ANALYSIS OF THE SEISMIC RESPONSE OF A DAMAGED MASONRY BELL TOWER

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SUMMARY

The dynamic behavior of an earthquake damaged masonry bell tower is analyzed. The structure, which is connected to other lower masonry buildings, suffered damages from the 1996 Reggio Emilia earthquake. The dynamic interaction between the tower and the adjacent buildings certainly played a fundamental role in the crack formation. The experimental study was performed as follows. First the dynamic characterization was performed by using both ambient and forced vibration tests. Then a accelerometric monitoring system was installed in order to record the effects of the aftershocks. Sixty-seven earthquakes of low magnitude were recorded in about two months, which are classified on the basis of the peak ground acceleration. The comparison between the results of the dynamic characterization and the seismic response of the tower is carried out. Changes in the dynamic characteristics of the tower under earthquakes with higher intensity are pointed out. Finally, the seismic response of the tower is analyzed in detail by plotting the instantaneous response frequency during the main recorded events.

INTRODUCTION

A moderate size earthquake (M_L=4.8, I_0=VII MCS, epicenter 44° 48' 00'' N 10° 42'00'' E) struck the Reggio Emilia province, northern Italy on October 15, 1996. A significant number of structures was seriously damaged especially in the cultural and historical heritage. Among these the medieval Bell Tower of S. Giorgio Church in Trignano was selected for testing innovative restoring technique, in the framework of the European Project ISTECH [Castellano et al., 1997]. This technique consists in the insertion of vertical post-tensioned tie bars to increase the bending resistance. Shape Memory Alloy devices will be placed with the tie bars. Because of their limited encumbrance, SMA devices are particularly suitable for the retrofit of cultural heritage. The reasons for this choice and the details of the design were already discussed [Forni et al., 1997].

The tower is connected to contiguous masonry buildings at the lower part of its height and shows near horizontal cracks, just above the roof level of the adjacent buildings. The dynamic characterization was first performed by means of ambient vibration tests, carried out by using eight seismometers Kinemetrix SS1. Then six accelerometers were deployed on the structure to record the effects of the aftershocks. A triaxial sensor was installed on the basement, three uniaxial horizontal sensors at the top. Sixty-seven low magnitude earthquakes were recorded in about two months. Changes in the dynamic characteristics of the tower under earthquakes of different intensity are pointed out by means of the usual spectral analysis. The seismic response of the tower is also analyzed by plotting the instantaneous response frequency.

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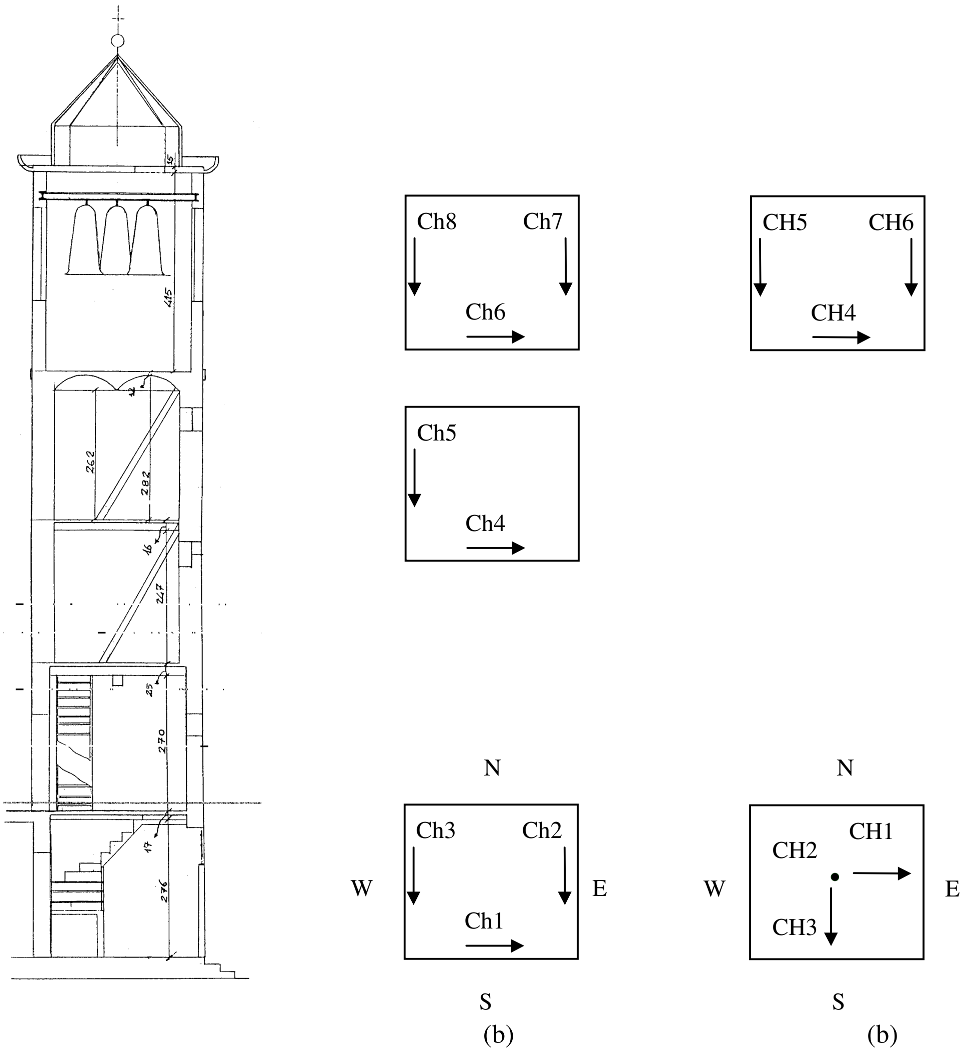

Figure 1: Velocimeter (a) and accelerometer (b) layouts

DESCRIPTION OF THE STRUCTURE

The Bell Tower of S. Giorgio Church in Trignano is 18.5 *m* tall and 3.35*3.00 *m* at the basement (Figure 1). It was built in 1302 as an ancient roman chapel, but the original structure withstood several changes and additions in the following centuries. The connected houses were built in 1700. The structure of the tower was heavily modified in the second half of 1800. Four masonry columns at the corners, about 40 *cm* thick, compose the main structure. Masonry walls fill the spaces between the columns. The connections between the walls and the columns are not effective. Materials are very poor: local burnt brick, common mortar with sand and pozzolana. The first three floors are made of timber, the forth one was substituted by a two brick little vault floor, supported by a central steel I-beam. The stairs were composed by wooden and steel flights. The tower is connected to the structure of the church and to other masonry buildings on three sides, up to the height between 6 and 7 *m*. The most apparent effect of the October 15, 1996 earthquake was the opening of a near horizontal crack in the freely rising part, above the roofs of the adjacent buildings. The crack interested three of the four sides of the tower. A 30 *mm* offset was also observed between the upper and the lower part of east wall, due to a clockwise rotation of the upper part with respect to the lower one. Other cracks were noticed at the basis of the bell cell windows. The restoration design consisted of the following works: consolidation of masonry by means of the injection of special mortar, insertion of vertical post-stressed steel tie bars at the four corners, reconstruction of all the floors and restoration of the bell cell. The SMA devices were supposed to be placed in series with the post-stressed bars.

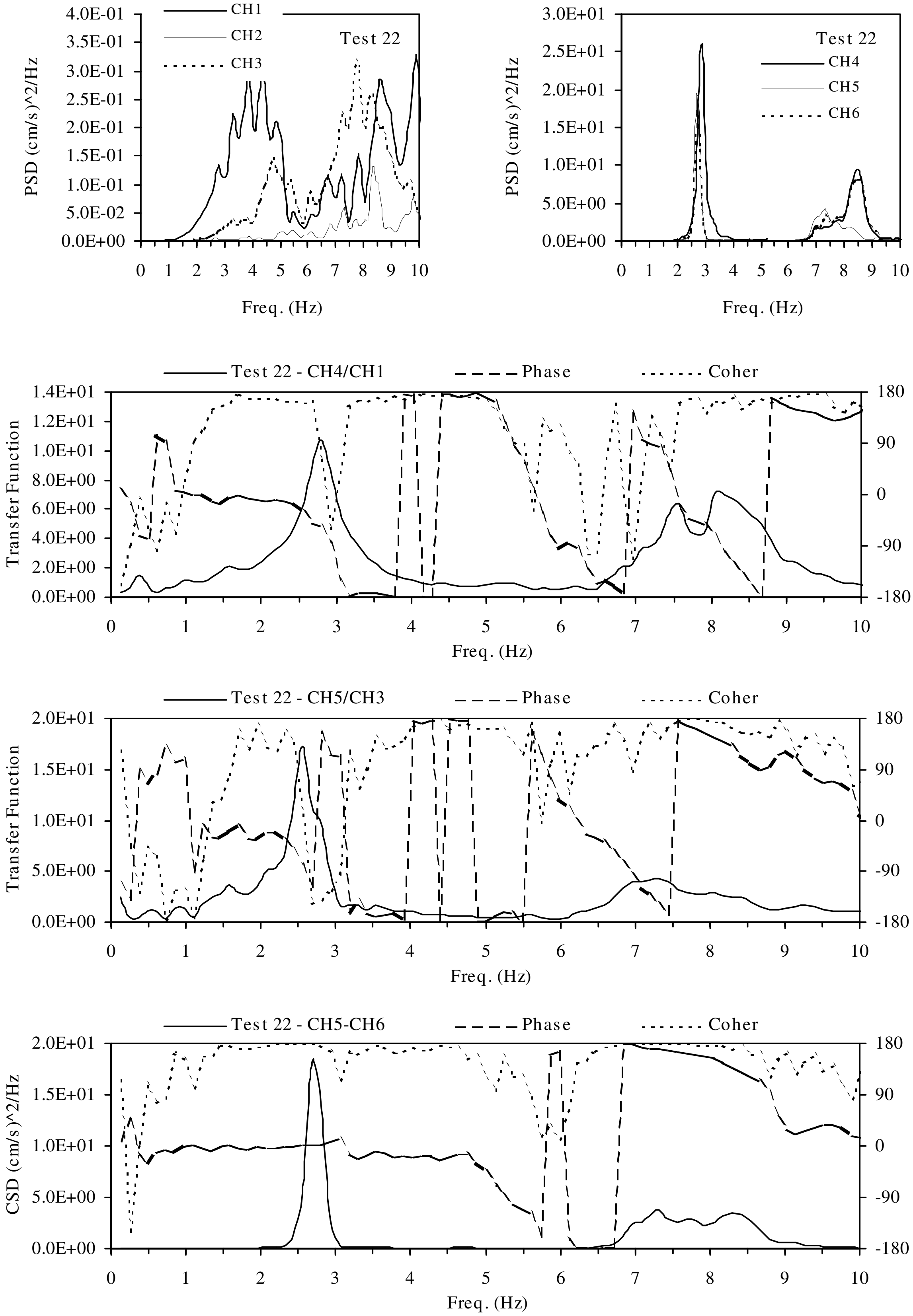

Figure 2: Spectra relative to Event No. 22

DYNAMIC CHARACTERIZATION OF THE TOWER

The dynamic characterization of the tower was performed by means of ambient vibration tests [Bongiovanni et al, 1998]. Forced vibrations were also considered, by recording the vibrations due to the effects of a mass dropped on the ground near the tower. The recorded data were studied in the frequency domain by means of cross-spectral analysis. The motion in terms of modal shapes was examined by means of the power spectral density amplitudes.

Several tests lasting 32 *sec* with a sampling rate of 0.004 *sec* were performed. The sensors were deployed in the configuration shown in Figure 1: Ch1, Ch2 and Ch3 were on the ground level; Ch4 and Ch5 were just under the cracks; Ch6, Ch7 and Ch8 were at the top.

The results of the dynamic characterization tests can be summarized as follows.

A resonant frequency of 2.7 *Hz* was apparent in the spectra relative to sensors in the N-S direction (Ch2, Ch3, Ch7, Ch8), while evident peaks at 2.9 *Hz* were found in the spectra of records in the W-E direction (Ch1, Ch4, Ch6). Several peaks are between 6 and 8 *Hz*.

The cross analysis allowed to point out that two modal shapes, with predominant displacements in the N-S and W-E directions respectively, were associated to the first two frequencies. Sensors in the same direction were always in phase at these frequencies. We deduced that no torsion rotations were associated to these frequencies.

A peak at 6.9 *Hz* was apparent in all the spectra. The cross analysis between the records at Ch7 and Ch8 showed that it was associated to a torsional modal shape.

The records of sensor Ch5 showed a very low contribute of the frequency components in the range [0, 4] *Hz*, while the spectral amplitudes were much higher in the range [6, 8] *Hz*. We deduced that location Ch5 did not take part in the first and second vibration modes, but it had a very important contribution in the torsional mode. This particular behavior was related to the presence of the other structures, which influenced the dynamic characteristics of the tower very much.

BEHAVIOUR UNDER SEISMIC ACTIONS

The fixed instrumentation consisted of a triaxial accelerometric sensor located on the ground floor and three uniaxial horizontal accelerometric sensors at the top (Figure 1). Sixty-seven aftershocks were recorded in about two months. The events were classified on the basis of peak ground acceleration. The analysis in terms of energy content gave the same result.

The records obtained under the lower energy earthquakes confirmed the results of the dynamic characterization. As an example the power spectral densities of records relative to a medium intensity earthquake, are shown in Figure 2. Peaks at the already mentioned resonance frequencies are evident. In more details the resonance frequency of 2.7 *Hz* is apparent for CH5 and CH6, while the resonance frequency of 2.9 *Hz* is present for CH4. The analysis of the phase factor pointed out that the signals at the top and the corresponding ones on the basement are 90 degrees out of phase at these frequencies. This occurrence allowed to consider the base records as the input and the top records as the response. The torsional resonant frequency of 6.9 *Hz* is also present.

The dynamic characteristics, in terms of resonance frequencies, modal shapes and damping, changed in presence of higher level earthquakes significantly.

In Figure 3 the power spectral densities relative to event No. 23 are plotted. The different frequency content of the top records in comparison to the basement ones is evident. In fact, records at the basement contain, as obvious, the characteristics of the seismic motion.

The reduction of the resonant frequencies, with reference to the previous example, relative to event No. 22, is apparent. In fact the first frequency is equal to 2.43 *Hz* and is associated to a modal shape with prevalent displacements in the N-S direction. The second frequency is at 2.60 *Hz* and is relative to a modal shape with prevalent displacements in the W-E direction. It is also evident that, in this case, the energy in the W-E direction is much higher than the energy in the N-S direction. Just the opposite happened in the spectra relative to other events. This occurrence may be related to the different directivity of the various earthquakes [Clemente et al., 2000].

The transfer functions of event No. 23 are plotted in Fig. 3. The amplification of the motion at the resonance frequencies is evident.

The frequency at 6.6 *Hz* is apparent in the cross spectral density of records of sensors CH5 and CH6, which is also plotted in Figure 3. The signals being 180 degrees out of phase, this happens to be the torsional frequency of the tower.

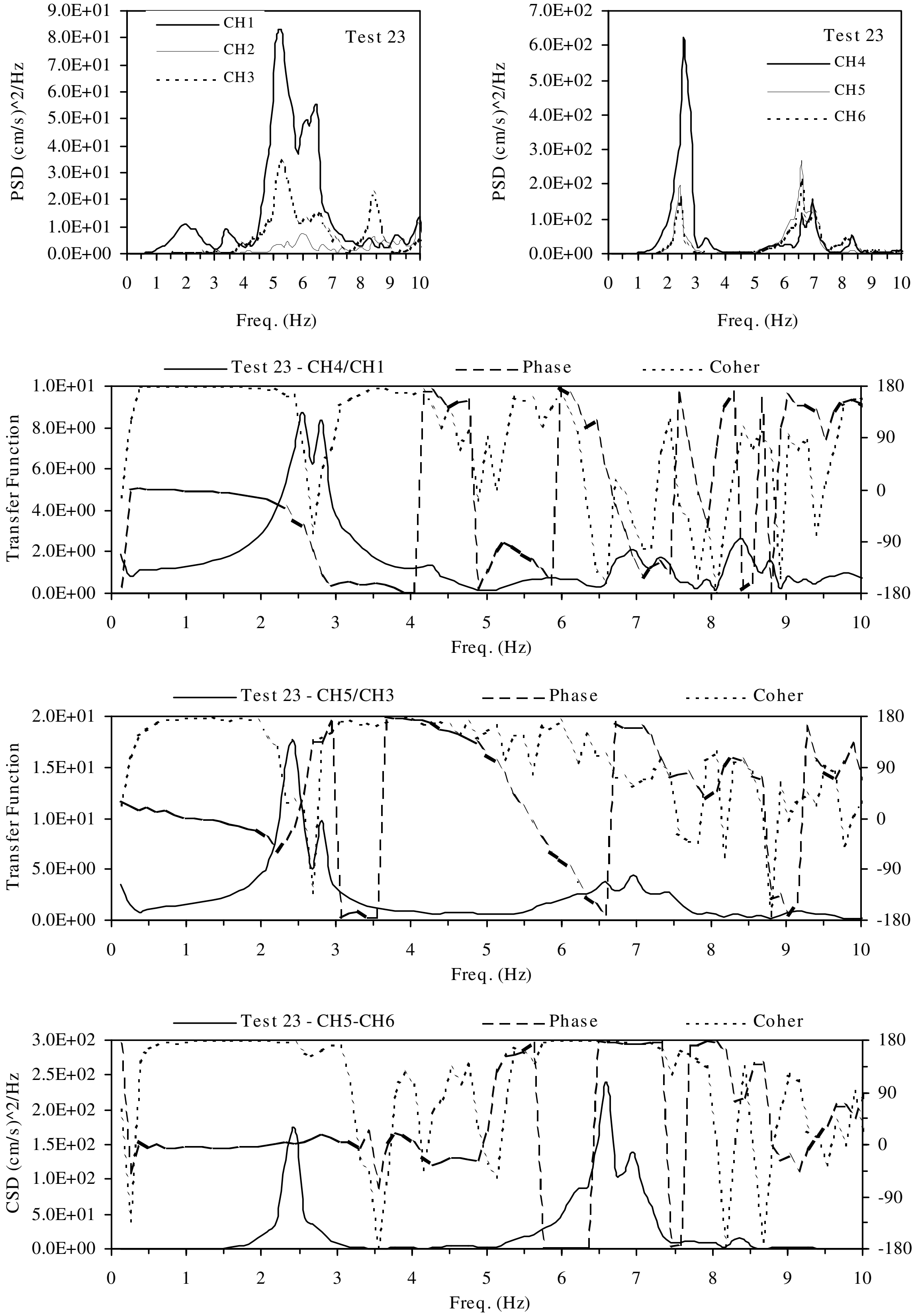

Figure 3: Spectra relative to Event No. 23

In a parallel study [Clemente et al, 2000] the events have been classified on the basis of the energy content estimate [Arias 1970]

$$I_A = \frac{\pi}{2g} \int \left(a_x^2 + a_y^2 + a_z^2 \right) dt$$

obtaining the same result as in terms of PGA. In more detail we choose the Arias scalar intensity I_A at the basement as a measure of the input energy for the structure. The analysis of the structural behavior of the tower under the recorded shocks allowed to identify a limit value of the Arias intensity, which must be viewed as a characteristic of the damaged structure. In fact if the Arias intensity was lower than the limit one the behavior of the tower did not change significantly. If the Arias intensity was greater than the limit one, the resonant frequencies decreased when the Arias intensity increased. Damping was calculated by means of the half power bandwidth method. The obtained values were almost dispersed and did not follow any simple law. Anyway the increment of damping with the Arias intensity was apparent.

It was also noticed that earthquakes of similar intensity had slightly different effects on the dynamic behavior of the tower. This occurrence demonstrated that even the higher recorded earthquakes did not cause significant additional damages to the tower.

In some cases a different behavior was detected under earthquakes of similar amplitude. This was attributed to the directivity of the earthquakes. In fact, different ratios between the power spectral densities of the three components were found.

All the records were analyzed by using the Hilbert transform [Kanasewich 1981] to obtain the instantaneous frequency according to the Claerbout approximation.

This analysis is intended first to substantiate the findings of the cross-spectral analysis in terms of natural frequencies and damping, and their variation with input energy. Then we tried to follow, when possible, the time evolution of the dynamic behavior of the structure and to correlate it with the presence of damage.

First we filtered the time histories in the frequency band [0.6-10.0] *Hz*, which represents the field of interest for the structure, to reduce numerical instability in the calculation. Then we calculated the instantaneous frequency for all the channels, plus that relative to the difference between CH5 and CH6, to analyze in detail the torsional effect.

We found that the first two natural frequencies are quite close each to the other and similar to those obtained from the spectral analysis. It is not significant to verify the exact value but it is satisfactory to observe that the value found in the record of CH4 is slightly higher than that found in the record of CH5, according to previous results.

Torsional frequency, clearly seen from the analysis of difference between the time history of record CH5 and the time history of record CH6, is confirmed to have a value just below 7.0 *Hz* for the lower input energy cases, and which falls down to about 5.5 *Hz* for the higher input energy cases. These facts are illustrated in Figure 4, where the time histories and the relative instantaneous frequencies of event No. 22, characterized by a medium value of energy and event No. 23, which is one of the events with higher energy.

Looking at the evolution of the dynamic behavior of the structure, it is apparent, for the differences CH5-CH6 of both the events in Figure 4, that during the action of p-waves, i.e. with lower input energy, the torsional frequency is close to the value obtained from the dynamic characterization. Instead, during the strong phase, when also the effects of s-waves are present, this frequency lowers, more for the higher input energy record relative to event No. 23; after this phase the torsional frequency oscillates around the lower value according to the amplitude of the acceleration.

During the strong phase of event No. 22 it is apparent that the dominant frequency for the single channels, CH4, CH5 and CH6, is the torsional one. Lowering of bending frequencies with time, going from p-waves phase to the end of strong phase is clear only for CH4.

The analysis of records CH4, CH5, CH6 for event No. 23 (Figure 4), shows that the dominant frequency is always the torsional one, while the bending one is limited in time. Also in this case the lowering from p-waves phase to strong phase and further is apparent.

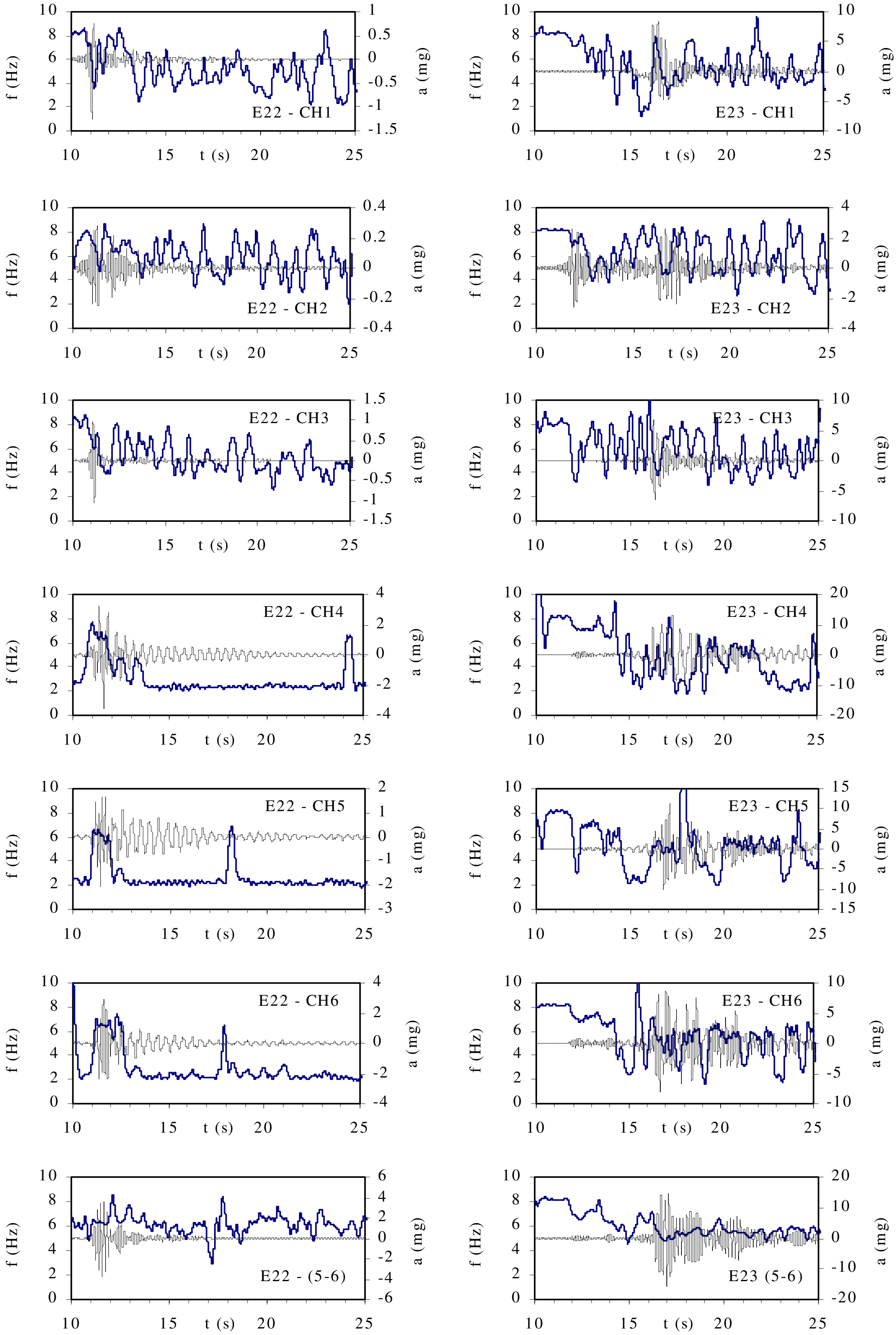

Figure 4: Time-histories and instantaneous frequencies

CONCLUSIONS

The approach used in this analysis can be assumed as a general methodology to investigate the presence of damage in structures, especially when visual inspection is not possible or damages are not apparent. The dynamic characterization tests allowed to evaluate the structural properties and to have preliminary information about the health status of the structure. The strong motion monitoring over a consistent time period allowed to analyze the structure under true seismic inputs.

The following features have been pointed out:

- The dynamic characteristics of the tower change remarkably according to the earthquake intensity; a simple correlation between the resonant frequencies and the Arias intensity has been found;
- The increase of the damping when the input energy gets higher is also evident but the experimental data are very dispersed, the phenomenon looking very irregular.

The analysis of the instantaneous frequency, developed by means of the Hilbert transform, has allowed to point out the changes of the frequency during each recorded earthquake and so the following aspect of the structural behavior:

- Decrease in the frequency during the earthquake is related to the mechanical non linearity of masonry in conjunction with damage and also to increase of energy dissipation, i.e. increase of damping;
- Increase in input energy leads the structure to be driven by the torsional behavior; this occurrence is related to the characteristics of the damage.

The collected experiences allowed to state that the instrumentation of structure after earthquakes, to record the effects of the aftershocks, is a suitable way to improve our understanding of the seismic behavior of damaged structures [Clemente et al, 1999]. This is an important step in the limit analysis of structures and is needed especially when several events follow one the other as in the case of the 1997 Umbria-Marche seismic crisis.

ACKNOWLEDGEMENTS

Funding of ISTECH Project (Development of innovative techniques for the improvement of stability of cultural heritage, in particular seismic protection) by the European Commission under Environmental and Climate programme (Contract No. ENV4 - CT95 - 0106) is gratefully acknowledged.

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