Structural Health Monitoring of Reinforced Concrete Structures using Nonlinear Interferometric Analysis

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SUMMARY:
Structural Health Monitoring (SHM) aims to improve knowledge of the safety and maintainability of civil structures and infrastructures. Within the Italian research project RELUIS-DPC 2010-2013, funded by the Italian Department of Civil Protection, a specific task deals with the possibility of set up a fast procedure to determine the damage evolution on a large number of structures after seismic events. This paper presents an upgrade of a method for damage detection based on a statistical approach that uses the most significant data recorded on both the top floor and the bottom of a building, with the purpose of extracting the value of the maximum inter-story drift expected along the building height, adopted as damage indicator. Nonlinear interferometric analyses combined with the S-Transform are used to evaluate frequencies and damping variation of the monitored structure during an earthquake. The method has been tested on numerical reinforced concrete framed structures.

Keywords: Interferometric Analyses, S-Transform, Structural Health Monitoring, Nonlinear Dynamics, Damping Evaluation

1. INTRODUCTION

The assessment of an increasing number of aged structures and infrastructures requires a huge effort, especially if the purpose is to provide a faithful evaluation of seismic risk. The current practice of periodic visual inspections, for the safety evaluation appears more and more inadequate. A specific task of the Italian research RELUIS-DPC 2010-2013 project, funded by the Department of Italian Civil Protection (DPC), deals with devising and implementing a fast procedure (Ponzo et al., 2010) to get useful information about the damage evolution in a large number of strategic buildings during and after seismic events, using the records of just few sensors located on the structure. The feasibility and the cost optimization are the most important goals for the simplified monitoring system in order to favour a widespread use of such systems. During the past two decades, a significant amount of researches have been carried out in the field of Non-destructive Damage Evaluation (NDE) methods using the changes in the dynamic response of a structure (Chen et al., 1995; Capecchi and Vestroni, 1999; Ponzo et al., 2010). The NDE methods can be classified into four levels (Stubbs et al., 2000), according to the specificity of the information provided by a given approach (Rytter, 1993): (i) Level I methods, i.e. those methods that only identify if damage has occurred. (ii) Level II methods, i.e. those methods that identify if damage has occurred and simultaneously determine the location of damage. (iii) Level III methods, i.e. those methods that identify if damage has occurred, determine the location of damage as well as estimate the severity of damage. (iv) Level IV methods, i.e. those methods that identify if damage has occurred, determine the location of damage, estimate the severity of damage and evaluate the impact of damage on the structure. Each level of damage identification described above requires a gradual increasing amount of data and more complex algorithms. Consequently, their set-up and effectiveness often require increasing costs, with higher error probability. Ponzo et al. (2010) proposed an innovative approach for the simplified structural damage detection thanks to which it is possible to obtain information about the state of the health of the monitored structure few minutes after a seismic event. The methodology is based on the acquisition of the structural dynamic response by mean a three-directional accelerometer installed on the top floor of the structure. From the

accelerometric recording maximum top acceleration, frequency variation and equivalent viscous damping variation are extracted and combined to obtain an estimation of the maximum inter-storey drift.
After a brief description of the method proposed by Ponzo et al. (2010), this paper focuses on the interferometric analyses (Snieder and Safak, 2006; Picozzi et al., 2011) useful to obtain the dynamic response of the monitored structure. Particularly, the Impulse Response Function (IMF) obtained by mean the interferometric analysis, applied on the data recorded on the monitored structure, is combined with the S-Transform (Stockwell et al., 1996) to perform a pseudo time-frequency analysis with the aim to automatize the procedure to evaluate both frequency and damping variation during earthquakes.

2. A SIMPLIFIED METHOD FOR STRUCTURAL DAMAGE DETECTION

The method proposed by Ponzo et al. (2010) starts from a limited number of records acquired on top floor (acceleration) and overcomes some limitations of the Level I methods. Indeed, the method considers some parameters evaluated by the recorded signals: (i) Maximum Absolute Top Acceleration (MATA); (ii) variations of the fundamental frequency (iii) variation of the equivalent viscous damping, and provides a combination of these parameters to estimate the maximum inter-storey drift by means an empirical relationship (Eqn. 2.1). All signals are filtered with band-pass filter centred on the fundamental frequency of the monitored structure.

$$\Delta_{an} = c_1 \cdot a_{\max}^2 + c_2 \cdot a_{\max} + c_3 \cdot \Delta f_1^{\ 2} + c_4 \cdot \Delta f_1 + c_5 \cdot \Delta f_2^{\ 2} + c_6 \cdot \Delta f_2 + c_7 \cdot \Delta\xi^2 + c_8 \cdot \Delta\xi \quad (2.1)$$

Where $c_1,\dots,c_8$ coefficients are related to the mechanical and geometrical characteristics of the monitored structure. The contribution of the weight of each single instrumental parameter on the maximum analytical inter-storey drift is estimated by the following expression:

$$W_i = \frac{|F_i|}{\sum_i |F_i|} \quad (2.2)$$

The MATA represents the first instrumental parameter considered in this method. It can be evaluated directly by the filtered signal (acceleration) recorded on the top floor of the building. An appropriate arrangement of recording sensors on the structure permits to reconstruct all displacement and rotation components of the floor. The other two instrumental parameters considered in the method are i) the percent variations (Δf_1) between the fundamental frequency of the building before the seismic event f_{init} and the minimum one f_{min}, corresponding to the maximum non linear behaviour of the building and ii) the percent variations (Δf_2) between initial and final frequency (f_{fin}), as given by equations 2.3 and 2.4.

$$\Delta f_1 = (f_{init} - f_{\min}) / f_{init} \quad (2.3)$$

$$\Delta f_2 = (f_{init} - f_{fin}) / f_{init} \quad (2.4)$$

In the method proposed by Ponzo et al. (2010) all frequencies were evaluated by using a Short Time Fourier Transform (STFT) (Gabor, 1946) applied on the whole signal. This kind of analysis allows describing the main Eigen frequencies variation over time.
The last instrumental parameter considered in the method is the variation of equivalent structural viscous damping $\Delta\xi$ related to the first structural mode. Information about damping can enrich the quality and quantity of the knowledge on the global damage, particularly if the damping is associated to the other parameters above described. For non stationary signal the damping can be estimated using

the only output non-parametric technique elaborated by Mucciarelli and Gallipoli (2007). It measures the viscous equivalent damping of the signal recorded using a semi-probabilistic approach. Several applications of the method have been done on both numerical and real structures (Ponzo et al., 2010). In the following it is possible to observe the results obtained applying the method proposed by Ponzo et al. (2010) to a numerical reinforced concrete framed structure. The considered structure is depicted in Figure 1.

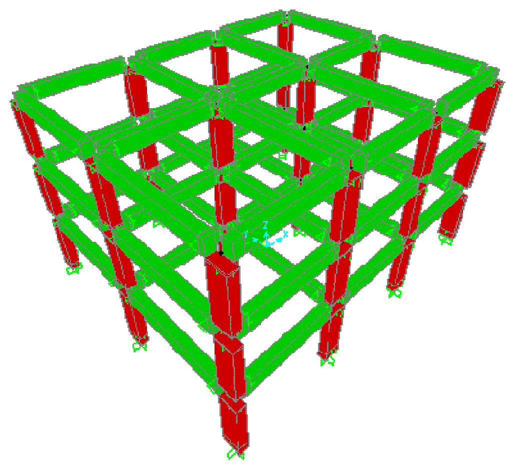


Figure 1. Nonlinear numerical model considered for the analyses

The structural typologies considered in non-linear dynamic simulations refer to framed structural systems. The selected types have 3 storeys, with an inter-storey height equal to 3m, representative of Italian standard buildings. The building has a rectangular plan shape with 12×15m global dimensions. The considered structure has been designed following the criteria of the Italian seismic code (OPCM 3431/2005) for high ductility class (CDA), high seismic intensity (PGA 0.35g) and for soft soil type D. In order to take into account the presence of infill panels within the structural R/C frames and their interaction with the columns, both the masonry strength and stiffness contribution have been considered (Dolce et al., 2004) by inserting two equivalent structural elements in the models. The mechanical characteristics of these elements were evaluated considering the Mainstone model (Mainstone, 1974).

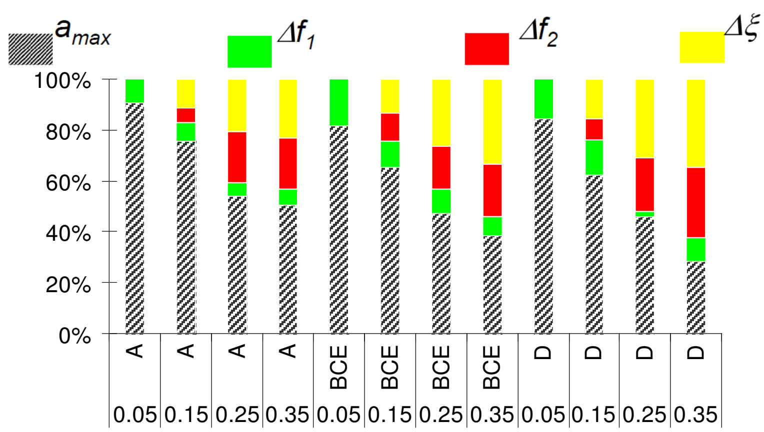


Figure 2. Analyses of weight factors W_i for different soil types

From Figure 2 shows that for low intensities of PGA values the most important parameter for determining the maximum interstorey drift is represented by the maximum top acceleration. This

structure exhibits a quasi-linear variation of relative importance of each parameter ($W_i = f\ (a_{max}, \Delta f_1, \Delta f_2, \Delta\xi)$) with PGA intensities increasing, that highlights the importance of other parameters, namely Δf_1, Δf_2, $\Delta\xi$, when the behavior of the structure becomes nonlinear. This particular behavior can be considered almost invariant for all soil types considered in the analyses.
The results showed in Figure 3 highlight the capability of the simplified method proposed by Ponzo et al. (2010) to estimate the maximum inter-storey drift of a reinforced concrete framed structure during a seismic event.

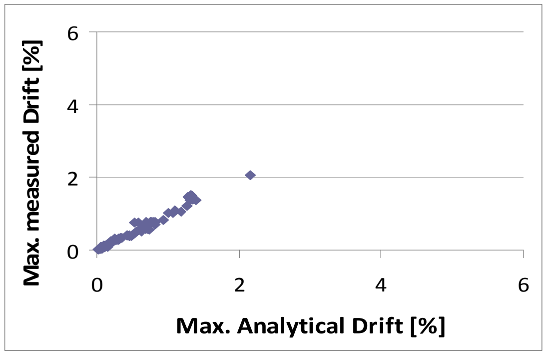


Figure 3. Comparison between maximum analytical and measured inter-storey drift

3. STRUCTURAL EIGENFREQUENCIES AND DAMPING FACTOR

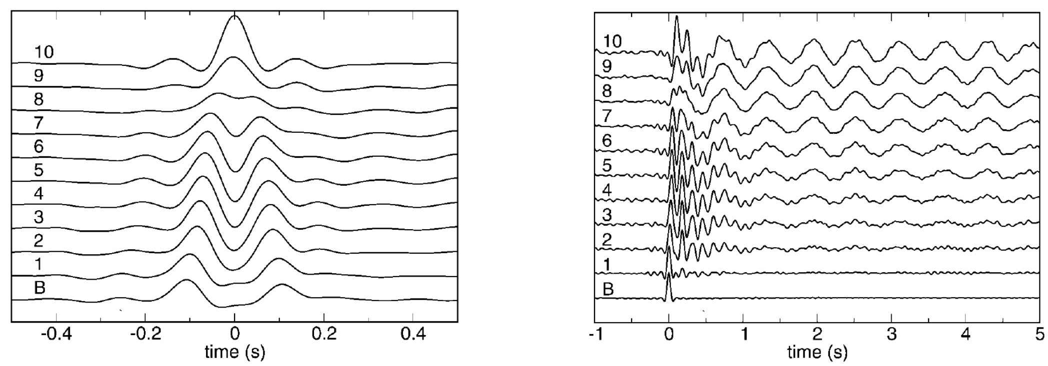
As discussed in the previous section, the methodology proposed by Ponzo et al. (2010) is based on the evaluation of the variation of both structural fundamental frequency and equivalent viscous damping factor during a seismic event. In this section will be proposed a combined approach to automatic estimate structural fundamental frequency and damping factor, related to the main mode of vibration, by using both interferometric analysis (Snieder and Safak, 2006; Todorovska and Trifunak, 2008; Picozzi et al., 2011), S-Transform (Stockwell et al., 1996) and decrement logarithmic method (Inman, 2008).
The building response is extracted by deconvolving the accelerograms recorded at different floors either with respect to the waveform from the top floor or from the base of the building (Snieder and Şafak, 2006). Given the inevitable presence of noise and the limited bandwidth of the recorded signals, a straightforward application of the deconvolution operation, that is to say a simple spectral ratio of the two signals $u_1(\omega)$ and $u_2(\omega)$, provides unstable results. For this reason, in practice the deconvolution is always supported by some form of regularization. Therefore, following Snieder and Şafak (2006), it has been adopted, among other possible approaches (see also Parolai et al., 2009 and Bindi et al., 2010), the Tikhonov Regularization (see Bertero and Boccacci, 1998, and references therein). This regularization method is simple, and no other particular advantages provided by other approaches (e.g., constraint deconvolution) are necessary. Essentially, it consists of taking into account a penalty term that is proportional to the energy of the object (Engl et al., 1996). In practice, given the Fourier spectra of two signals $u_1(\omega)$ and $u_2(\omega)$, by using the Tikhonov regularization, an estimate of the deconvolution is given by the Eqn. 3.1.

$$H(\omega) = \frac{u_1(\omega)^* \cdot u_2(\omega)}{|u_2(\omega)|^2 + \varepsilon} \quad (3.1)$$

where the star denotes the complex conjugate, and ε is a positive number, termed the regularization parameter. The value of the regularization parameter ε determines the degree of filtering applied to the solution: the smoothness of the solution increases with the increased values of ε, reducing the effect of numerical instabilities but also decreasing the spatial resolution of the solution. Assuming that the building behaves, over the entire duration, or a portion, of the earthquake shaking as a linear time invariant system, $H(\omega)$ represents the system transfer function in the frequency domain. Therefore, by computing the inverse Fourier transform of $H(\omega)$, it is possible to represent the system function in the time domain (i.e., the defined IRF),

$$h(t) = F^{-1}(H(\omega)) \quad (3.2)$$

Where F^{1} denote the inverse Fourier transform. It is worth noting that using the top floor as a reference station it is possible to retrieve information about the wavefield propagated into the building while using the bottom floor as a reference station it is possible to extract the impulse response function of the building as shown in Figure 4.



(a) (b)

Figure 4. (a) Results obtained by using the station located at the top floor as a reference station (b) Impulse Response Function obtained by using the bottom floor as a reference station (from Snieder and Safak (2006))

It is clear, as discussed in Snieder and Safak (2006) and Picozzi et al. (2011), that from the impulse response function it is very easy to retrieve information about the equivalent viscous damping factor applying the logarithm decrement method directly on the waveform at the top floor. If higher modes contamine the recording it is possible to extract information about the fundamental mode by using a band-pass filter centered on the fundamental frequency of the monitored building.

4. AUTOMATIC DAMPING AND FREQUENCY EVALUATION

In order to explain the algorithm proposed for the automatic frequency and damping evaluation one example of application will be shown. The structural typologies considered in the non-linear dynamic simulations refer to framed structural systems. The selected types have 5 storeys, with an inter-storey height equal to 3m, representative of Italian standard buildings. The building has a rectangular plan shape with 12×15m global dimensions. The considered structure has been designed following the criteria of the Italian seismic code (OPCM 3431/2005) for high ductility class (CDA), high seismic intensity (PGA 0.35g) and for soft soil type D. In order to take into account the presence of infill panels within the structural R/C frames and their interaction with the columns, both the masonry strength and stiffness contribution have been considered (Dolce et al., 2004) by inserting two equivalent structural elements in the models. The mechanical characteristics of these elements were evaluated considering the Mainstone model (Mainstone, 1974). Input used for the numerical analysis has been able to cause a nonlinear behaviour of the structure during the strong motion phase, and a linear behaviour during the weak motion phase. At this purpose, in order to simulate real recordings, 20s of white noise has been added before and after the considered earthquake. Figure 5 show both the model and the input used for the numerical analysis.

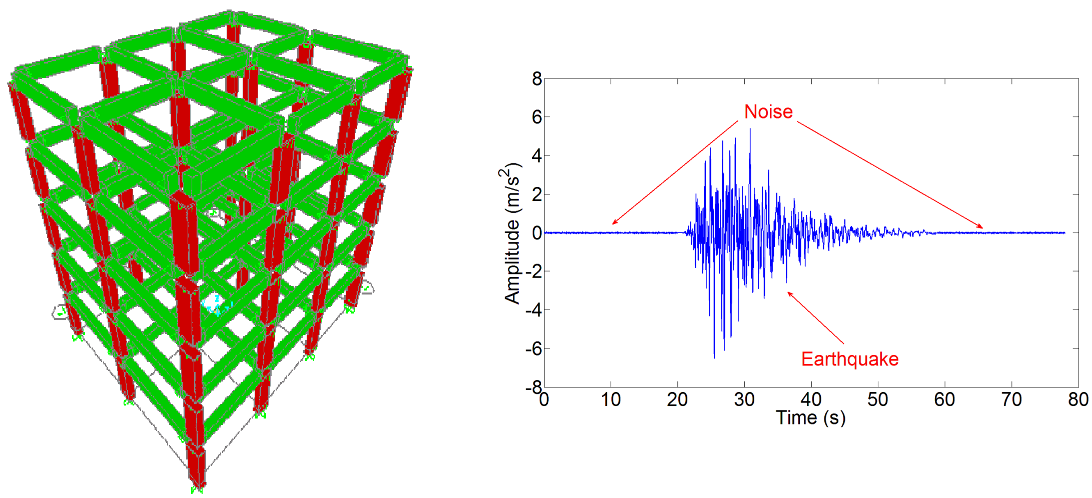


Figure 5. (Left) Numerical Model (Right) Input used for the analyses

It is clear that damping factor can be evaluated before and after the earthquake during ambient vibration acquisition using the impulse response function. Figure 6 shows an example of impulse response function evaluated for the considered framed structure using the bottom floor as a reference station. From the IRF it is directly possible to apply the logarithmic decrement method and evaluate the equivalent viscous damping factor. Using this kind of approach damping factor can be automatically evaluated on noise recordings before and after an earthquake.

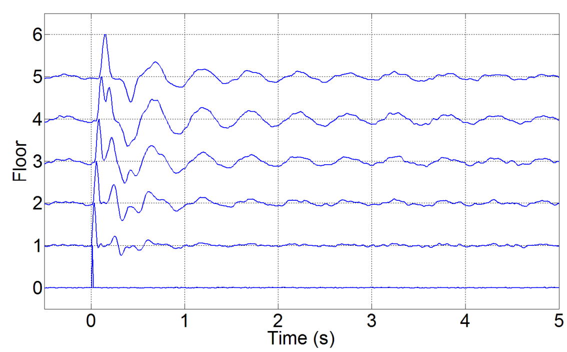


Figure 6. IRFs evaluated using the bottom floor as a reference station

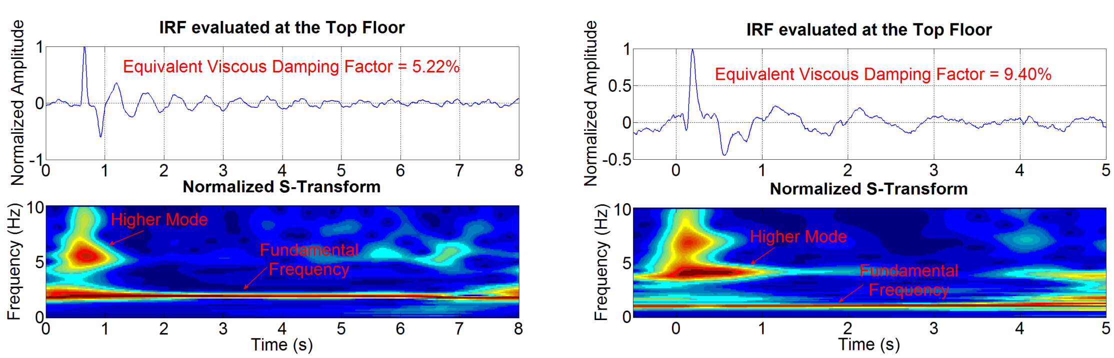


Figure 7. (Left) IRF evaluated before the earthquake at the top floor and related S-Transform (Right) IRF evaluated after the earthquake at the top floor and related S-Transform

It is worth noting that from the IRFs evaluated at the top floor (Figure 7) it is also possible to extract interesting information related to the fundamental frequency of the structure. In fact, comparing the results obtained from the fundamental frequency of the structure before and after the earthquake it is possible to note a shift of the frequency from 2.0Hz (before the earthquake) to 1.10Hz (after the earthquake). Applying the logarithm decrement method on the evaluated IRFs before and after the earthquake it is possible to note a shift of the equivalent viscous damping factor from 5.22% (before the earthquake) to 9.40% (after the earthquake).

Picozzi et al. (2011) shown that it is possible to evaluate the IRF also from a windowed signal acting on a single time-window. In this paper it is shown the possibility to evaluate the time-varying behaviour of the structure in terms of frequency variation by using both the IRFs evaluated on the windowed signal and the related S-Transform. In fact, taking into account the capability of the S-Transform to evaluate the local spectrum it is possible to evaluate the frequency content of each single IRF evaluated from the windowing of the signal recorded at the top floor of the structure where the fundamental mode is very evident.

As discussed in the previous section, the method proposed by Ponzo et al. (2010) is based also on the frequency evaluation before, during and after an earthquake. In the method, a partially solved problem is the possibility to automatic evaluation of the fundamental frequency changes during the strong motion phase. Here a new approach for the automatic evaluation of the fundamental frequency over it will be shown.

It is necessary to consider that initial fundamental frequency is constant if structural behaviour is linear during an earthquake. It could be only lower if the structure exhibits a nonlinear behaviour during earthquake. Then identifying the elastic fundamental frequency it is possible to establish an upper bound for the frequency domain where the fundamental frequency is contained (Figure 8).

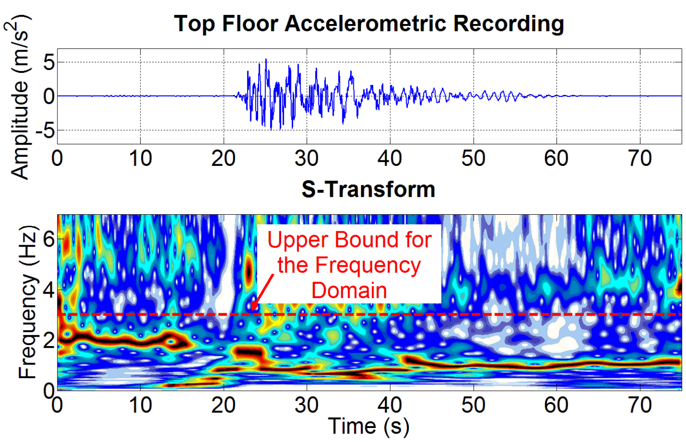


Figure 8. Top Floor accelerometric recording and related S-Transform

When the frequency domain upper bound is established, just using ambient noise recordings, it is possible to use a limited domain for the interferometric analyses and for the S-Transform of the IRFs evaluated at the top level (using the windowed signal). At this purpose it is important to decide how is the length of the selected moving time-window and the related overlap. Generally, the time-window length is fixed as a function of the fundamental period of the structure (Eqn. 4.1):

$$w \geq 10 \cdot T \quad (4.1)$$

where w (in seconds) is the moving time-window length and T is the fundamental period of the monitored structure. With regards to the moving time-window overlap, basing on the results obtained in this work, a good rule seems to be 50% of the considered time-window length.

In the following it is possible to find an example of application of the proposed procedure to automatic evaluate the fundamental frequency variation of the structure before, after and during the earthquake. The elastic starting frequency of the structure, as mentioned before, is equal to 2.0Hz with a period equal to 0.5sec. Using the rule established before it is necessary to use a moving time-window length greater than 5sec. It is worth noting that during the earthquake, if the structure exhibits a nonlinear

behaviour, the fundamental frequency decreases with an elongation of the fundamental period. Then, in order to consider this kind of phenomenon, it is important to use a moving time-window length greater than 5sec. In this case-study a length equal to 15sec has been used.
Figure 9 shows the result of the proposed approach (nonlinear interferometric analysis) combined with the S-Transform with the aim to evaluate the main frequency content of the IRF related to each selected time-window.

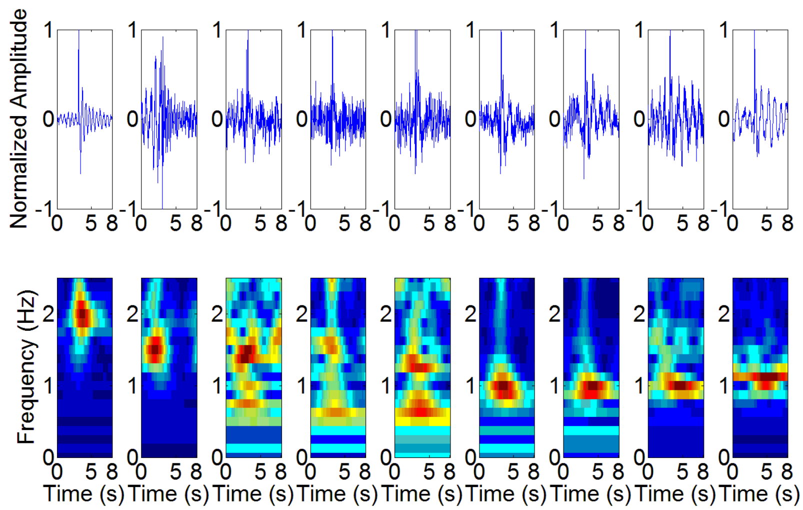


Figure 9. Nonlinear Interferometric Analysis performed on the top floor accelerometric recording and S-Transform evaluated on the single IRF

It is worth noting that the instantaneous fundamental frequency of the structure changes over time starting from a value equal to 2.0Hz, reaching a minimum frequency equal to 0.85Hz and concluding with a fundamental frequency equal to 1.10Hz.
For each time-window the fundamental frequency can be automatically evaluated considering the S-Transform results. In fact, the fundamental frequency corresponds, for each time-window, to the frequency related to the maximum value of the S-Transform for (as shown in Figure 10).

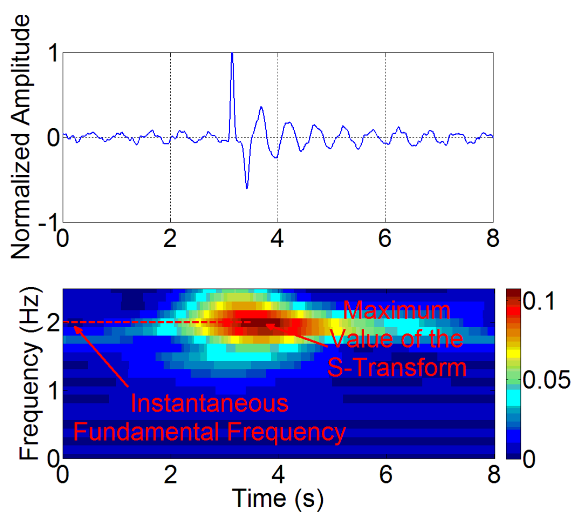


Figure 10. Automatic evaluation of the instantaneous fundamental frequency of the monitored structure by using a limited frequency domain S-Transform of the local IRF

Further analyses are necessary to better calibrate the length of the moving time-window in order to minimize the spurious frequency within each impulse response function evaluated on both weak and strong motion. It is important to highlight that just two three-directional stations are necessary to perform these kinds of analyses on structures: the former installed on the bottom floor and the latter installed on the top floor.

5. CONCLUSIONS

Structural Health Monitoring together with all the dynamic identification techniques is increasing in popularity in both scientific and civil community. The assessment of an increasing number of aged structures and infrastructures requires a huge effort, especially if the purpose is to provide a faithful evaluation of seismic risk. The current practice of periodic visual inspections, for the safety evaluation appears more and more inadequate. A specific task of the Italian research RELUIS-DPC 2010-2013 project, funded by the Department of Italian Civil Protection (DPC), deals with devising and implementing a fast procedures to get useful information about the damage evolution in a large number of strategic buildings during and after seismic events, using the records of just few sensors located on the structure. The feasibility and the cost optimization are the most important goals for the simplified monitoring system in order to favor a widespread use of such systems. During the past two decades, a significant amount of researches have been carried out in the field of Non-destructive Damage Evaluation (NDE) methods using the changes in the dynamic response of a structure (Chen et al., 1995; Capecchi and Vestroni, 1999; Ponzo et al., 2010). Techniques based on Fourier transform provide good results when the response of the system is stationary, but fail when the system exhibits a non-stationary, time-varying behaviour. To hamper classical techniques, it is not necessary that a building reaches damage: even the non-stationarity of the input and the possible interaction with the ground and/or adjacent structures can show the inadequacy of classic techniques (Ditommaso et al., 2010). In 1996, Stockwell introduced a new powerful tool for the signals analysis: the S-Transform. Compared with the classical techniques for the time-frequency analysis, this transformation shows a much better resolution and also offers a range of fundamental properties such as linearity and invertibility (Ditommaso et al., 2012). The ability to investigate the non-stationary response of structures opens new scenarios, giving the opportunity to explore new possibilities.
In this paper using a combined approach based on the S-Transform (Stockwell et al., 1996) and on the seismic interferometric analysis (Snieder and Safak, 2006), an upgrade of a simplified procedure (Ponzo et al., 2010) has been proposed. The upgrade consists in the possibility to better automatize the damping and fundamental frequency evaluation during the nonlinear strong motion of the monitored structure. Results are consistent with those expected if compared with other techniques. Further analyses are necessary to better calibrate the length of the moving time-window in order to minimize the spurious frequency within each IRF evaluated on both weak and strong motion phases.

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