Damage Identification in a Benchmark Cable-Stayed Bridge Using the Interpolation Method

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DAMAGE IDENTIFICATION IN A BENCHMARK CABLE-STAYED BRIDGE USING THE INTERPOLATION METHOD

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ABSTRACT

In this paper the damage localization algorithm based on Operational Deformed Shapes (ODS) and known as Interpolation Damage Detection Method (IDDM), is applied to the numerical model of a cable stayed bridge. Frequency response functions (FRFs) have been calculated basing on the responses of the bridge to low intensity seismic excitation and used to recover the ODS both in the transversal and in the vertical direction. The analysis have been carried in the undamaged configuration and repeated in several different damaged configurations Results show that the method is able to provide the correct location of damage, provided an accurate estimation of the ODSs is available.

KEYWORDS : *damage localization, cable-stayed bridge, noise, interpolation method.*

INTRODUCTION

Long span cable stayed bridges play an important role on the social and economic life and are, by all means, strategic structures. Able to span long distances, they are expected to be able to promise serviceability in different conditions, among the others, after extreme loading events. In this light, damage assessment techniques, can give an important contribution. One of the major problems in the assessment and calibration of analytical methods for damage identification of large civil structures and infrastructures is the scarce availability of data recorded on really damaged structures. To overcome this shortcoming, a detailed finite element model, able to correctly and reliably reproduce the real behavior of the structure under ambient excitation can be an invaluable tool enabling the simulation of several different damage scenarios that can be used to test the performance of any monitoring system. Stay cables are the most critical load bearing elements for cable-stayed bridges thus the monitoring of their damage state is a very important task. In this paper some preliminary results on the application of a method of damage localization known as Interpolation Damage Detection Method (IDDM) are presented with reference to the numerical model of an existent cable stayed bridge subject of an international benchmark.
The authors have recently developed a new finite element model of the benchmark structure addressing new issues in the simulation of the bridge dynamic. The numerical model has been used to simulate the structural response of the structure in the undamaged state, and in several different damage states, under a seismic excitation having the intensity of after-shock events. Earthquake records are the natural accelerograms, as adopted in the original benchmark, applied in a multi-support configuration of the structure. Basing on responses calculated by the finite element model, the operational deformed shapes of the structure have been calculated and used to check the reliability of the IDDM in detecting damage simulated through a reduction of stiffness in a number of stay cables. Several damage scenarios were simulated with different location and severity of damage in order to check the sensitivity of the damage identification method to both the location and the severity of damage.

1. THE DAMAGE LOCALIZATION METHOD

The Interpolation Damage Detection Method (IDDM) has been successfully applied for damage localization of multistory buildings [3, 5], supported bridges [4], suspension bridges [6] and recently extended to the case of two-dimensional structures [7].

Thanks to its formulation based on the detection of reduction of smoothness in the Operational Deformed Shapes (ODS), the IDDM can be applied to any type of structure provided the (ODS) can be estimated accurately in the original and in the damaged configurations and a proper continuous function is used to interpolate the ODS in order to detect possible reductions of smoothness.

The basic idea of the IDDM for beam-like structures can be described with reference to Figure 1.

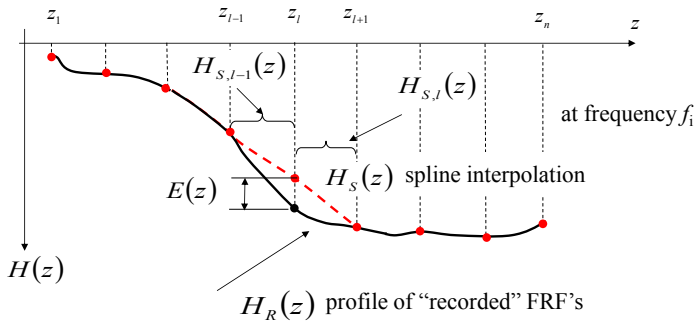


Figure 1: The interpolation error

Let $z_1,\ldots z_n$, be instrumented location of the structure where responses in terms of acceleration have been recorded. At each frequency value, the set of frequency response functions $H_R(z)$ measured at the instrumented locations, give the operational deformed shape (ODS) at that frequency (red dots in Figure 1). At the l-th location z_l the FRF can be calculated through the spline interpolation using the following relationship:

$$H_S(z_l, f) = \sum_{j=0}^{3} c_{j,l}(f)(z_l - z_{l-1})^j \ . \tag{1}$$

where the coefficients (c_{0l}, c_{1l} c_{2l} c_{3l}) are calculated from the values of the transfer functions "recorded" at the other locations:

$$c_{j,l}(f_i) = g(H_R(z_k, f_i)) \qquad k \neq l \tag{1}$$

The explicit expressions of the coefficient of the spline function $c_{j,l}$ in terms of the FRF's are determined imposing continuity of the spline function and of its first and second derivative in the knots (that is at the ends of each subinterval). More details on the spline interpolation procedure to calculate acceleration responses can be found in reference Limongelli (2003). In terms of FRF's the interpolation error at location z (in the following the index l will be dropped for clarity of notation) at the i-th frequency value f_i, is defined as the difference between the magnitudes of recorded and interpolated frequency response functions:

$$E(z, f_i) = \left| H_R(z, f_i) - H_S(z, f_i) \right| \tag{2}$$

In order to characterize each location z with a single error parameter, the norm of the error on the significant frequency range (that is the frequency range with a signal to noise ratio sufficiently high to allow a correct definition of the FRF) is calculated:

$$E(z) = \sqrt{\sum_{i=1}^{N} E(z, f_i)^2} \tag{3}$$

The significant frequency range is selected limiting the summation in equation (3) to the frequency range of the fundamental modes of the structure. This frequency range can be tuned basing on vibration tests carried out on the undamaged structure.

If a reduction of stiffness (damage) occurs at a certain location, the operational shapes change in the region close to that location and specifically their smoothness decreases due to the discontinuity of curvature induced by damage. If the estimation of the error function through Eq. (3) is repeated in the baseline (undamaged) and in the inspection (possibly damaged) configuration, the difference $\Delta E(z)$ between the two values, denoted respectively by $E_0(z)$ and $E_d(z)$, can provide an indication about the existence of degradation at location z. An increase ($\Delta E(z) > 0$) of the interpolation error between the reference configuration and the current configuration at a station z, i.e., highlights a localized reduction of smoothness and therefore, it is assumed to be a symptom of a local decrease of stiffness at location z associated with the occurrence of damage. Basing on this assumption the following conditions will be assumed to define the damage index $IDI(z)$:

$$\begin{aligned} IDI(z) &= \Delta E \qquad \text{if} \qquad \Delta E(z) \geq 0 \\ IDI(z) &= 0 \qquad \text{if} \qquad \Delta E(z) < 0 \end{aligned} \tag{4}$$

In order to remove the effect of random variations of ΔE and assuming a Normal distribution of this function, the 98% percentile is assumed as a minimum value beyond which no damage is considered at that location. In other words a given location is considered close to a damaged portion of the structure if the variation of the interpolation error exceeds the threshold calculated in terms of the mean $\mu_{\Delta E}$ and variance e $\sigma_{\Delta E}$ of the damage parameter ΔE on the population of available records that is:

$$\Delta E(z) > \mu_{\Delta E} + 2\sigma_{\Delta E} \tag{5}$$

The damage index is then defined by the relation:

$$IDI(z) = \Delta E(z) - (\mu_{\Delta E} + 2\sigma_{\Delta E}) \tag{6}$$

2. THE BILL EMERSON MEMORIAL BRIDGE

This bridge at the base of this study is a fan-type cable stayed bridge (Figure 1) which crosses the Mississippi River near Cape Girardeau (USA) with a composite concrete-steel deck stiffened by two longitudinal steel girders (Figure 2).

Figure 2: The Bill Emerson Memorial Bridge (Framerotblues, 2007; with permission)

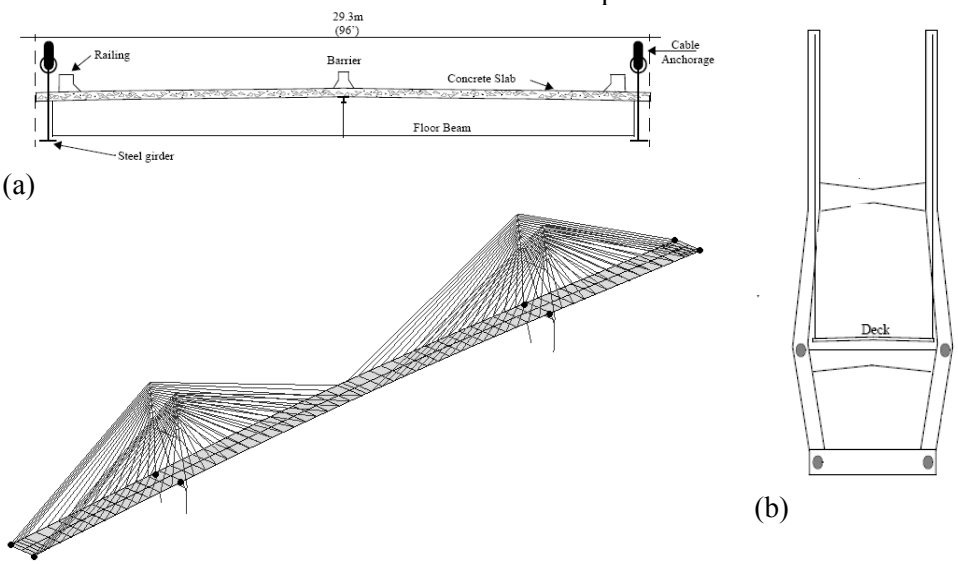
The bridge is 1206 m long with a main span length of 350.6 m. One hundred and twenty eight stays, made of high-strength, low-relaxation steel, are arranged according to a fan-type distribution. The smallest cable area is 28.5 cm^2 and the largest cable area is 76.3 cm^2. The deck is supported by two towers in the cable-stayed spans. Twelve additional piers support the Illinois approach spans. Each tower has a solid section below the cap beam, and a hollow section in the upper portion

(Figure 3). For the out-of-plane behavior, the upper portion of the towers above the cap beams remains nearly elastic with a significant margin of safety. The lower portion of the towers, however, likely experiences moderate yielding out of plane during the design earthquake though the safety of the bridge is not a concern. The in-plane behavior of the two towers is always in the elastic range under the design earthquake, with a large margin of safety. For a more detailed description of the structure, as well as of its members, the reader is referred to [2].

3. THE NUMERICAL MODEL OF THE BRIDGE

This bridge was the subject of a well-known benchmark on bridge control [2]. The model of the cable-stayed bridge is set-up [8, 9, 11] in the ANSYS multipurpose finite element framework, with some enhancements with respect to the original model distributed along with the benchmark files. Firstly, the rendition of the numerical model adopted in this study [10, 11] comprises soil-structure interaction through the use of impedance functions and lumped masses, springs and dampers acting in the vertical, transversal and longitudinal direction at each foundation (bents and piers). Furthermore the modelling of cables has been enhanced moving from a single rod type representation (also called a one-element cable system) to a description with six rope elements for each cable enabling an improved modelling of the stays-deck coupled response. The non-linearity between deformations and displacements is also accounted for by evaluating the dynamic equilibrium of the structure at any instant in the deformed configuration.

The resulting finite element mesh in ANSYS [1] comprises (Figure 4) linear beam elements for towers and the deck frame, linear shells elements for the concrete deck slab, tension only elements for the stay cables , for a total number of about 2600 nodes and 2800 elements. The materials are characterized as linear elastic. High performance concrete is adopted for the piers (E= 50 GPa); high-strength, low-relaxation steel for the stay cables (E= 210 GPa). The mixed structure of the deck (steel frame with concrete slab) is modeled by concrete shell elements connected to steel beams. The two materials retain the specified characteristics. A structural damping equal to 3% of the critical one is assigned to the bridge model as a Rayleigh type damping computed between the first (0.28s) and the sixth (0.64s) mode, ensuring reasonable values of the damping ratios for the modes which contribute the most to the seismic response.



c)

Figure 3: a) Deck cross-section. b) Towers'elevation c) FEM model.

The 3-D response and behavior of the cable-stayed bridge model is such that the vibration modes are characterized by coupled shapes. The dynamic characteristics of the bridge also indicate that the cable-stayed structure is more flexible in the vertical direction and less so in the longitudinal direction.

In order to verify the feasibility of the IDDM for this type of structure, the Frequency Response Functions must be calculated to obtain the Operational Deformed Shapes. To this aim any type of known excitation could be applied in order to calculate the FRFs.

Herein a seismic type excitation is applied at the support of the bridge (base of towers and bents) in a multi-support configuration, accounting for a time delay due to wave propagation. The signal recorded during the Gebze earthquake (recorded at the Gebze Tubitak Marmara Arastirma Merkezi on Aug. 17, 1999) scaled to a peak accekeration of 0.02g was used as input [9]. The scaling of the input to a low value of the peak acceleration is meant to simulate the acquisition of information from responses induced by after-shocks not likely to induce (additional) damage to the structure or to induce strong non-linear behavior of the structure and of the dissipative control devices, thus keeping the structural response in the linear range.

4. SIMULATED DAMAGE SCENARIOS

Stay cables are the key components in cable-stayed bridges bearing most of the weight of deck hence the prompt identification of damage in these structural elements is of paramount importance for a proper post-event strategy of intervention and maintenance. Due to the large number of stay cables in one cable stayed bridge, a monitoring technique able to give indication about the location of a damaged stay cable without requiring the placement of sensors on each cable would allow the optimization of the monitoring system reducing the costs.

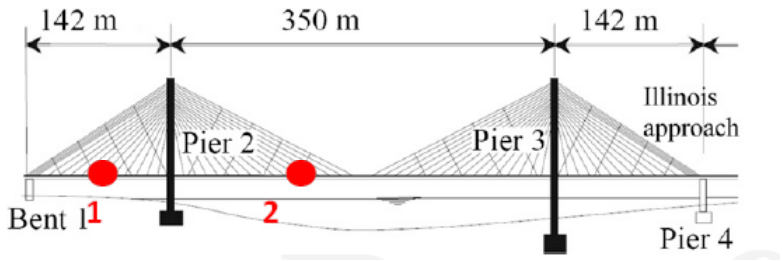


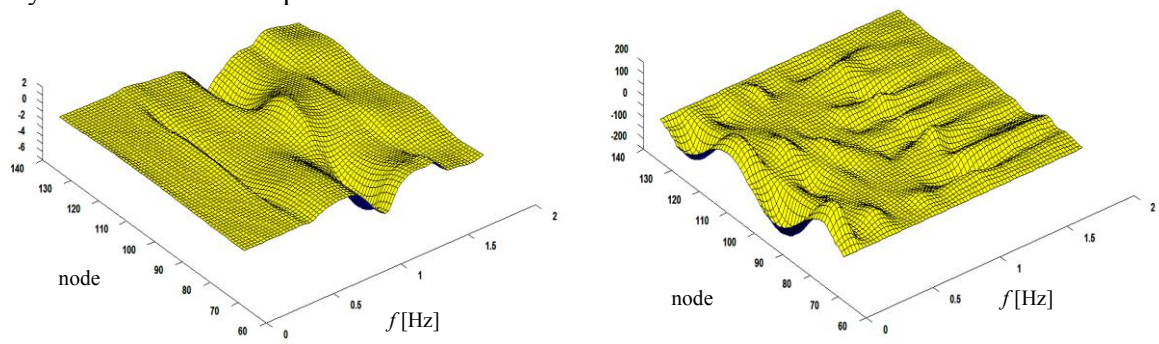
Figure 4: Damage scenarios

Damage to cable stays is one of the most difficult to identify due to its local character that needs the identification and the analysis of higher modes, usually the most difficult to detect reliably. In this paper in order to check the sensitivity of the IDDM the method is tested against this very challenging task of damage detection in stays. Damage has been simulated by reducing the transversal section of 3 adjacent stays of 10%, 25% and 50% of the original section. Two different damage locations have been considered (see Figure 4): position 1 is located at half span between Bent 1 and Pier 2; position 2 is placed between 1/4 and 1/3 of the central span, close to Pier 2. The name of each damage scenario indicates the location (1 or 2) of the damaged stays and the amount of section reduction. For example scenario C1_10 corresponds to a 10% reduction of the transversal section of three stays at location 1. Both single (only one damaged location) and multiple (two damaged locations) damage scenarios have been considered.

5. DISCUSSION OF RESULTS

Responses calculated by the finite element model have been used to check the reliability of the IDDM in locating the damaged portion of the bridge. The operational deformed shapes of the deck in the transversal and in the vertical directions of the bridge are reported respectively in Figure 5 limited to the frequency range 0-2Hz. The ODSs have been obtained from the Frequency Response

Functions calculated from the responses at the nodes of the deck in the transversal and in the vertical direction. In order to give a measure of the severity of damage related to the considered scenarios, in columns 3 to 11 of Table 1, are reported the percentage variations $\Delta f=(f-f_o)/f_o$ of the frequencies with respect to their value fo in the undamaged configuration, of the first 10 modes that mostly contribute to the response in the vertical or transversal direction.



Transversal Vertical

Figure 5: Operational deformed shapes of the bridge deck for frequencies in the range 0-2 Hz.

The highest variation is found for the most severe scenario (C1_2_50) corresponding to a reduction of 50% of stiffness in 6 cables of the bridge. In this case a variation of 0.61% of the modal frequency of the 10th mode is found. These variations of frequency are very low and would hardly allow the detection of damage, not to consider that in a real case noise would affect the estimation of modal parameters thus completely hampering the identification of damage through the estimated values of modal frequencies.

Table 1: Percentage variation of modal frequencies.

M	f_o[Hz]	C1_10	C1_25	C1_50	C2_10	C2_25	C2_50	C12_10	C12_25	C12_50	dir
1	0.27	0.00	-0.01	-0.03	0.00	-0.01	-0.04	-0.01	-0.02	-0.06	V
3	0.40	0.00	0.00	-0.01	-0.01	-0.03	-0.08	-0.01	-0.03	-0.08	T
4	0.47	0.00	0.00	0.00	0.00	-0.01	-0.03	0.00	-0.01	-0.03	T
6	0.60	-0.01	-0.04	-0.13	-0.01	-0.02	-0.05	-0.02	-0.06	-0.17	V
10	0.73	-0.01	-0.04	-0.08	-0.06	-0.18	-0.42	-0.08	-0.23	-0.61	V
16	0.97	0.00	-0.01	-0.02	-0.01	-0.02	-0.06	-0.01	-0.03	-0.09	V
18	1.03	-0.07	-0.18	-0.37	0.00	-0.01	-0.03	-0.07	-0.18	-0.39	V
19	1.10	0.00	-0.01	-0.04	-0.01	-0.02	-0.04	-0.01	-0.03	-0.08	T
20	1.13	0.00	-0.01	-0.04	0.00	-0.01	-0.02	-0.01	-0.02	-0.08	T
23	1.16	-0.04	-0.11	-0.23	0.00	0.00	0.00	-0.04	-0.11	-0.23	T

On the contrary, under the same assumptions, that is neglecting the effect of noise in recorded data, the IDDM allows both the detection and the localization of damage for the all the considered damage scenarios. The IDDM has been applied using responses in both the transversal and the vertical direction for all the considered damage scenarios but due to space limitation only a selection of results is reported herein. Figure 6 reports the results relevant to the 'worst' cases from a damage detection point of view that is the ones corresponding to the lower severity of damage and specifically scenarios C1_10, C2_10 and C12_10.

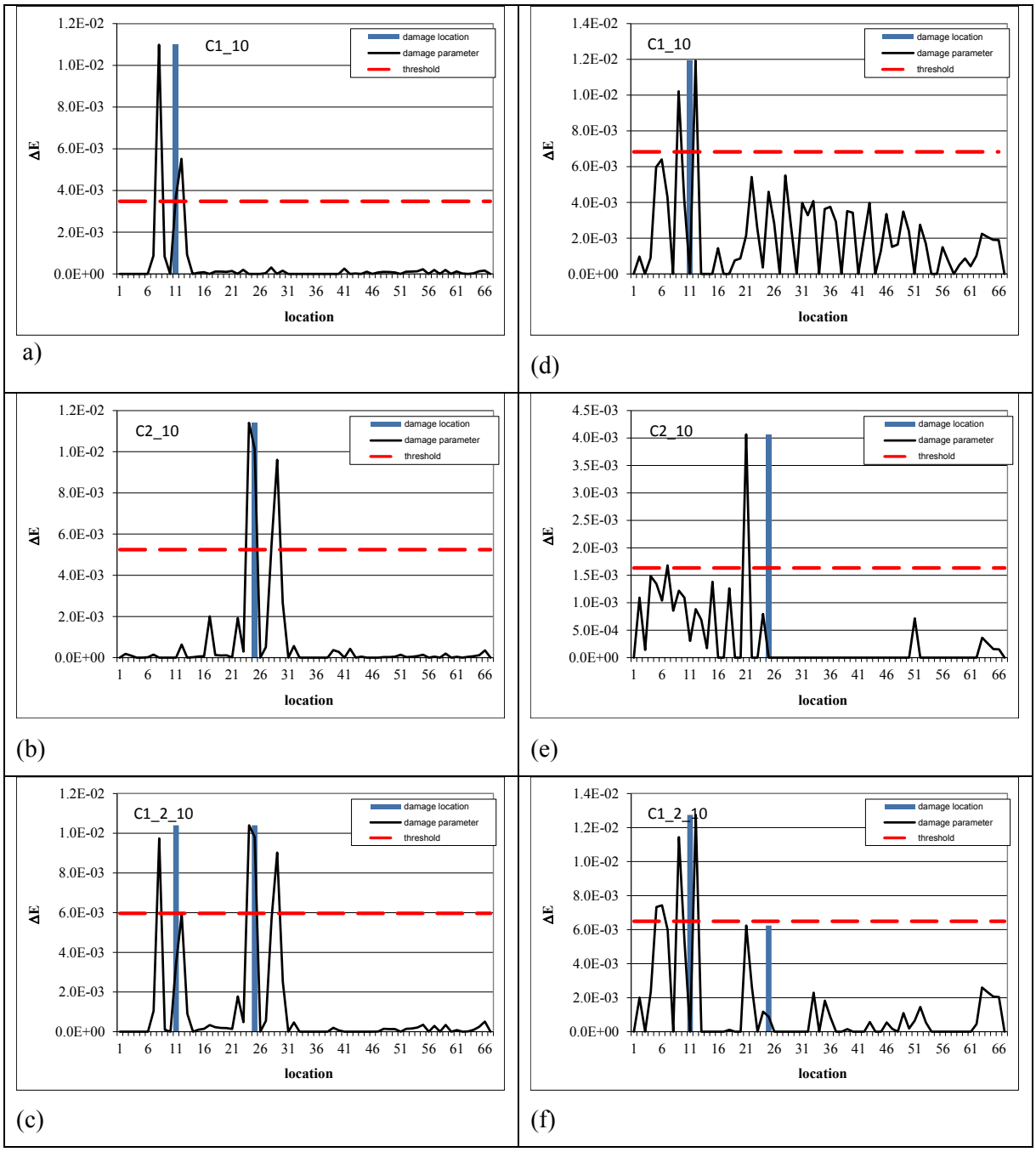


Figure 6. Damage parameter and threshold for the three damage scenarios C1_10, C2_10, C12_10. a) b) c) FRF in the transversal direction d) e) f) TF in the vertical direction.

The values of the damage parameters ΔE, calculated basing on FRF recovered from transversal (T) are reported in Figure 6 a,b,c. The IDDM has been applied also using the transmissibility functions of the vertical responses at the nodes with respect to the vertical response measured at a reference node. As reference has been assumed the node located on the section of the deck corresponding to Pier2. Results are reported in Figure 6a to 6f for scenarios C1_10, C2_10 and C1_2_10. In the figures a blue vertical bar indicates the actual location of damage assumed at the node joining the deck with the damaged stay. The red dotted bar represents the threshold corresponding to the 98% percentile of the damage parameter distribution. This threshold defines the minimum value that the damage parameter $\Delta E(z)$ has to reach in order to tag location z as "close

to a damaged portion of the structure". In all cases, even if damage is very low (10% reduction of transversal section) the damaged section is correctly identified. Of course the method is not able to indicate if the damage is located in the deck or in the cables, since only responses on the deck were considered in the procedure, but the damaged portion of the structure is correctly identified. The procedure shows a good accuracy and reliability being able to correctly locate damage location in all the considered cases. Results obtained using FRFs calculated with respect to the base input shown a higher degree of reliability with respect to those calculated basing on transmissibility functions. In the second case function ΔE presents values greater than zero at several non-damaged nodes and this, for the case C12_10 hampers damage detection at location 2 if the percentile of 98% is considered.

6. CONCLUSIONS

The Interpolation Damage Detection Method was applied to detect damage in stays using the numerical model of the Bill Emerson Memorial cable stayed bridge. Results show that, if the Frequency Response functions can be accurately estimated, the method is successful in detecting small and localized damages. This result can be accomplished using both Frequency Response Functions of the responses calculated with respect to the base input, both using Transmissibility Functions calculated with respect to the response at a reference node. In the first case a higher reliability of results is obtained: the actual location of damage is correctly identified in all cases and also for multiple damaged locations.

REFERENCES

[1] ANSYS. (2011). Academic release 12 user manual, ANSYS, Canonsburg, PA.

[2] Caicedo, J. M., Dyke, S. J., Moon, S. J., Bergman, L. A., Turan, G., and Hague, S. (2003). "Phase II benchmark control problem for seismic response of cable-stayed bridges." J. Struct. Control, 10(3–4), 137–168.

[3] Limongelli, M.P., (2014). Seismic health monitoring of an instrumented multistorey building using the Interpolation Method. Earthquake Engng. Struct. Dyn. doi: 10.1002/eqe.2411.

[4] Limongelli M.P. (2010). "Frequency response function interpolation for damage detection under changing environment". MSSP doi:10.1016/j.ymssp.2010.03.004

[5] Limongelli, M.P., (2011). The interpolation damage detection method for frames under seismic excitation. Journal of Sound and Vibration, 330. 5474–5489.

[6] Domaneschi M., Limongelli M.P., Martinelli L., (2012). "Damage detection in a suspension bridge model using the Interpolation Damage Detection Method" Bridge Maintenance, Safety, Management, Resilience and Sustainability – Biondini & Frangopol (Eds) © 2012 Taylor & Francis Group, London, ISBN 978-0-415-62124-3.

[7] Limongelli M.P. (2014). Two dimensional damage localization using the interpolation method. Submitted to *Journal of Sound and Vibrations*.

[8] M. Domaneschi, L. Martinelli (2012), "Performance Comparison of Passive Control Schemes for the Numerically Improved ASCE Cable-Stayed Bridge Model", Earthquakes and Structures, 3(2):181-201. Techno Press. ISSN: 2092-7614 (Print), ISSN: 2092-7622 (Online).

[9] M. Domaneschi, L. Martinelli (2014), "Deepening the ASCE bridge benchmark: transversal response under seismic loading", Journal of Bridge Engineering ASCE, 19 (3), art. no. 4013003. DOI: 10.1061/(ASCE)BE.1943-5592.0000532

[10] M. Domaneschi, M.P. Limongelli, L. Martinelli (2013), "Vibration Based Damage Localization Using MEMS on a Suspension Bridge Mod2012el", Smart Structures and Systems, 12(6), 679-694.

[11] M. Ismail, J. Rodellar, G. Carusone, M. Domaneschi, L. Martinelli (2013), "Characterization, modeling and assessment of Roll-N-Cage isolator using the cable-stayed bridge benchmark", Acta Mechanica, 224, 525–547.