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Monitoring the structural dynamic response of a masonry tower: comparing classical and time-frequency analyses

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ABSTRACT

The monitoring of the evolution of structural dynamic response under transient loads must be carried out to understand the physical behaviour of building subjected to earthquake ground motion, as well as to calibrate numerical models simulating their dynamic behaviour. Fourier analysis is one of the most used tools for estimating the dynamic characteristics of a system. However, the intrinsic assumption of stationarity of the signal imposes severe limitations upon its application to transient earthquake signals or when the dynamic characteristics of systems change over time (e.g., when the frequency of vibration of a structure decreases due to damage). Some of these limitations could be overcome by using the Short Time Fourier Transform (STFT). However, the width of the moving window adopted for the analysis has to be fixed as a function of the minimum frequency of interest, using the best compromise between resolution in both the time and frequency domains. Several other techniques for time-frequency analysis of seismic signals recorded in buildings have been recently proposed. These techniques are more suitable than the STFT for the application described above, although they also present drawbacks that should be taken into account while interpreting the results. In this study, we characterize the dynamic behaviour of the Falkenhof Tower (Potsdam, Germany) while forced by ambient noise and vibrations produced by an explosion. We compare the results obtained by standard frequency domain analysis with those derived by different time-frequency methods. In particular, the results obtained by the standard Transfer Function method, Horizontal to Vertical Spectral Ratio (HVSR), Short Time Fourier Transform (STFT), Empirical Mode Decomposition (EMD) and S-Transform are discussed while most of the techniques provide similar results, the EMD analyses suffer some problems derived from the mode mixing in most of the Intrinsic Mode Functions (IMFs).

Keywords: Structural Health Monitoring – Dynamic Identification – Empirical Mode Decomposition – S-Transform – Masonry Tower

INTRODUCTION

Several techniques for signal analysis have been proposed with the aim of overcoming the limitations of the classical Fourier analysis when signals are non-stationary (Gabor, 1946; Cohen, 1989; Young, 1993; Addison, 2002; Dehghani, 2009). For structural engineers, non-stationarity in the seismic signal recorded within a building is generally linked to the non-linear behaviour of the structure, to dynamic interaction between structure and soil and/or with adjacent structures.

Techniques based on the Fourier Transform, as well as all the tools that have their basis on the assumption of stationary behaviour of structures, are not always appropriate when applied to structures whose response changes over time. Some of the limitations related to the classical Fourier analysis could be overcome by using the Short Time Fourier Transform (STFT). However, the width of the moving window adopted for the analysis must be fixed as a function of the minimum frequency of interest, using the best compromise between resolution in both the time and frequency domains. It is clear that this limitation could significantly affect the results. For this reason, over the last few years, several other techniques for time-frequency analysis of seismic signals have been proposed (e.g. Stockwell *et al.*, 1996; Huang *et al.*, 1998). These techniques appear more suitable than STFT for the structural dynamic identification, although they also present drawbacks, such as for example the fact that the algorithm needs high-performance computers. A tool that allows adapting naturally the time resolution depending on the analysed frequency is the S-Transform (Stockwell *et al.*, 1996). This integral transformation has already been applied successfully to engineering and seismology (e.g. Bindi *et al.*, 2009; Pakrashi and Ghosh, 2009; Mucciarelli *et al.*, 2010; Schimmel and Gallart, 2007; Parolai, 2009, Puglia *et al.*, 2011, Picozzi et al. 2011, Smith et al., 2012), as well as for applications in other scientific fields, (such as, Portnyagin, 1999; Assous *et al.*, 2005; Ruthner *et al.*, 2005; Jena *et al.*, 2006; Jones *et al.*, 2006; Pulkkinen and Kataoka, 2006; Dehghani, 2009; Faisal *et al.*, 2009).

Another tool for analysing the dynamic non-linear and non-stationary response of a system has been proposed by Huang *et al.* (1998) and used for several applications (Flandrin *et al.*, 2005; Bin Altaf *et al.*, 2007; Bradley *et al.*, 2007; Huang and Milkereit, 2009; Gallego *et al.*, 2010; Rehman and Mandic, 2010; Rehman and Mandic, 2011). The key part of the method is the *Empirical Mode Decomposition* (EMD) that allows us to decompose any complicated signal into a finite and often small number of *Intrinsic Mode Functions* (IMF). The method is adaptive and is based on the local characteristics of the data. Moreover, it is applicable to linear, non-linear and non-stationary signals. Furthermore, each IMF could also be analyzed in the time-frequency domain using the Hilbert transform (Huang *et al.*, 1998). In recent years the technique has also been applied to civil engineering purposes, for example, the health monitoring of structures (Poon and Chang, 2007) and pipelines (Davood and Farid, 2010).

In this study, we characterize the dynamic behaviour of the Falkenhof Tower (Potsdam, Germany) while it is forced mainly by ambient vibrations and by a transient produced by the vibrations of an explosion. In particular, we compare the results obtained by standard frequency domain analysis with those derived from different time-frequency methods: the standard Transfer Function method; the Horizontal to Vertical Spectral Ratio (HVSR); the Short Time Fourier Transform (STFT); Empirical Mode Decomposition (EMD); and the S-Transform.

EXPERIMENT AND DATASET DESCRIPTION

On the 9th of July, 2008, several kilometres outside of the inhabited area of Potsdam, a bomb dating back to WWII was destroyed. This offered the opportunity to investigate the dynamic behaviour of the tower, and its interaction with the adjacent structure, using both the noise and vibration induced by the bomb (Ditommaso *et al.*, 2010a). In order to collect the necessary experimental data set, 11 velocimetric stations were installed by the Helmholtz Centre Potsdam GFZ German Research Centre for Geosciences in cooperation with University of Basilicata. Eight sensors were installed inside the tower, located in the area surrounding the explosion site, while three sensors were used to monitor the free-field ground motions. The north–south direction of the instruments coincides with the direction joining the tower and the explosion site (radial direction), and was used to orient the sensors. The radial direction between the location of the explosion and the building coincides with one of the main structural directions (geometrical axis in the plan view).

Each station is equipped with a 24 Db digitizer and a 1 Hz tri-directional geophone. The sampling rate was set to 100 samples per second. The bomb was detonated about 300m from the building and had a mass of about 10kg. The energy released was estimated to be around 40MJ. The maximum amplitude recorded was similar to what could be expected for a magnitude 3 earthquake situated 30km from the site (Ditommaso et al., 2010a).

The building (Fig. 1a), henceforth referred to as the tower, is a brick-masonry, bearing-wall structure. It has a square footprint (4m × 4m) and is about 16m high. It is built on sandy ground and has no underground levels. The structure consists of 6 storeys used as residential apartments and an additional level for the roof. The inter-storey height is 2.70m. The thickness of the walls and the characteristics of the staircase are unknown. The tower was monitored by installing the sensors along two vertical directions, indicated as A and B in the plan view shown in Fig. 1b. Along vertical direction A, stations were located at all storeys, one for each level, starting from the ground level, up to the roof, with the only exception being the first floor where access was denied by the owner for privacy reasons. In the vertical direction B, stations were installed at the ground level, the first and at the sixth floor. Figure 1b, also depicts the position of the stations installed outside the building. Station T1 was located at the bottom of an existing well at 2.5m depth. It is worth noting that the installation was carried out several hours before the explosion, and that the de-installation of the network was done the day after. Therefore, a large amount of ambient noise data (several hours) was also available for the analysis, together with the signal generated by the explosion (Ditommaso *et al.*, 2010a).

Figure 2 shows examples of the recorded signals (EW components), including the vibration induced by the explosion, at different levels within the tower. Signals recorded at different levels are shown with different colours. The top panels show a 3500s time-window, centred around the explosion.
The recorded ambient noise was stationary before and after the explosion. Figure 2, left panel, shows a 1-second time-window selected around the signal generated by the explosion. The propagation of waves through the tower (indicated by a black dotted line) and the following vibration of the building can be easily identified. Figure 2 (right panel) shows a 7 second time-window selected where the signal only consist of ambient noise. Note that the dominant mode consists of a nearly stationary wave propagating within the tower. In fact, the signal frequency is the same, independent of the level at which it was recorded.

The results obtained by standard frequency-domain analysis are compared with those derived by different time-frequency methods. Two of these methods are based on the assumption of stationary system behaviour, while three others overcome this limitation. In particular, the Transfer Function method (Chopra, 1995), Horizontal to Vertical Spectral Ratio (Mucciarelli, 1998), Short Time Fourier Transform (Gabor, 1946), Empirical Mode Decomposition (Huang *et al.*, 1998) and S-Transform (Stockwell *et al.*, 1996) are considered and discussed.

IDENTIFICATION OF STRUCTURAL EIGENFREQUENCIES USING TRANSFER FUNCTIONS, STFT AND HVSR ANALYSES

In a previous study (Ditommaso *et al.*, 2010a), in order to estimate the structural eigenfrequencies of the Falkenhof Tower, three classical techniques were used: Transfer Functions, Horizontal to Vertical Spectral Ratios and Short Time Fourier Transform (we summarize here the main results carried out in Ditommaso *et al.*, 2010a). Transfer functions were evaluated using a reference station located within the tower, along the vertical A, at the ground floor level. Ditommaso et al. (2010a) carried out several analyses both using ambient noise and forced vibrations, and in particular the transfer functions were evaluated using ambient noise signals collected both before and after the explosion, showing that the structural behaviour remained unchanged after the explosion, hence no damage was inflicted on the structure. Figure 3a shows the transfer functions estimated using ambient noise that can be used to easily identify the eigenfrequencies of the structure. A complete description of the procedure is reported in Ditommaso *et al.* (2010a). Figure 3a shows that the first mode of the tower is 2.73 Hz (along the WE direction) while the second mode (along the NS direction) at 2.87 Hz. The first rotational mode was estimated to be at 6.20 Hz, while several peaks can be observed in the frequency range 10-15 Hz. These peaks are related to the higher modes of the tower along both the WE and NS directions, and represent the fourth and fifth modes, respectively. They have a different shape when compared with those related to the other modes due to the interaction of the tower with the adjacent structure. The sixth mode of the tower is estimated to be at 22.10 Hz and it represents the second rotational mode. Figure 4a shows that, due to the stationarity of the signal, when dealing with ambient noise, the structural eigenfrequencies can be correctly identified by estimating the transfer function using a Fourier spectra. On the other hand, the identification of structural eigenfrequencies by calculating the transfer function using forced vibration signals might be biased by the non-stationarity of the signal.

The larger spectral peaks observed during the explosion in the 5-20 Hz frequency range were due to the interaction of the tower with the small adjacent building (Ditommaso *et al.*, 2010a) that was optimally excited due to the frequency content of the explosion signals.

By performing a time-frequency analysis (STFT) of the signals recorded during the explosion within the tower at different levels, it is possible to discriminate the tower eigenfrequencies from other signal components dominated by the seismic signal input and the interaction with adjacent structure (Ditommaso *et al.*, 2010a). This integral transformation, for a signal *h(t)*, is defined as

$$STFT(\tau, f) = \int_{-\infty}^{+\infty} h(t) \cdot w(t-\tau) \cdot e^{-i2\pi ft} dt \qquad (2.1)$$

where $w(t-\tau)$ is the moving window.

The structural eigenfrequencies can be easily identified, since, for a structure characterized by linear behaviour like in the case at hand, they do not vary within the considered time-window (Figs. 4a and 4b). Moreover, other non-stationary spectral peaks appears within the time-frequency plots of different floors, such as for example a 7.5 Hz peak observed by Ditommaso et al. (2010a), at the first level. As discussed by Ditommaso et al. (2010a), these peaks are due to the interaction with the adjacent structure, which decreases with increasing height (i.e., level). The eigenfrequencies of the structure were also evaluated by rotating the horizontal components of the recorded motion and then performing an HVSR analysis on the ambient noise (Ditommaso *et al.*, 2010a).

These analyses showed that simple rotational HVSR allows us to identify the structural frequencies, in particular those related to the first three main modes of vibration. However, the relative amplitudes of the HVSR peaks might be different from those estimated by the transfer function method due to the amplification of the vertical component of motion in the building. Table 1 summarizes the structural eigenfrequencies evaluated using spectral ratios (H/H) by Ditommaso *et al.* (2010a), which coincides within the experimental error with HVSR and STFT.

TIME-FREQUENCY ANALYSIS USING S-TRANSFORM

As discussed in the previous sections, a tool that allows us to adapt naturally the time resolution depending on the analysed frequency is the S-Transform (Stockwell *et al.*, 1996). This integral transformation, for a signal *h(t)*, is defined as:

$$S(\tau, f) = \frac{|f|}{2\pi} \int_{-\infty}^{+\infty} h(t) \cdot e^{-\frac{(\tau-t)^2 \cdot f^2}{2}} \cdot e^{-i \cdot 2 \cdot \pi \cdot f \cdot t} dt \qquad (3.1)$$

where t is time, f is frequency and τ is a parameter that controls the position of the Gaussian window along the time axis. One of the main advantages of this transformation is the possibility of easily applying a time-frequency filter to the S-Transformed signal (e.g., Pinnegar and Eaton, 2003; Schimmel and Gallart, 2005; Parolai, 2009) which is especially suitable for extracting the non-linear dynamic modal response of soil and structures (Ditommaso *et al.*, 2010b and 2012).

Figure 4 shows an application of the normalized S-Transform method to both the ambient noise and explosion signals recorded within the Falkenhof Tower (NS and WE components). Figure 4 shows that the eigenfrequencies of the tower can be identified from the S-Transform of the ambient noise. In fact, the tower oscillations are stationary under ambient noise excitation and eigenfrequencies in both the NS and WE directions, i.e., they do not vary with time. By contrast, when the ground motion generated by the explosion excites the tower (Figs. 4c and 4d) there is stronger excitation of the higher modes at frequencies greater than 10 Hz. It is clear that the eigenfrequencies are the same, but the S-transform provides the possibility to follow the tower's behaviour in time-frequency domain with a high resolution.

Figure 5 shows a comparison between S-Transform (Figs. 5a and 5b) and STFT (Figs. 5c and 5d) analyses for both the WE and NS components at the third level. These signals have been selected because, as shown by the modal shapes retrieved by Ditommaso et al. (2010a), they allow the observation of all the main modes of the structure. The analyses show similar results, but S-Transform provides a better resolution in the time-frequency domain then it is possible to better understand which is the energy distribution within different eigenmodes. In fact, while the WE-1 and NS-1 modes are very clear from both analyses, the frequency variations over time of higher modes are not so clearly defined from the STFT analyses.

STRUCTURAL DYNAMIC IDENTIFICATION USING EMPIRICAL MODE DECOMPOSITION

In this work, by using the EMD method, we aim to decompose the stationary response of the tower into a sum of elementary responses, described by IMF that should be representative of a single mode of vibration of the tower. For a signal *h(t)* the decomposition is represented by the following relationship:

$$h(t) = \sum_{k=1}^{N} IMF_k(t) + r_N(t) \qquad (4.1)$$

where IMF_k is the k-th IMF, N is the total number of IMF and $r_N(t)$ represents the trend (Huang *et al.*, 1998).

The procedure proposed by Huang *et al.* (1998) was applied to ambient noise and explosion related signals recorded on the tower. We selected for each floor only the WE component because it corresponds to the direction where the fundamental mode of vibration was observed, and it is the direction free of influence from the adjacent structure. Using ambient noise vibrations, all recorded signals were decomposed into 10 IMFs. In this work, for the signal decomposition, the maximum standard deviation was fixed a $5 \cdot 10^{-6}$ and to evaluate the single IMF, a cubic spline interpolation was used. The first four IMFs are related to the dynamic behaviour of the tower:

- IMF1 should be representative of the mode R2 (22.10 Hz);
- IMF2 should representative of the mode WE2 (12.22 Hz);
- IMF3 should be representative of the mode R1 (6.20 Hz);
- IMF4 should be representative of the mode WE1 (2.73 Hz).

However, considering that 2.73Hz is the lowest (i.e., fundamental) frequency of vibration of the tower, it is clear that the IMFs after the fourth one (not shown here), are not related to any physical behaviour of the oscillating tower.

Under ambient noise excitation, the tower behaviour is linear, stationary and no mode of vibration is interacting with another. This should be a very simple application for EMD analysis because in these conditions, each IMF should be directly related to a single mode of vibration. Unexpectedly, Figure 6 shows that for each IMF, the frequency content changes considerably over time, hinting at mode mixing. We apply the FFT (Fast Fourier Transform) to the single IMF because, due to the signal at hand, they should represent modes of the structure that are expected to be harmonic.

Therefore, although in general the IMFs are not expected to be always harmonic functions, in this case they are expected to be.

Applying the Fourier Transform to the single IMF, we can obtain an overview of the main harmonics contained in the signal. Figure 7 shows that instead of a single peak representing a particular mode of vibration, the spectra are characterized by several harmonics, each of them related to a different mode of vibration. For example, within the spectra related to the IMF 1, instead of only the R-2 mode, the WE-2, R-1, and WE-1 modes can also be observed. Similarly, analysing the frequency content of IMF2, for which we expected the peak representative of the mode WE-2 (see Table 1), we also observe two predominant frequencies of 2.73 Hz and 6.20 Hz. On the other hand, from the dynamic analyses, we know that these frequencies are those related to the fundamental and third mode of vibration. Clearly, in this case the frequency content is also coherent with that expected for the tower, but the frequencies are mixed.

As further confirmation that the Huang decomposition does not seem suitable for this engineering application, as shown by Ditommaso *et al.* (2010a), the IMF3 and IMF4 at all levels should have the same phase angle and should be related to the well defined modal shapes for the R-1 and WE-1, respectively. However, as shown in Figure 6c and 6d, the IMFs for the different levels are not in phase. Moreover, Figure 7 shows that the IMF3 has a frequency content corresponding roughly to the first mode of vibration WE-1.

Furthermore, in order to stress what are the differences resulting from EMD and classical analysis, a comparison in the time-domain was performed between the original signal, the filtered signal around the first mode of vibration (WE-1), and the IMF3, using a 10 seconds time-window recorded at the third level.

As expected, the original and 2-4 Hz filtered signals are in phase. By contrast, the IMF3 shows a behaviour comparable with the other signals only in some parts (Figure 8a) from 1 to 4 seconds, while from 4 to around 7 seconds it changes both its amplitude and frequency content. Most importantly, the non-stationary character of IMF3 affects the modal shape estimation. In fact, while the modal shape evaluated for the 2-4 Hz filtered signals is in agreement with that one obtained by Ditommaso *et al.* (2010a) for the first mode of vibration, the modal shape obtained for IMF3 presents an unrealistic amplitude and shape. In fact, for the fundamental mode of vibration, the particular mode shape derived from IMF3 cannot be justified from the theory of structures. On the other hand, it is clear that between 4 and 8 seconds the EMD (IMF 3) is not able to represent a physical signal anymore.

Finally, similar observations about the performance of the Huang decomposition hold also when the method is applied to the explosion derived signals (not shown here).

CONCLUSIONS

To understand the physical behaviour of buildings during earthquake ground motion, it is important to understand which are the best signal analyses techniques able to study a physical problem related to the structural behaviour. Then, appropriate tools for understanding the structural behaviour should be tested and applied.

In this paper, a comparison between several techniques has been carried out using both classical and innovative time-frequency analyses. In particular, for time-frequency analyses, the STFT, the S-Transform and the Empirical Mode Decomposition have been used.

From a first comparison between STFT and S-Transform, we found that the performance of the two methods are consistent, however, the S-Transform allows, especially for higher modes of vibration, the evolution in time of the signal in the time-frequency domain to be better followed. Therefore, in agreement with the results obtained by Ditommaso et al. (2010b and 2012), the S-Transform appears to be a useful tool for the dynamic identification of nonlinear structural systems.

Interestingly, the application of the Empirical Mode Decomposition proposed by Huang *et al.* (1998) provided results that were not consistent with those from standard techniques. We observed that the main problem in applying the EMD method for structural identification of the Falkenhof Tower is related to the mode mixing. In fact, despite the stationary dynamic behaviour of the tower, the EMD algorithm was found to be very sensitive to the instantaneous energy content of the different modes of vibration. We believe that the EMD problems arise from the order used during the IMFs selection. In fact, the selection of IMFs starts from higher towards lower frequencies. Considering that, when excited by ambient noise and explosion derived signals, the mass participation coefficient of the tower for the lower frequencies is higher, the fundamental mode is associated to higher energy content. For these reasons, during the selection of the IMFs for the higher frequencies, the results are always contaminated by the higher energy low frequency signals. In particular, the EMD approach appeared to act as a low-pass filter with a band-stop variable over time as a function of the energy distribution contained within the analysed signal. From a practical point of view, we showed that due to the EMD problems, the IMFs extracted from the different levels cannot be used to obtain reliable modal shapes.

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Table 1. Main frequencies of vibration of the Falkenhof Tower derived using classical techniques

Mode name	**WE-1**	**NS-1**	**R-1**	**WE-2**	**NS-2**	**R-2**
Mode type	Traslational WE	Traslational NS	Rotational Z	Traslational WE	Traslational NS	Rotational Z
f (Hz)	2.73	2.87	6.20	12.22	12.95	22.10

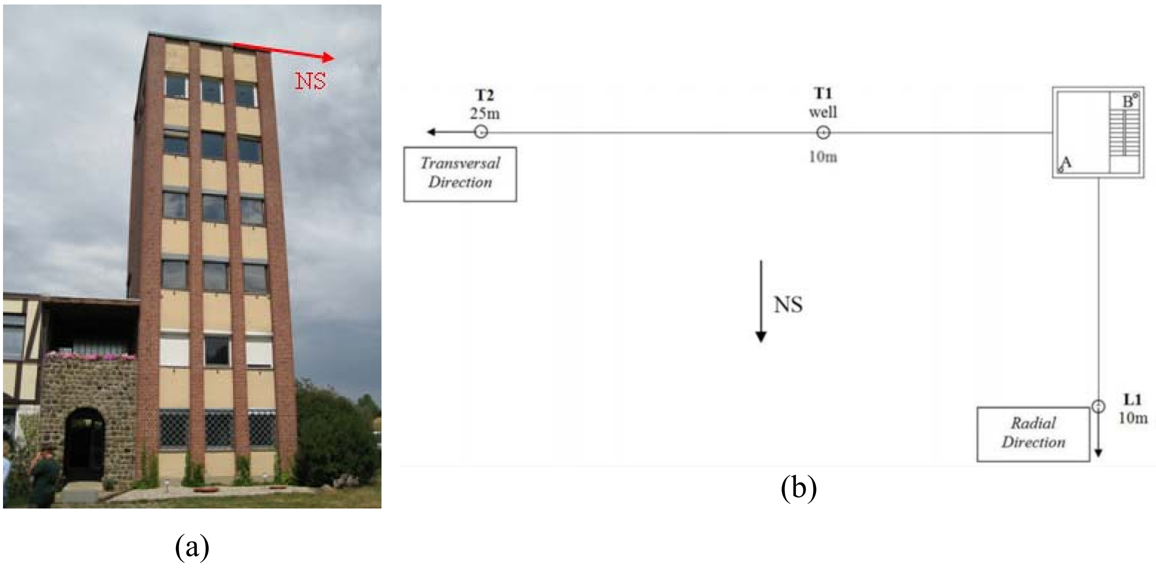


Figure 1: (a) The Falkenhof Tower and (b) instrumentation plan view (Ditommaso *et al.*, 2010a)

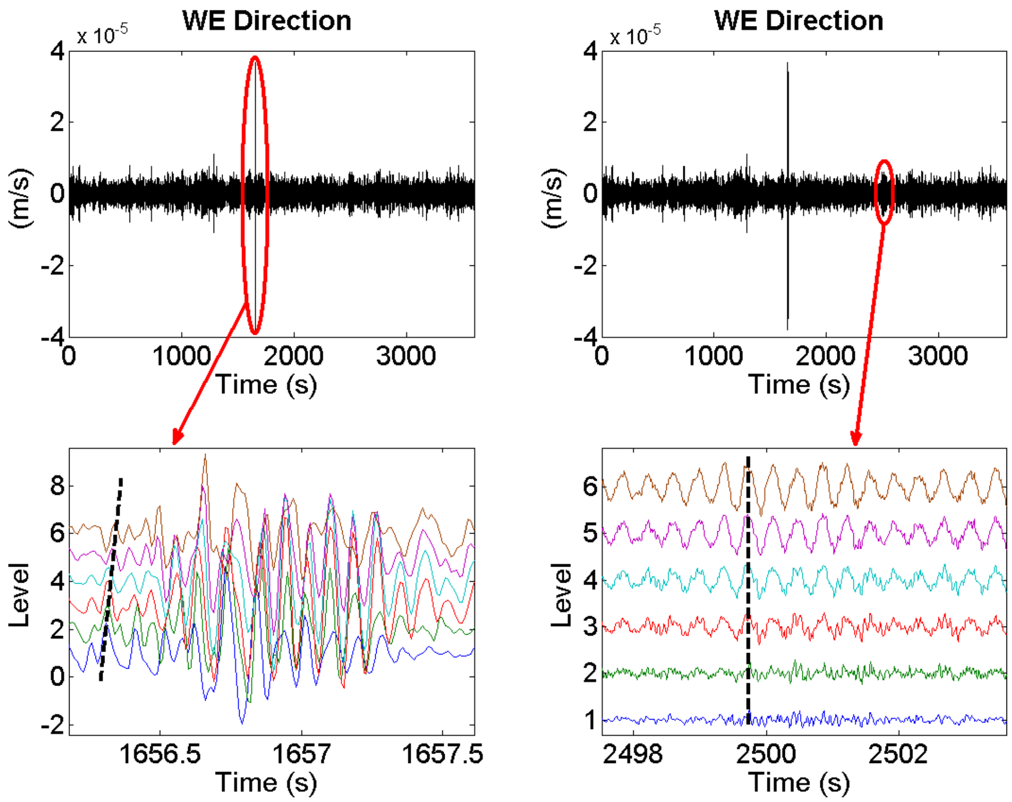


Figure 2: Examples of recorded signals at all floors on the Falkenhof Tower. The zoom on the explosion's signals (left column, bottom) includes a dashed line that highlights the up-going wave through the building; the dashed line in the zoom on the noise signals (right column, bottom) highlights the stationary nature of the noise wave.

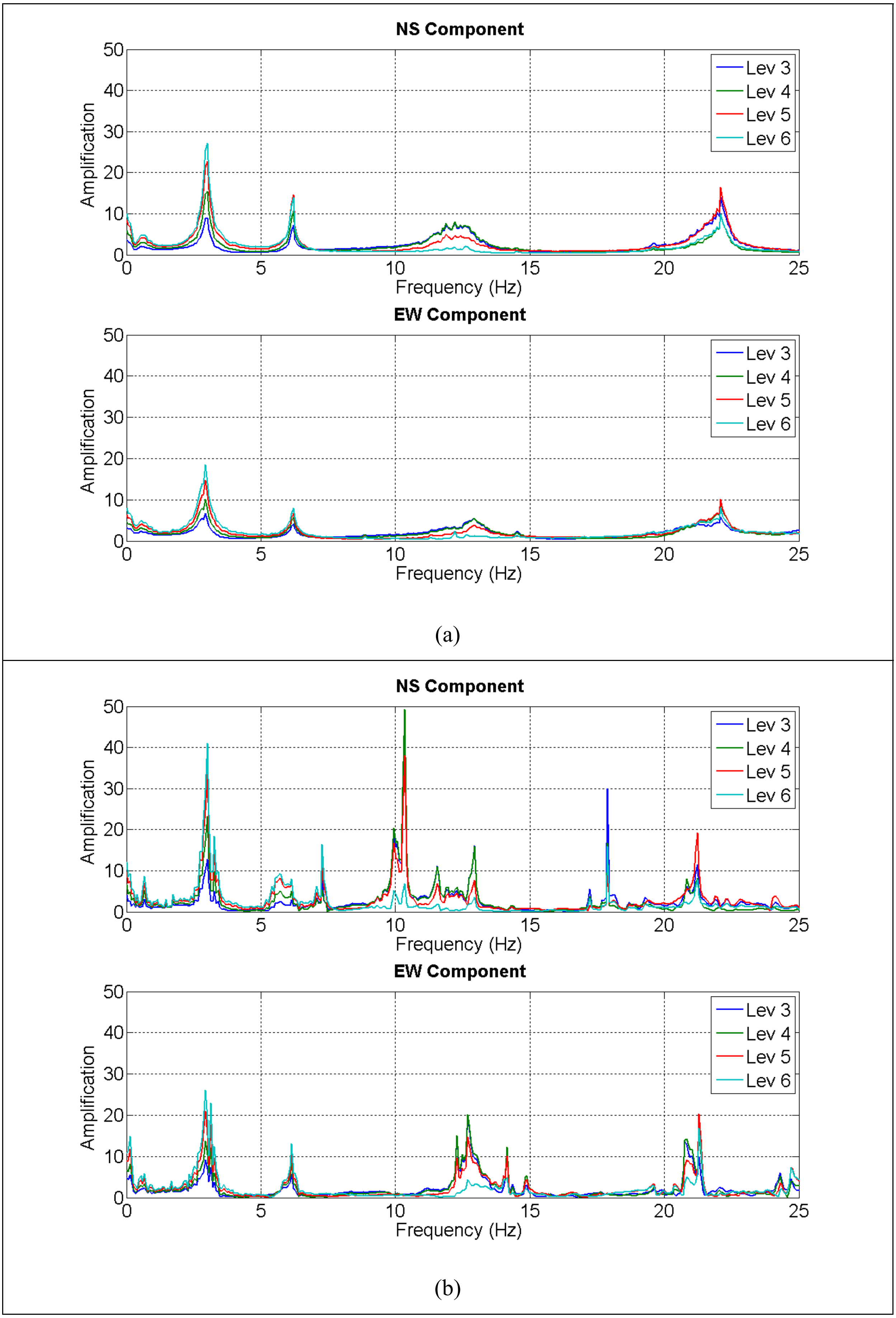


Figure 3: Transfer functions evaluated using: (a) ambient noise and (b) explosion (Ditommaso *et al.*, 2010a).

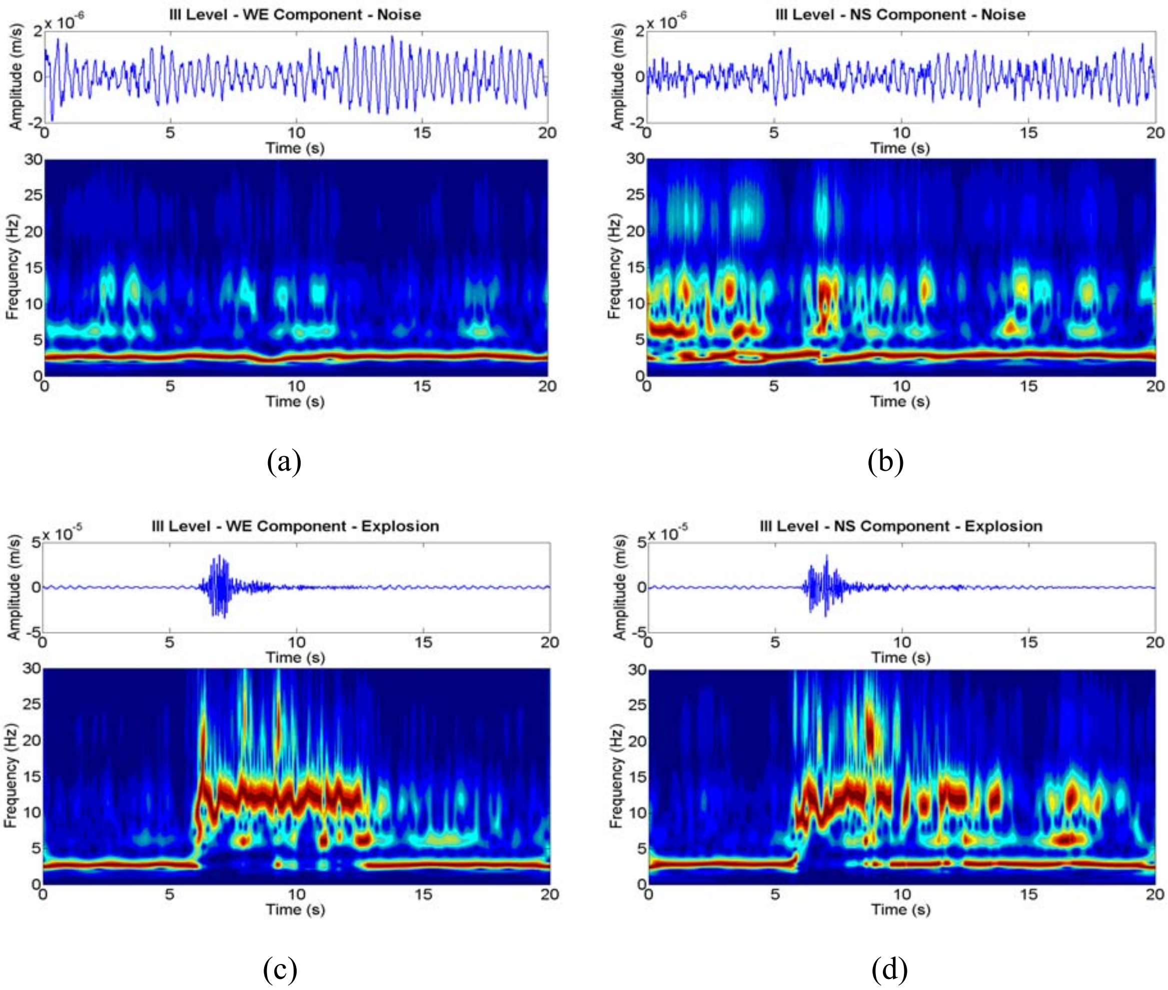
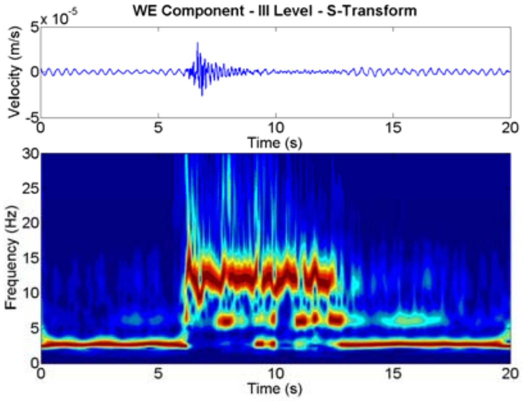
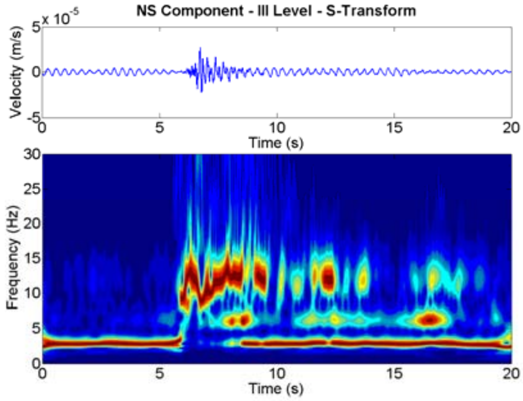


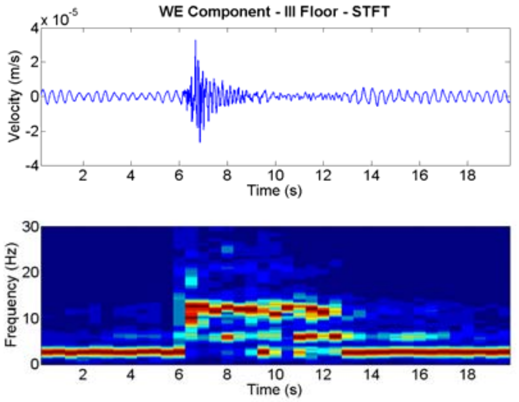
Figure 4: Example of time-frequency analysis using S-Transform of signals recorded at the third level of the tower: (a) WE component (noise) – (b) NS component (noise) - (c) WE component (explosion) – (d) NS component (explosion)



(a)

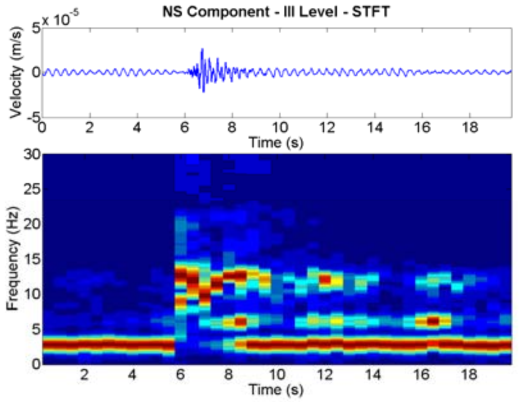


(b)



(c)

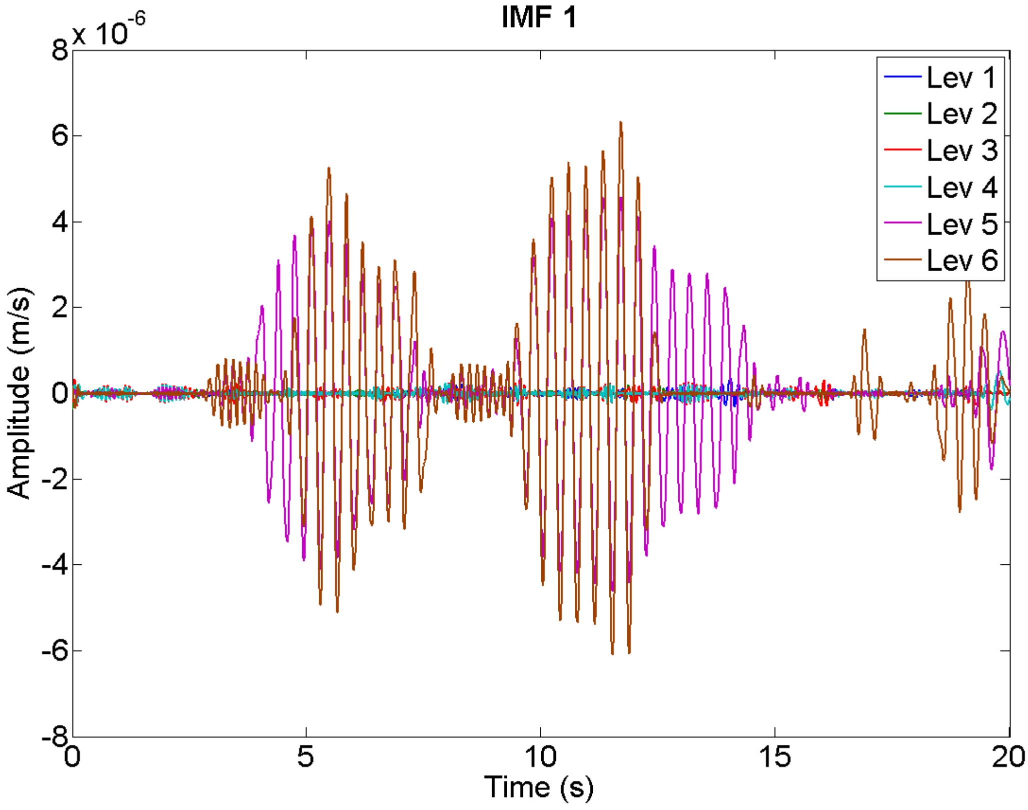
WE Direction



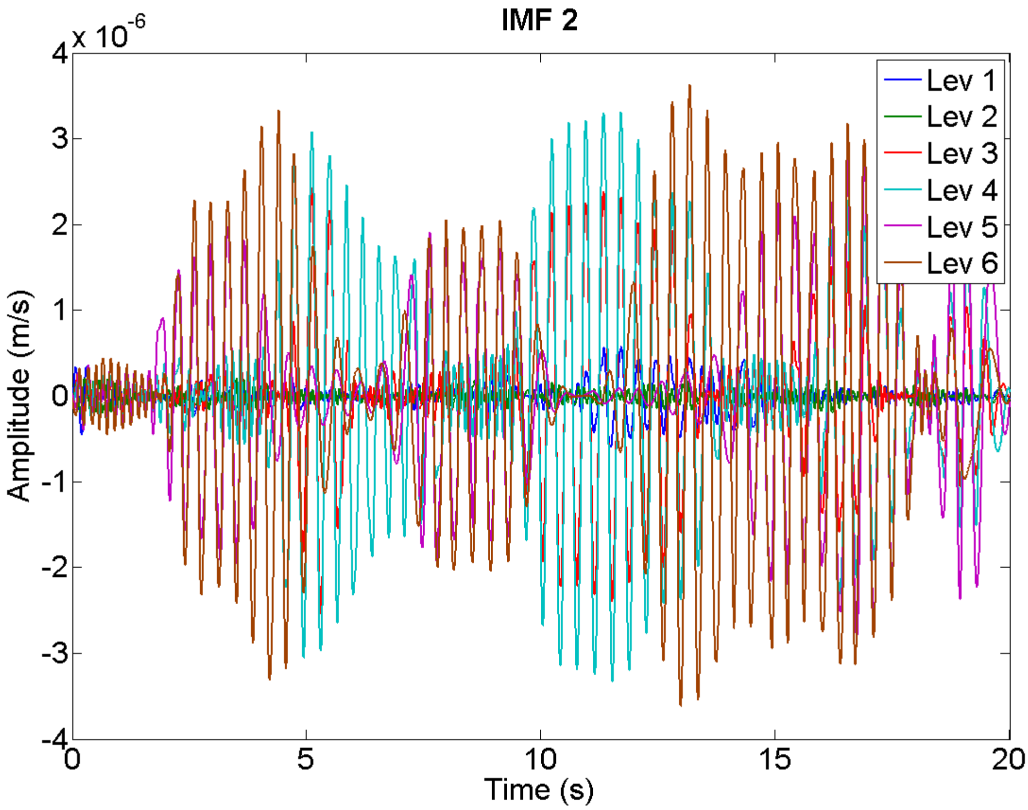
(d)

NS Direction

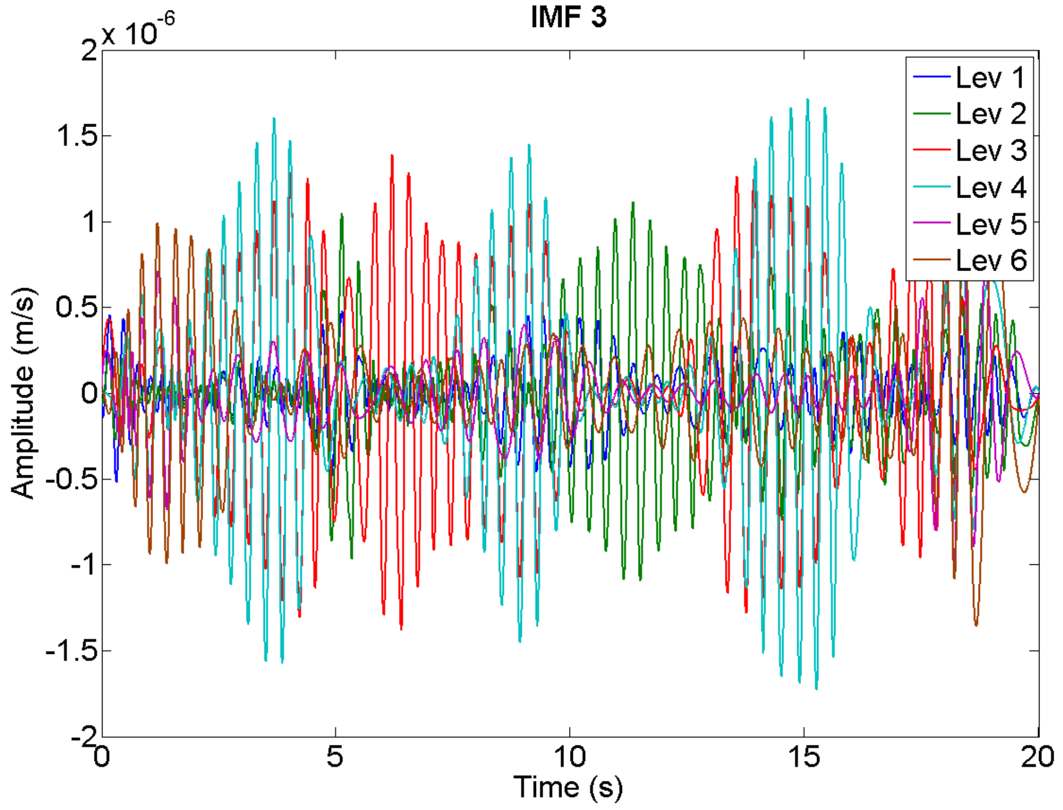
Figure 5: Analyses of signals recorded at the third level of the tower: comparison between normalized S-Transform (first row) and normalized STFT (second row) results.


IMF 1
x 10^-6
Amplitude (m/s)
Time (s)
Lev 1
Lev 2
Lev 3
Lev 4
Lev 5
Lev 6

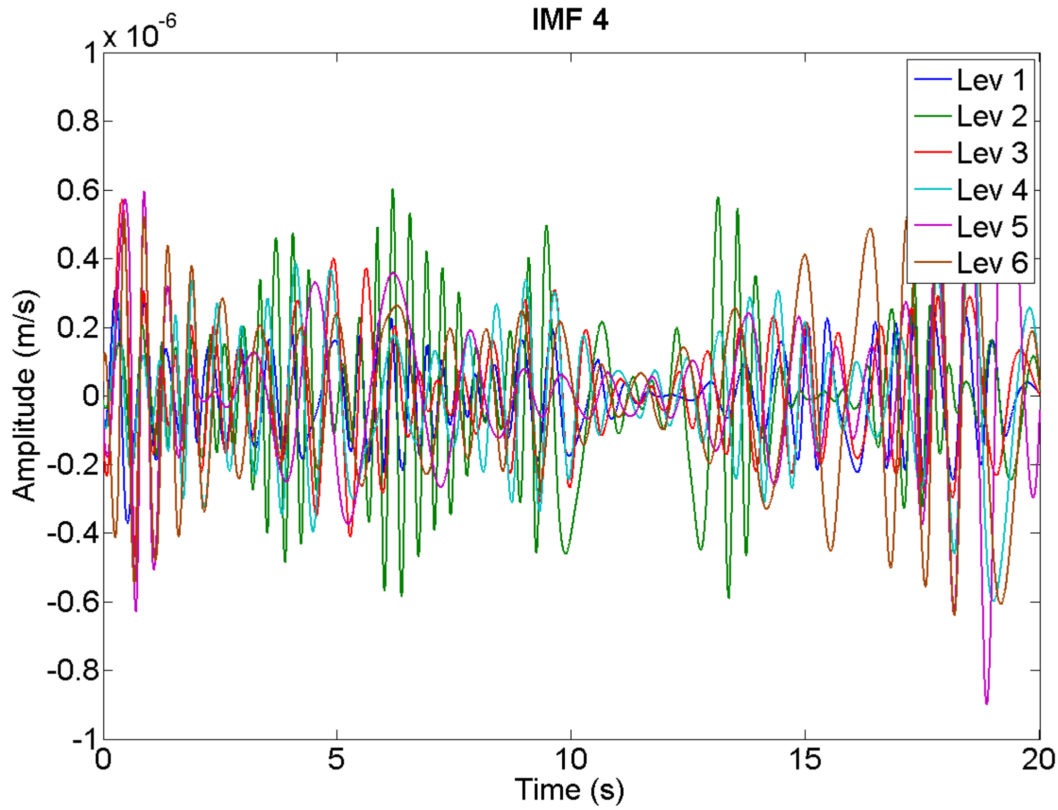
(a)


IMF 2
x 10^-6
Amplitude (m/s)
Time (s)
Lev 1
Lev 2
Lev 3
Lev 4
Lev 5
Lev 6

(b)



(c)



(d)

Figure 6: IMFs (from 1 to 4) of signals (ambient vibration) recorded on the tower at different levels and in the WE direction

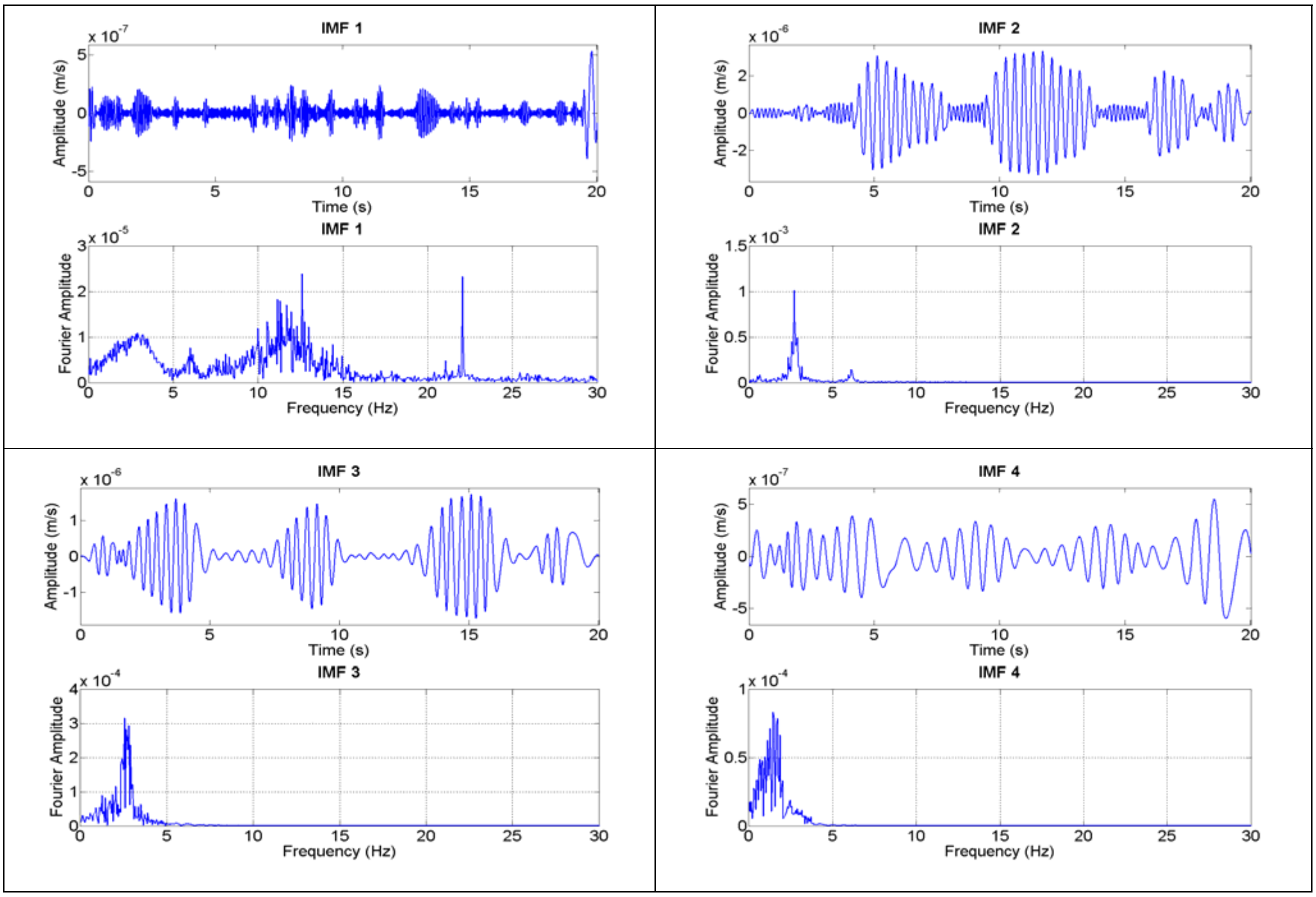


Figure 7: Frequency analyses of the firsts 4 IMFs of the signals recorded on the Falkenhof Tower at the third level

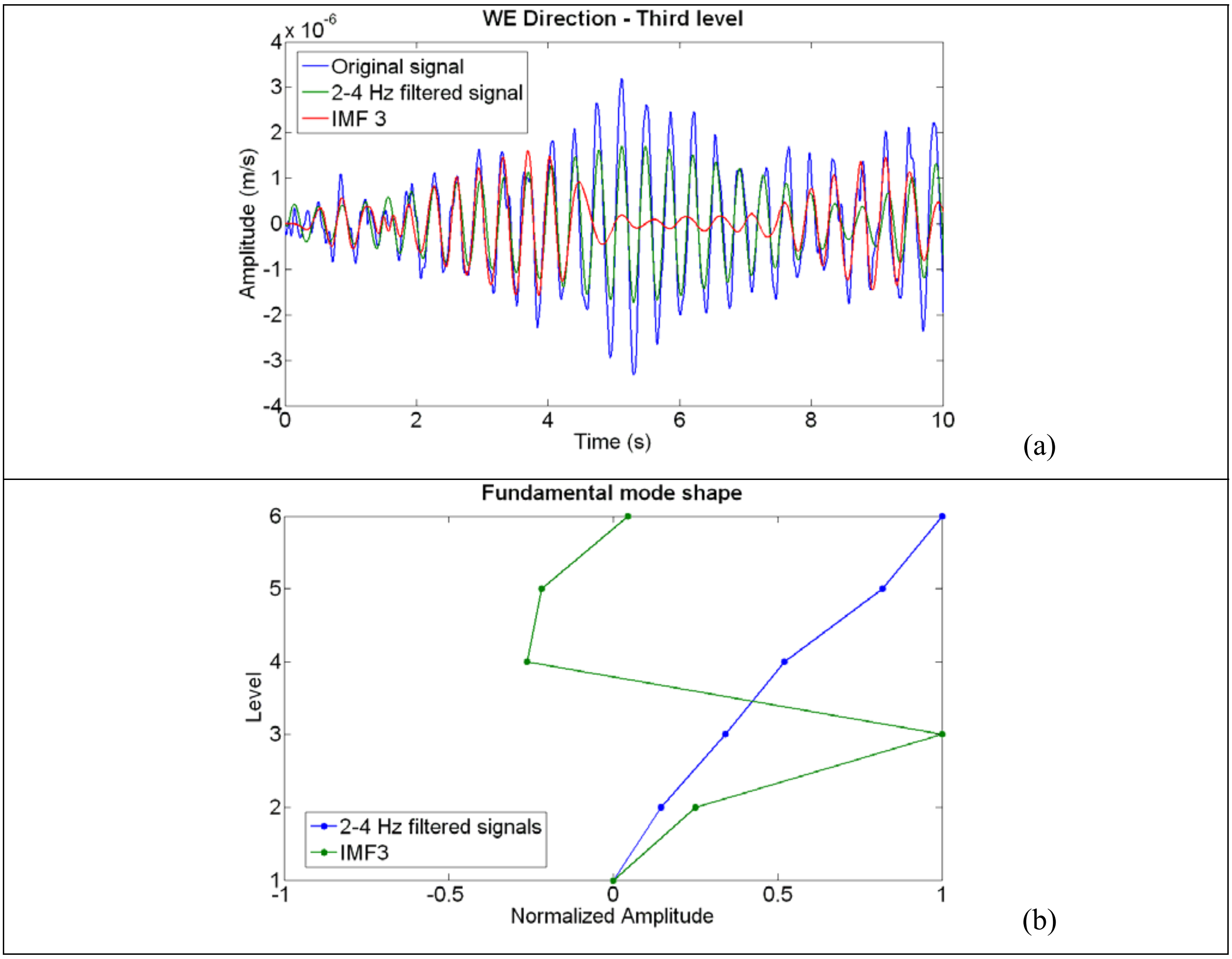


Figure 8: (a) Comparison between observed noise signal, 2-4 Hz filtered signal and IMF3 of Empirical Mode Decomposition; (b) Comparison between modal shapes evaluated using both the classical approach and Empirical Mode Decomposition