had been completed, as highlighted by the red dashed box. This, however, makes sense, since a hole is simply the case of material missing in some area, which is not the same as an area of degraded concrete. Also, several low *NVI* values, e.g., around x = 5.49 to 6.71 m (18 to 22 ft) and y = 0.91 to 2.13 m (3 to 7 ft), which would point to degradation or delamination, are visible that could not be associated with a high depth of removed concrete. Note that the *NVI* value for test point B1 was consistent with the nearby concrete core that was found to have a delamination, as is discussed in Section 5.1. The discrepancy away from this test point highlights the need for additional work to better understand other factors not yet considered.

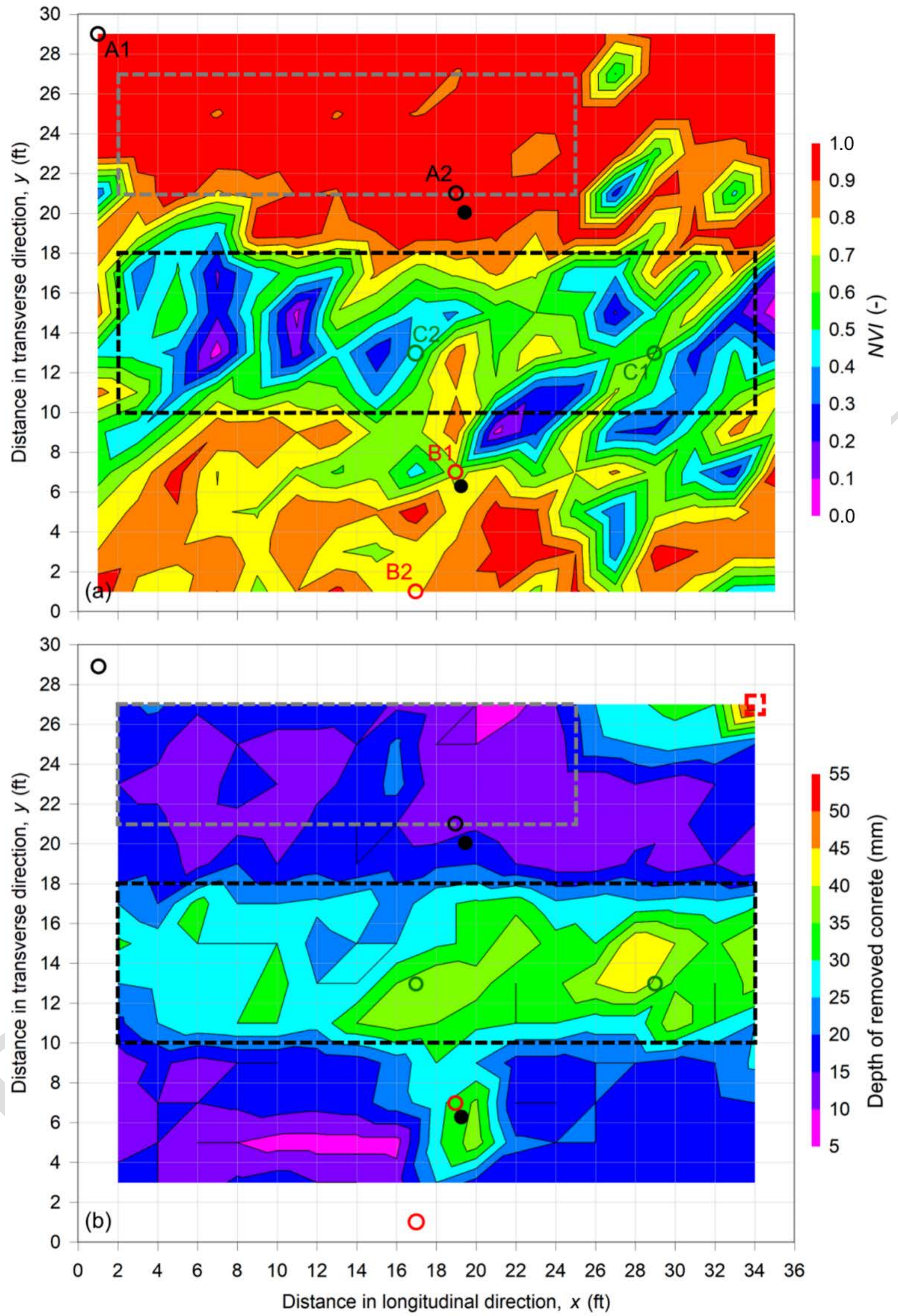

Fig. 14. Contour plots for (a) *NVI* values and (b) depth of removed concrete for Deck #2. Circles depict select test points discussed in more detail in Section 5.1. Unit conversion: 1 ft = 0.305 m.

Fig. 15 shows a correlation plot between *NVI* values and depth of removed concrete. The data behind this plot were generated by interpolating the two datasets shown Fig. 14 over a range x = 0.610 to 10.4 m (2 to 34 ft) and y = 0.914 to 8.23 m (3 to 27 ft) using the generate mesh function with planar interpolation available in DPlot (Hyde 2014). Linear least-squares regression was performed on these data to determine the mean prediction curve (red dash-dotted line) and 95% prediction limits (blue dotted lines). While a linear relationship with statistical significance at the 95% confidence level exists, the correlation coefficient, R = -0.532 is low and considerable scatter is present. As such, this relationship should only be interpreted as an indication of an overall trend. Orange and blue dots correspond to data points from within the grey and black dashed boxes, respectively, shown in Fig. 14, and the red dot corresponds to the location of the hole discussed earlier [at x = 10.4 m (34 ft), y = 8.23 m (27 ft)]. While it can be concluded that the *NVI* cannot be directly used to predict the amount of removed concrete during hydro-blasting, Fig. 15 nonetheless indicates that our proposed method is not only capable of detecting the onset of delaminations but may also be sensitive to distributed damage. Note that a data analysis following conventional IR parameters (ASTM 2016) did not reveal any of these issues (Clem 2013).

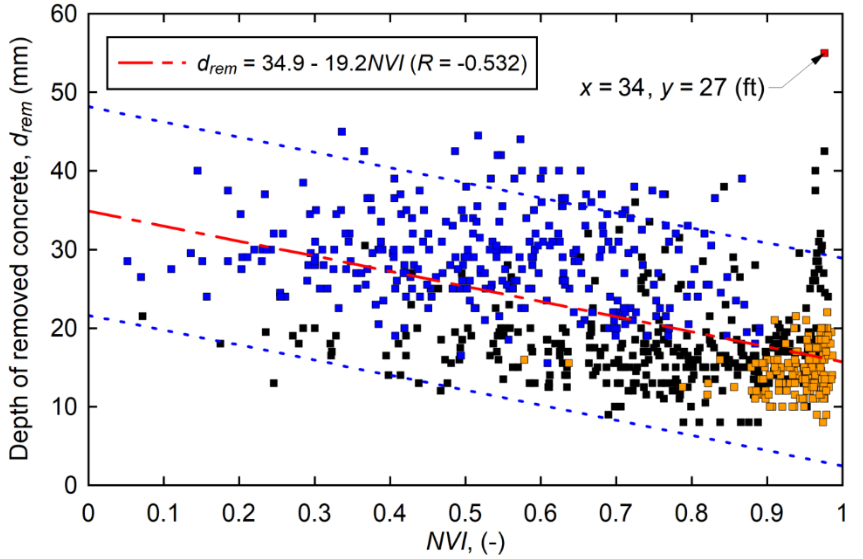

Fig. 15. *NVI* - Depth of removed concrete vs. *NVI* correlation plot. Orange and blue dots correspond to data points from within the grey and black dashed boxes, respectively, shown in Fig. 14. The red dash-dotted line and blue dotted lines represent the mean prediction curve and 95% prediction limits, respectively. Unit conversion: 1 ft = 0.305 m.

6. Summary and Conclusions

The presented results demonstrate the potential for our proposed nonlinear vibration index (*NVI*) method to detect degradation and delamination in reinforced concrete bridge decks. The *NVI* method is based on the concept of deviation from linearity, which is determined by computing the frequency response functions (FRFs) for a set of increasing impact forces applied to a specific test point and comparing them via correlation coefficients. The hypothesis is that if the FRFs remain constant and change, this can be associated with healthy and degraded or damaged areas, respectively. A numerical study using a finite element (FE) model demonstrated that nonlinear behavior was indeed exhibited for a deck with a delamination when subject to increasing impact forces. The proposed method was then evaluated using data from an in-service bridge deck. The equipment is the same as is used for impulse response (IR) tests on concrete plates. The results of the field study support the proposed hypothesis. A comparison between *NVI* results and visual inspection results, extracted concrete cores for six test locations, as well as depth of removed concrete from hydro-blasting was performed. The final observations and conclusions are as follows:

- *NVI* results were able to distinguish healthy areas in the bridge deck with ones that had degradation or delaminations. Additionally, *NVI* results were in excellent agreement with visual inspection and core test results.
- A strong match was found between the results of the *NVI* method and visual inspection and cores for detecting areas that could potentially have degradation or delamination.
- A qualitative comparison between *NVI* values and depth of removed concrete showed acceptable agreement in terms of areas of degradation or delaminations.
- An overall trend was found between *NVI* values and depth of removed concrete. While notable scatter exists, a linear regression revealed a trend consistent with the proposed hypothesis. Note that the mean prediction equation found through linear least-squares regression should not be used to predict depth of removed concrete.

It should be noted that the proposed method, at this point, cannot distinguish between type of damage. Additional research is required before predictions with respect to type of damage and depth of removed concrete can be made reliably. Future work includes additional modeling and laboratory work to establish firm relationships as well as define the limitations of the method. For example, additional scenarios should be studied where crack depth, crack extent, applied force, etc. are varied, to ensure the method works under many possible configurations in the field.

7. Acknowledgments

The authors thank The Higher Committee for Education Development in Iraq for supporting the first author. Test equipment was made available by the Department of Civil and Environmental Engineering at the University of Delaware. The data analyzed in this article were collected as part of a research project sponsored by the Delaware Department of Transportation. We thank Dr. Lassaad Mhamdi, Daniel J. Clem, Tayler C. Wennick, and Kaitlyn N. Gisonda for assisting with data collection. Access to the bridge as well as traffic management during testing was provided by Dr. Kaz Tabrizi and Dr. Manuel Celaya from Advanced Infrastructure Designs and Greg Bitsko from Cherry, Weber & Associates, and are greatly appreciated.

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