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Damage detection on *output-only* monitoring of dynamic curvature in composite decks

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Abstract. Installation of sensors networks for continuous in-service monitoring of structures and their efficiency conditions is a current research trend of paramount interest. On-line monitoring systems could be strategically useful for road infrastructures, which are expected to perform efficiently and be self-diagnostic, also in emergency scenarios. This work researches damage detection in composite concrete-steel structures that are typical for highway overpasses and bridges. The techniques herein proposed assume that typical damage in the deck occurs in form of delamination and cracking, and that it affects the peak power spectral density of dynamic curvature. The investigation is performed by combining results of measurements collected by long-gauge fiber optic strain sensors installed on monitored structure and a statistic approach. A finite element model has been also prepared and validated for deepening peculiar aspects of the investigation and the availability of the method. The proposed method for real time applications is able to detect a documented unusual behavior (e.g., damage or deterioration) through long-gauge fiber optic strain sensors measurements and a probabilistic study of the dynamic curvature power spectral density.

Keywords: bridge; traffic; PDF; detection; output-only; fiber optic sensors

1. Introduction

Concrete deck supported by steel stringers is a frequently used construction system all around the world; consequently, the development of an effective damage detection and localization tool for this structural typology is of importance for guaranteeing continuous serviceability and safety (DOT-FHWA 2014, Volkovas 2013, Kaundinya and Heimbecher 2011). The progress of several monitoring schemes and damage identification techniques over the past decades, with the availability of instrumentation on civil structures, is paving the path both to new applications and to innovative developments.

A set of system identification techniques have been recently proposed using structural response variables only: they are usually mentioned as *output-only* techniques. These approaches originated by the need of operating without disrupting the normal activities (e.g., traffic flow) or by the

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difficulty of consistently measuring the input loading (e.g., wind pressure, traffic load, etc.). Such innovative identification technique can successfully perform when the response of the structural system is independent of the input, or in other words, when the transfer function of the system is independent of the external loading. It is usually related to statidonary (or weakly stationary) white signals (Vicario *et al.* 2015).

Ambient vibration tests and *output-only* measurements on large civil engineering structures have been effectively used in literature for identifying modal properties (Ren and Zong 2004). Modal data assessment is employed for damage detection in structures within the output-only framework. Ambient vibrations and *output-only* measurements are also used for multi-degree-of-freedom frame systems for modal identification in (Nagarajaiah and Basu 2009). The proposed approach is demonstrated also useful for detecting changes in modal frequency due to structural damage. Likewise, damage detection on scale model of a four story steel frame using *output-only* modal data is reported in (Kharrazi *et al.* 2002).

A proposed non-modal damage localization method, devised for long span bridges subjected to a known seismic input (Domaneschi *et al.* 2013) has been recently extended, within an *output-only* scheme, to ambient vibrations induced by an unknown light wind excitation (Domaneschi *et al.* 2016). Wind induced vibrations in long span bridges can be recorded without closing the infrastructure to traffic, providing useful data for health monitoring purposes and identification schemes. The damage detecting feature is defined in terms of the error related to the use of a spline function in modeling the deformation profile of the structure.

Sigurdardottir and Glišić (2013, 2014) recently proposed an *output-only* non-modal damage identification method by using the neutral axis variation as damage sensitive feature. The method is potentially applicable to a large variety of beam-like structures. Bridge applications in particular are investigated through several examples on real structures. Deviation of displacement curvature from polynomial functions has been originally employed in (Sampaio *et al.* 1999, Ratcliffe 2000) for damage indicator. The damage index is defined in terms of variations of the displacement curvature calculated from frequency response functions with respect to a reference intact state. Monitoring data statistical methods are also a consolidated procedures for interpret rough measures for structural health monitoring (SHM) purposes. Owing to simplicity of formulation, speed of execution, availability of several model of correlation they have been extensively employed for structural identification techniques (e.g., in Ardito and Cocchetti 2006, De Sortis and Paoliani 2007)).

This work researches the damage identification in concrete-steel structures that are typical for highway overpasses and bridges. The concrete deck supported by steel stringers is a frequently used structural system all around the world, and consequently the development of a suitable and effective tool for damage detection, localization, quantification and prediction of the remaining service life (SHM Levels I-IV – Doebling *et al.* 1998) is of paramount importance for guaranteeing continuous serviceability and safety.

The method researched in this paper employs measurements collected by long-gauge fiber optic strain sensors (FOS) installed on a real monitored bridge structure where local damage, or deterioration, is expected. The peak value of the power spectral density (PSD) of dynamic curvature measurements has been selected as damage sensitive feature. As previously discussed in literature for bridge applications (Hong *et al.* 2012), the peak values of the magnitudes of PSDs of the distributed long-gauge dynamic strain response was efficient for detecting the damage under ambient excitation and robust with respect to typical excitations. The central topic of the paper is focused not specifically on the use of curvature as damage detection parameter, rather than on the processing of *output-only* curvature monitoring data through a probabilistic approach for real time applications. The achievement of this task represents the novelty of this research work. Finite element simulations have been used in parallel for interpreting the physical behavior and assessing the availability of the damage detection methodology with fiber optic sensors monitoring only. The FE model of the monitored bridge structure has been prepared by using design parameters and validated by comparing the results from modal and transient analyses with dynamic frequencies and strain intensity values, as obtained from monitoring system.

This study confirms the capability of the method for real time applications to detect a documented unusual behavior on the structure (Sigurdardottir and Glišić 2013, 2014) with high confidence. Furthermore, the comparison with the finite element model response allows to validate the procedure and to collect essential indications on the possible damage extension. The contents of this paper are: discussion of the structure and monitoring system, description of the FE model and validation, analysis of peak values of dynamic curvature PSD from real data and comparison with FE analysis, and conclusions.

2. Dynamic curvature analysis - US202/NJ23 highway overpass

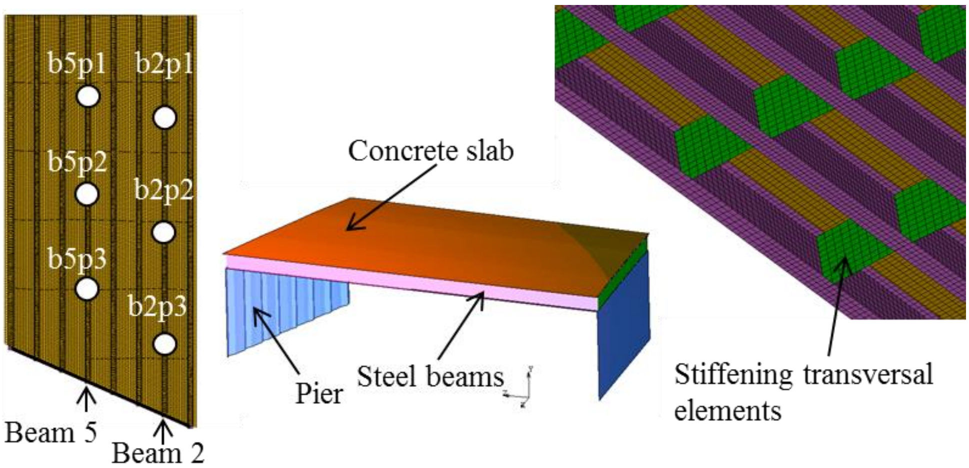
2.1 Structure and numerical model

In order to simplify the presentation, the researched method is illustrated through a real-life application. The structure selected for application is the Span 2 of US202/NJ23 overpass, which is skewed simply supported slab shown in Fig. 1. Approximate dimensions of the slab are 18 m×35 m, and the piers are approximately 7-10 m tall. The Span 2 of US202NJ23 overpass represents an ordinary structure, typically found in highway infrastructure all over the world. This aspect is important for the researched method as it offers wide application opportunities, and convenience for both users (limited service interruption) and authorities (cost saving and effective maintenance). Girders #2 and #5 are equipped with three sets of parallel long-gauge fiber-optic strain sensors each (total of 12 sensors), based on fiber Bragg-gratings (FBG) (Doebling *et al.* 1998). The location used in this study has sensors with gauge length of 2 m. The approximate locations of sensors are in the middles and quarters of corresponding girders as shown in Fig. 2(a). All the discrete FBG sensors were equipped with a temperature sensor, facilitating thermal compensation and temperature monitoring of the concrete. Additional details on the bridge structure and the monitoring system, including determination of uncertainty in measurements and thermal compensation, can be found in (Sigurdardottir and Glišić 2013).

The Finite Element Model (FEM) has been created for the bridge structure by following the available design table, and the validation has been performed by FOS measurements and data analysis. In other words, first the model preparation has been performed by implementing dimensions, boundary conditions and materials, as collected from the original design. Subsequently, the natural vibration characteristics of the model have been compared to the processed monitoring data. Both the procedures have been developed from independent sources and the satisfactory match of the comparison represents the effective validation of the model. Some details of the adopted model are shown in Fig. 2. The model consists of 55230 shell elements and 55556 nodes. The finite element mesh has been prepared within the Marc MSC code due to the effectiveness of its preprocessor for managing the mesh details and connections between longitudinal, transversal, vertical, and horizontal components of the structure. The FE code has proved effective in the simulation of structural systems, in highly nonlinear dynamic conditions also, as demonstrated in (Domaneschi 2012, Colombo *et al.* 2016). Subsequently the mesh has been exported to Abaqus code processor for performing both preliminary static and modal analyses, as well as subsequent transient analyses with truck passage loading. Transient dynamic analysis, also called time-history analysis, is a technique usually employed to determine the dynamic response of a structure under the action of general time-dependent loads, such as the traffic one. The time scale of the loading is such that the inertia or damping effects are considered significant (Bathe 1996).

(a) (b) (c)

Fig. 1 US202/NJ23 overpass consisting of concrete slab supported on steel girders. General side view (a), detail of steel girders #2 and #5 under the concrete slab (b), detail of the sensor positioning at the top and the bottom flange where longitudinal strains are collected, respectively ε_{top} and ε_{bottom} (c) (Sigurdardottir and Glišić 2013)



(a) (b) (c)

Fig. 2 Span 2 of US202/NJ23 overpass, 3D model and FE mesh. (a) Top view of the concrete slab with sensor locations (b=beam, p=point), (b) side view of the 3D model. Mesh details of the steel beams with stiffening transversal elements under the concrete slab (c)

In the next section, the validation of the model with shell elements is discussed. It is worth mentioning that a more refined, but also computationally onerous, model version with solid elements for the concrete slab has been considered and evaluated. Both models, with shells and solid elements for the slab, result equivalent in terms of modal dynamics and transient dynamic responses. Therefore, the more manageable shell elements option has been employed.

Design linear elastic properties for the concrete and steel materials have been adopted as follows: steel 210 GPa longitudinal modulus and 0.3 Poisson's ratio; concrete 30 GPa for the slab and 35 GPa for the piers with 0.2 Poisson's ratio. Table 1 reports the natural vibration characteristics of Span 2 of the US202/NJ23 FEM. Axes *x*, *y* and *z* in Table 1 correspond respectively to the transversal, vertical, and longitudinal directions. Mode shapes are shown in Fig. 3.

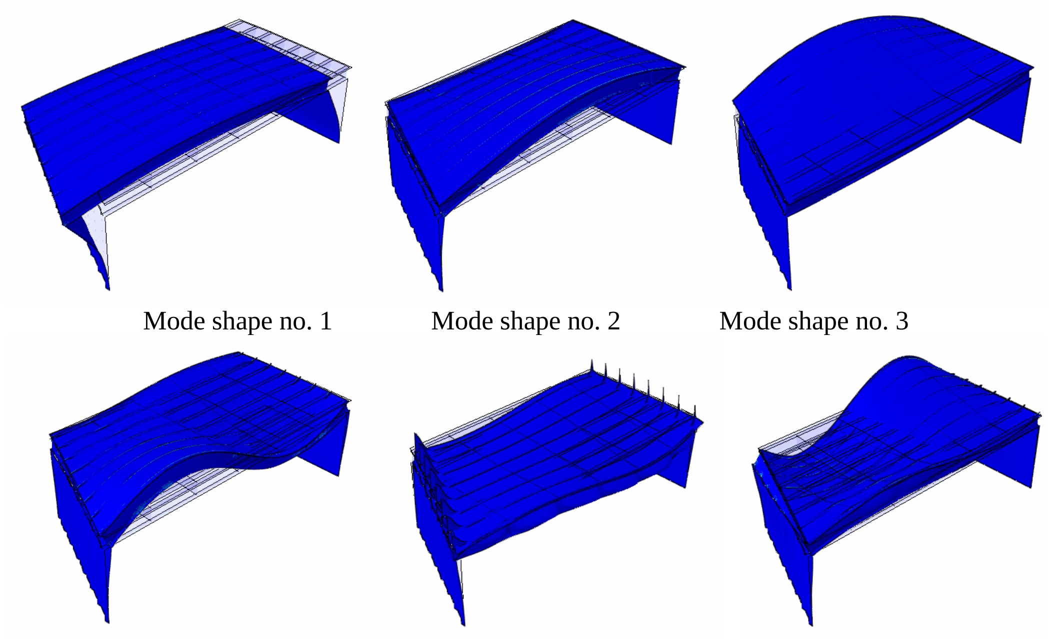


Fig. 3 Mode shapes of the Span 2 of US202/NJ23 overpass as per FE model (transparent parts of images represent undeformed shape)

Table 1 FE model natural frequencies and effective masses

Mode no.	*Frequency* [Hz]	*Period* [s]	*x-component* [%]	*y-comp.* [%]	*z-comp.* [%]
1	2.8	0.36	2.8	1.4	61.1
2	3.2	0.31	0.7	22.3	2.1
3	4.2	0.24	0.3	10.7	0.5
4	10.7	0.09	1.7	0.0	0.3
5	11.7	0.08	39.2	0.0	2.5
6	13.3	0.07	0.7	0.0	0.1

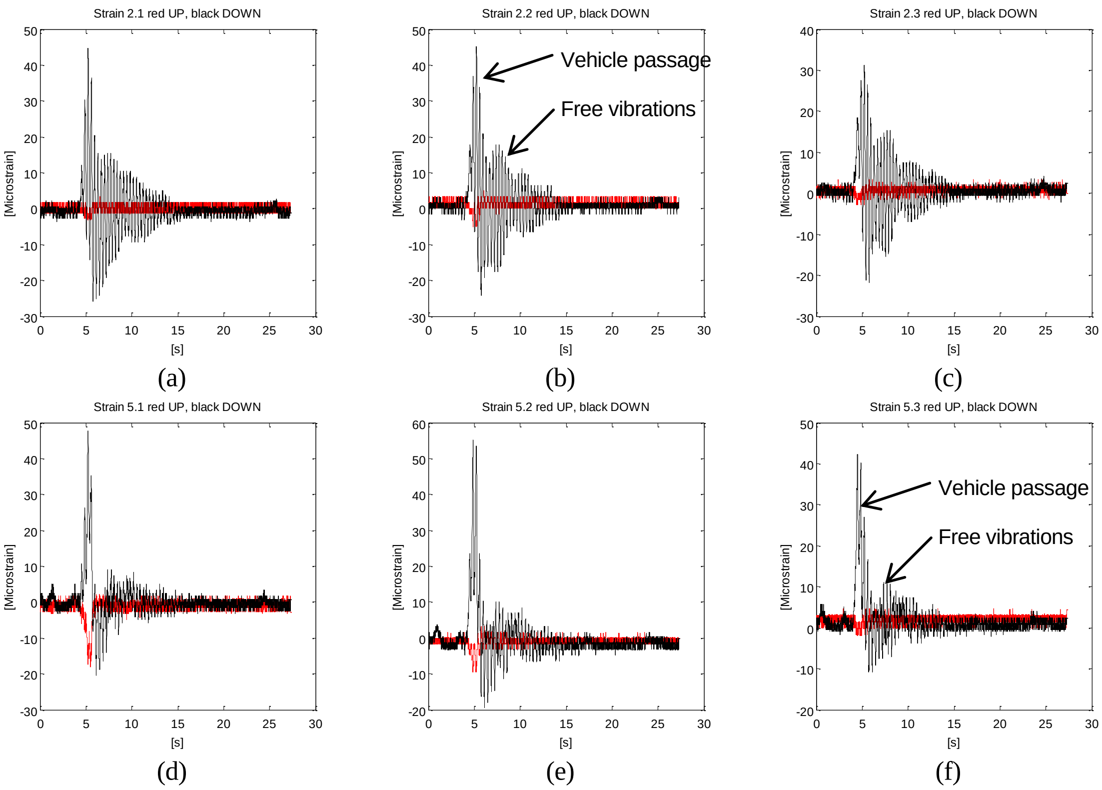


Fig. 4 Traffic event A: measured strain time histories at the monitoring positions

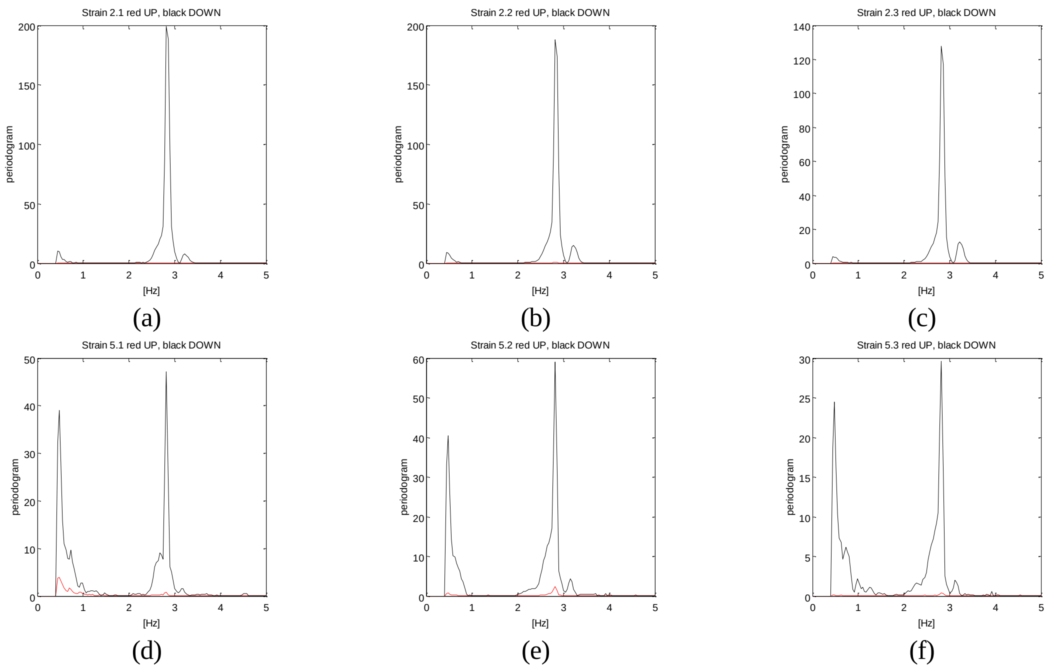


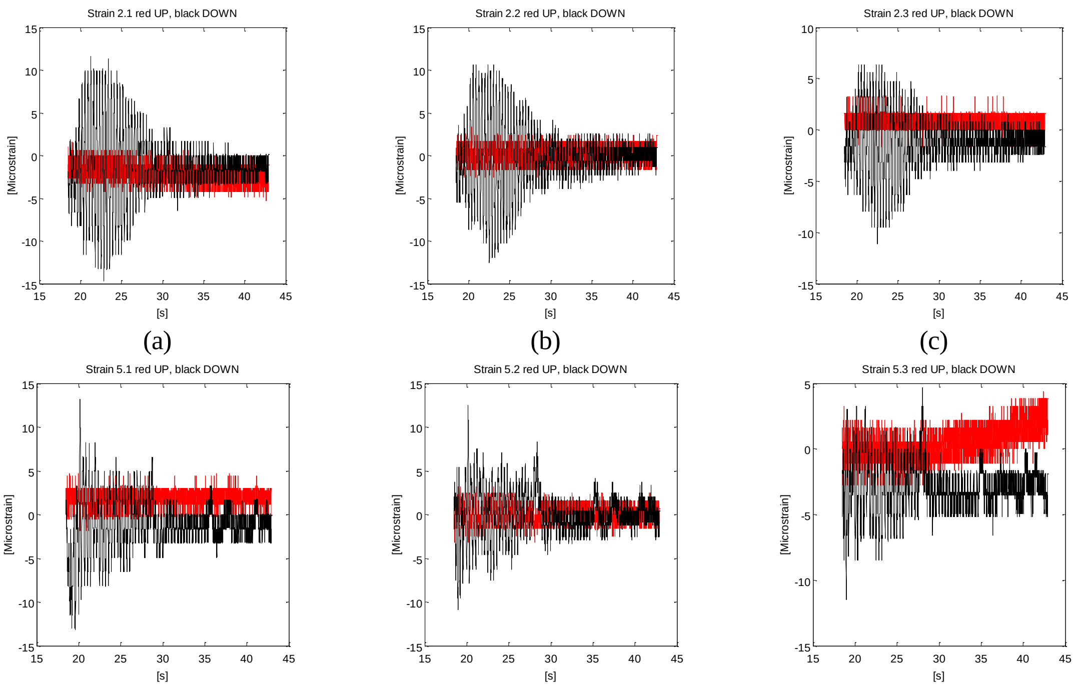
Fig. 5 Traffic event A: structural responses at the monitoring positions in the frequency domain

2.2 Validation of the model by FOS measurements and data analysis

The US202NJ23 overpass has been equipped with a permanent monitoring system, as described in previous section. It is an *output-only* system for dynamic characterization of structures and it allows assessing the bridge slab condition without interrupting the traffic flow and saving completely operational costs. The data, i.e., the strain measurements, are acquired with frequency of 250 Hz. Under the regular traffic, the strain changes registered by sensors are small, in the order of few microstrains. However, passage of a heavy vehicle create events in which the strain magnitude reaches several dozen of microstrains.

Fig. 4 reports the strain-time history for the traffic event “A” on Span 2 of US202NJ23 bridge. Fig. 5 shows the traffic event A in the frequency domain through the periodogram approximation of the PSD. A heavy truck passage is evident, as it is reflected through large deformations at the bottom flange of both instrumented steel beams #2 and #5 (Fig. 4). In addition, Figs. 5(d)-(f) suggests that the truck passage is probably nearer to beam 5 due to the higher amplification in the range 0.5-1 Hz with respect to beam 2 (Figs. 5(a)-(c)). The first two natural frequencies (2.8 Hz and 3.2 Hz) as calculated by the FE model are always clearly visible and this confirms the conformity of the model.

The PSD of a stationary random process is, by definition, a real symmetric function mathematically related to the autocorrelation sequence by the discrete-time Fourier transform. The periodogram is an estimate of the PSD of a wide-sense stationary (weakly-stationary) random process, such as a finite-length segments of a signal (Oppenheim and Schafer 2010, Papoulis and Unnikrishna Pillai 2002).



(a) (b) (c)

(d) (e) (f)

Fig. 6 Traffic event B: measured strain time histories at the monitoring positions

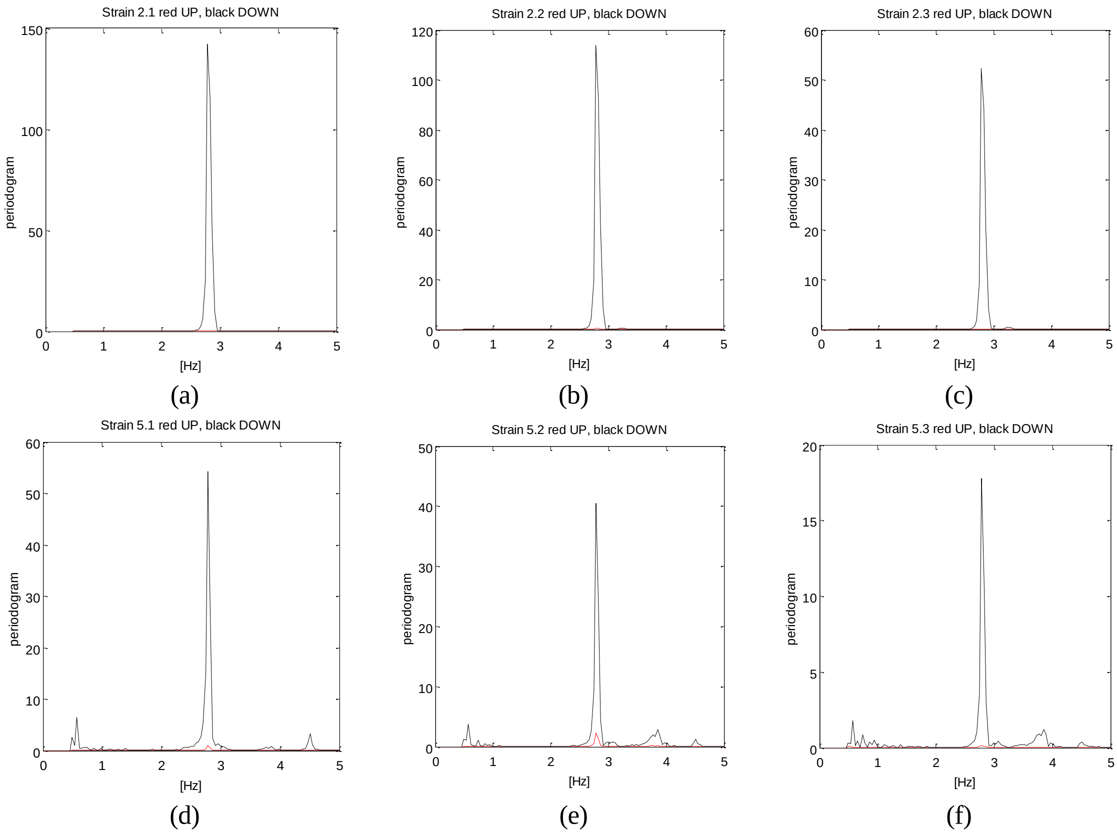


Fig. 7 Traffic event B: structural responses at the monitoring positions in the frequency domain

Comparable outcomes, as those from Figs. 4 and 5, are given in Figs. 6-7 for a different event B. It is important to highlight that the outcomes of the event B are relative to a time history where exclusively the free vibrations consequent to a truck passage are considered. In other words, the truck loading effect has been truncated from the processed signal. As a result, the output observed amplification, in both the time and the frequency domain due to input effect, disappears (compare Figs. 4 and 6, 5 and 7 respectively). Thus, loading disturbances into the frequency domain decomposition have been disregarded. Through this procedure it is possible to confirm the outcomes from event A and, furthermore, to emphasize higher frequency contents. Figs. 7(d)-(f) show the additional peaks close-enough to the third natural frequency in Table 1 (4.2 Hz). Hence, based on the available monitoring data and the FE model outcomes, the FE model of Span 2 of US202NJ23 overpass is validated.

2.3 Curvature from real events

Under the hypothesis of small displacements (deformed shape of the structure corresponding to the underfomed one) and bending moment vector orthogonal to the symmetry vertical axis of the cross section, the curvature is computed from two parallel strain sensors in the cross-section as follows (Corradi 1992)

$$\chi=(\varepsilon_{bottom}-\varepsilon_{top})/h \tag{1}$$

where ε_{bottom} and ε_{top} are longitudinal strains measured by the sensors while h is the vertical distance between the sensors. Through this procedure, the time history of the dynamic curvature is evaluated for each monitored event at the measurement points (see Fig. 1).

Subsequently, the curvature PSD is determined by both Welch and periodogram procedure. If, as anticipated, the periodogram is considered as a weak estimator of the true power spectral density of a wide-sense stationary process, Welch's technique is able to reduce the variance of the periodogram by breaking the time series into overlapped segments (Oppenheim and Schafer 2010, Papoulis and Unnikrishna Pillai 2002). Welch's method computes a modified periodogram for each segment and then averages these window estimates to produce the whole estimate of the power spectral density. Because the process is wide-sense stationary and Welch's method uses PSD estimates of different segments of the time series, the modified periodograms represent approximately uncorrelated estimates of the true PSD and averaging reduces the variability. For the purposes of comparison and evaluation, both techniques have been adopted in this research.

The dynamic curvature PSDs are computed for 26 heavy vehicle passages (events) on the bridge. As an example, four of them are shown in Fig. 8 in terms of frequency response surfaces. The right horizontal axis shows the location points along beam 5 (points 1-3, see Fig. 2) the left

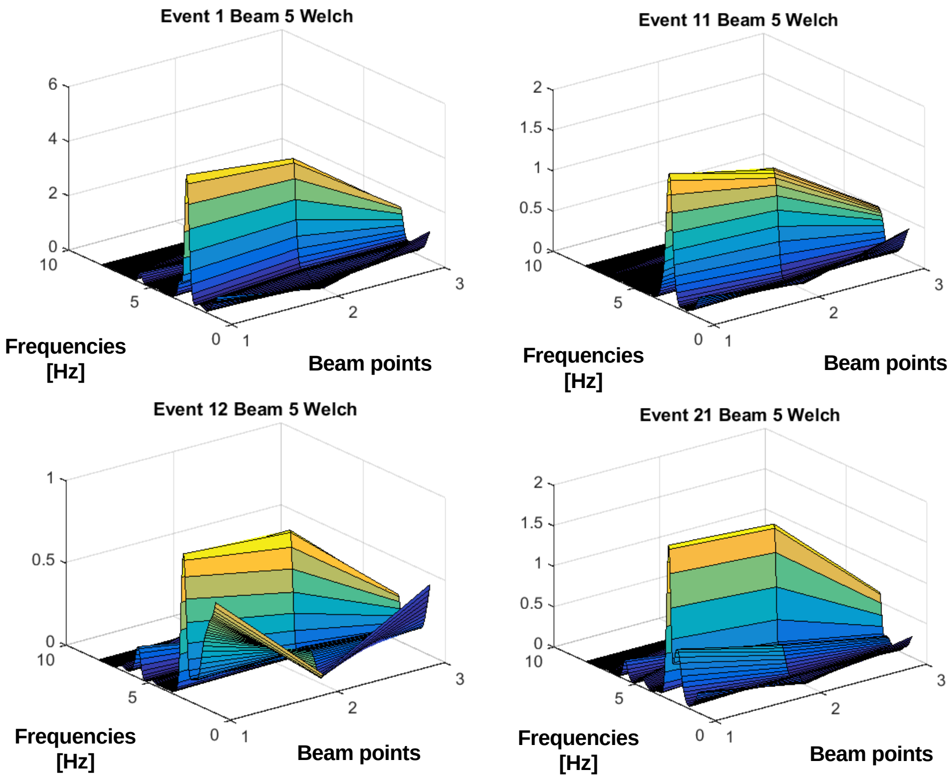


Fig. 8 Welch PSD at beam 5 & points 1, 2, 3 (right side horizontal axis) and frequencies higher than 1.5 Hz (left side horizontal axis)

horizontal axis shows the frequency, and the vertical axis shows the PSD. It is important to notice that the highest value of PSD is observed at the location b5p1, which is close to the quarter of the span. Fig. 9 shows Gaussian distributions of the PSD peaks from Fig. 10 (Events 1-26). Table 2 shows the mean and standard deviation in the data, for each point of girder 5. The mean and standard deviation are higher at b5p1 than the other quarter span b5p3 and approximately of the same intensity as at the mid-span (b5p2).

This behavior is unusual, since in a simply supported beam the maximal value is expected in middle of the span (b5p2), while the values at approximate locations of two quarters (b5p1 and b5p3) are expected to have smaller, mutually similar magnitudes. An unusual behavior at location

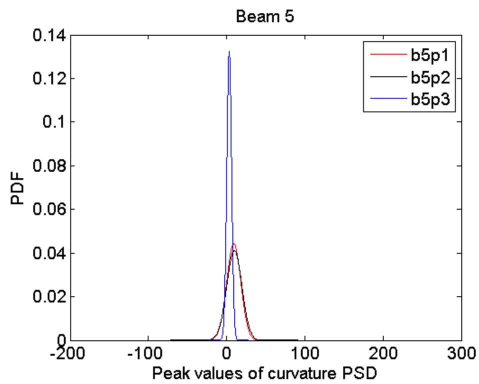
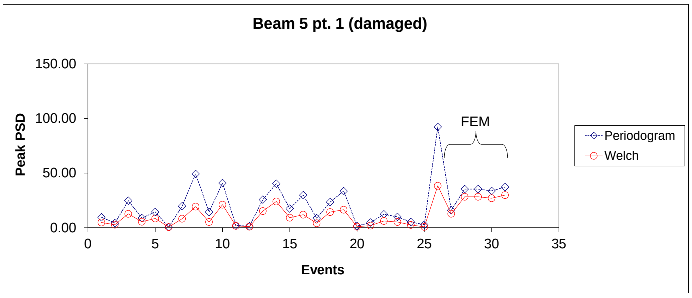
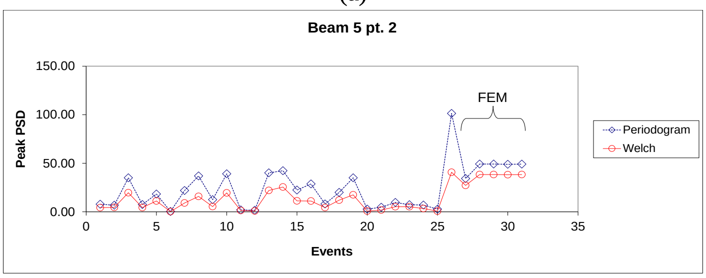


Fig. 9 Gaussian distributions of the dynamic curvature PSD peaks on beam 5

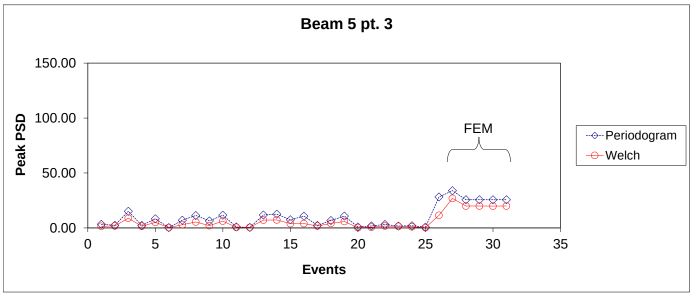


(a)



(b)

Fig. 10 Peak values of dynamic curvatures PSDs (Periodogram - lines with square points & Welch - lines with circle points) for all investigated events (real events and FE simulations) at beam 5. Measurement point 1 - damaged (a), point 2 (b), point 3 (c)



(c)

Fig. 10 Continued

Table 2 Average and standard deviation of PDF in Fig. 10

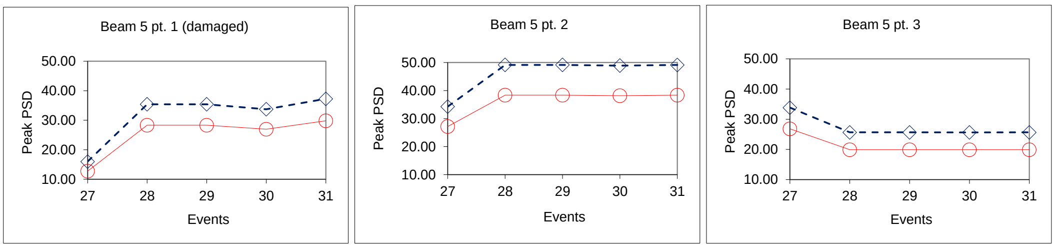
	b5p1	*b5p2*	*b5p3*
Average	20.7	21.3	7.7
Standard deviation	20.0	20.6	6.9

b5p1 has been earlier noticed in an independent study based on analysis of the location of the neutral axis (Sigurdardottir and Glišić 2013, 2014), which could be the reason for unusual value of PSD at that location.

2.4 Transient analyses with standard truck passage simulation:

To further examine and confirm the detected unusual behavior, transient dynamic FE analyses on the Span 2 of US202/NJ23 Bridge model are performed and peak values of dynamic curvature PSD are extracted at all three measurement points of girder 5. The same data processing is repeated for the available FOS monitoring data for all 26 real events, by considering free vibrations only (applying a high-pass filter to the FOS records for removing the heavy traffic loading disturbances). All the results from the numerical simulations and the monitoring data are shown in diagrams in Fig. 10 for comparison. FE results are summarized in detail in Fig. 11.

The true traffic loading is an unknown parameter with high level of randomness. However, as it has been observed from the measurements, only heavy vehicle passages produce valuable effects



(a) (b) (c)

Fig. 11 Detail of peak values of curvatures PSDs for the FE simulations at beam 5 (Periodogram - lines with square points & Welch -lines with circle points). Measurement point 1 - damaged (a), point 2 (b), point 3 (c) in terms of dynamic curvature response. This allows a simplification in the FE analyses: deterministic traffic loading is assumed by considering a standard truck vehicle overpassing the slab in fixed positions. Although this assumption is not able to reproduce closely the bridge quantitative response in terms of real traffic conditions, it is considered as acceptable for qualitative interpretation of the monitored data.

AASHTO LRFD Bridge Design Specification (2010) defines the characteristic of the design truck in terms of six point loads, two front drive wheels (8 kip~35 kN) and four back wheels (32 kip~140 kN). Such concentrated vertical forces are implemented into the FE model by an external user subroutine, which defines vertical nodal forces on a traffic line at a constant velocity (26.6 m/s), that is the real speed limit of the highway viaduct (60 MPH). The forces are constant in intensity along the vertical direction and variable in the position for reproducing the effect of the moving load (truck) on the FE model of the slab.

Several undamaged and damaged configurations of the bridge are investigated by the FE analyses with two traffic lines, respectively over steel girder 2 and girder 5. They correspond to the following last five events in Fig. 10:

- Event 27: undamaged configuration of the bridge with truck passage at beam 2
- Event 28: undamaged configuration of the bridge with truck passage at beam 5
- Event 29: damaged configuration of the bridge with truck passage at beam 5: damage consists of 1 m delamination between the steel beam and the concrete slab, centered at b5p1 (disconnection).
- Event 30: damaged configuration of the bridge with truck passage at beam 5: damage is increased to 3 m delamination between the steel beam and the concrete slab, centered at b5p1 (disconnection).
- Event 31: damaged configuration of the bridge with truck passage at beam 5: damage consists of 0.6 m transversal crack in the concrete slab, centered at b5p1.

The following observations arise: in the undamaged configurations (Events 27 and 28) the different loading show modifications in the output. In the damaged configurations (Events 29-31), only the largest delamination (Event 30) and the transversal cut (Event 31) show slight variations in the peak values of the dynamic curvature PSD at location of simulated damage b5p1 (Fig. 11(a)). No variation is noticed at other locations. Since the FOS measurements in undamaged configuration of the real structure were not available, the numerical model for events 27 and 28 (undamaged) is employed in the analysis. The model outcomes in intact condition show the expected smaller curvature PSD peaks at quarter span than the PSD peaks in the mid-span. Thus, the higher PSD peak values resulting from the FOS measurements at point b5p1 can be indeed related to an unusual behavior at that location. An independent study based on analysis of the location of the neutral axis confirmed the existence of the unusual behavior at that location (Sigurdardottir and Glišić 2013, 2014).

It is important to highlight that a comparison of the FE outcomes (Events 27-31) with the monitoring data (Events 1-26) shows that FE model is able to qualitatively reproduce structural behavior. However, quantitative differences between the data and the model outcomes are present, mainly due to the simplified deterministic nature of the loading conditions used in FE analysis. Nevertheless, the comparison of the FE model response with the sensor data allowed understanding and confirming essential indications on the damage location and extension. The slight modifications in the processed data highlighted by the damaged configurations of the FE model (Events 29-31 in Figs. 10-11) suggests that the source of unusual behavior on the real structure is probably more extended than in the simulated scenarios.

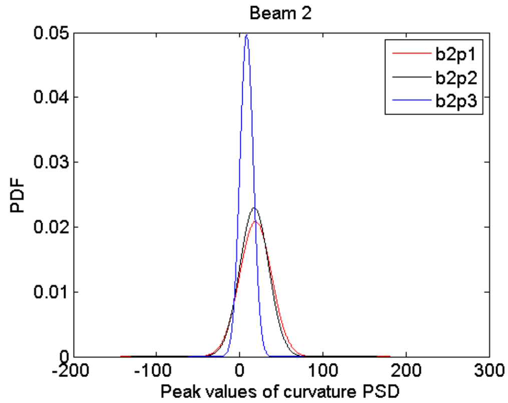


Fig. 12 Gaussian distributions of the dynamic curvature PSD peaks on beam 2

Table 3 Average and standard deviation of PDF in Fig. 12

	b2p1	*b2p2*	*b2p3*
Average	19.3	17.7	8.4
Standard deviation	19.1	17.4	8.0

This last consideration is also corroborated by considering the responses at beam 2: similar asymmetric outcomes with amplification at position b5p1 (Fig. 9) are highlighted also at b2p1 as reported by Fig. 12 and Table 3 in terms of Gaussian distribution of curvature PSD peaks. In other words, the damage extension affects more than the sensor at position b5p1, position b2p1 resulting also perturbed. While more case studies in terms of controlled traffic patterns and damage cases should be considered to fully validate the proposed methodology, this was not carried out due to impossibility to close the busy highway overpass for testing purposes, and impracticality to implement reversible damage to real structure.

3. Conclusions

This work researches damage detection in composite concrete-steel structures that are typical for highway overpasses and bridges by using only structural response variables (*output-only* technique). The method developed in this paper is based on dynamic curvature analysis of real strain data from an in-service structure for real time applications. A FEM approach is also developed in parallel for interpreting the structural behavior and then assessing the effectiveness of the method. The data are acquired from long-gauge fiber optic strain sensors under traffic loading of the structure. The FE mesh is prepared within an available code and processed for modal analyses and subsequent dynamic transient analyses with truck loading lines as well. Probabilistic analysis of the peak values of dynamic curvature PSDs is used to study the real data and compared to FEM. The real data show unusual behavior at one location where the average and standard deviation of the peak values of curvature PSDs are significantly higher than expected. These outcomes are in accordance with a previous study that identified unusual behavior at the same location. The FE model is validated through modal analysis and used to study different damage scenarios at the affected location. The model is sensitive to the location of the load and shows variations in the transient response under more severe damage scenarios. The transient analyses outcomes support the sensors monitoring outcomes for detection of unusual structural behavior. In addition, it suggests that the source of unusual behavior affects area that is larger than the one that is modeled.

In conclusion, the novelty introduced by the present research work consists in a new *output-only* method for detection of unusual structural behaviors, based on direct curvature measurements using long-gauge fiber optic sensors, and statistical analysis of PSD peak values. The principles are qualitatively explained using FE analysis, and the method is successfully validated in real-life settings. Future work will include improved FE modelling in order to enable evaluation of the source (nature) and the magnitude of the unusual behavior.

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