Proceedings of the 9th U.S. National and 10th Canadian Conference on Earthquake Engineering
Compte Rendu de la 9ième Conférence Nationale Américaine et
10ième Conférence Canadienne de Génie Parasismique
July 25-29, 2010, Toronto, Ontario, Canada • Paper No 1326

DAMAGE EVOLUTION ASSESSMENT OF A 6-STORY MASONRY BUILDING DURING AFTERSHOCKS AFTER WENCHUAN EARTHQUAKE

C. X. Mao[1,2], H. Li[2], W. S. Zhou[2] and J. P. Ou[2,3]

ABSTRACT

As an effective time-frequency analysis tool, wavelet transform is commonly used in damage detection of civil infrastructures to ambient vibration, wind or earthquake through identifying the frequency variation of structures. However, accurate identification results always can not be obtained in the case of earthquake excitation because of frequency contamination of quake. To solve this problem, a new damage detection and evaluation method using transfer function in wavelet domain is proposed in this paper. The new method is called the wavelet energy transfer function spectrum (WETFS). The frequency variation of structures during earthquake is identified by this method without frequency contamination of earthquake. The structural damage is then evaluated through changes in structural frequencies. Another advantage of this method is that it requires minimum sensor number on buildings. The WETFS method is applied for damage detection and evaluation of a 6-story actual masonry building in Guangyuan city, Sichuan province, China. This building was slightly damaged in the Wenchuan earthquake occurred in May 12, 2008. A monitoring system was installed on the building after Wenchuan earthquake and the structural seismic responses under total 10 aftershocks were measured. The natural frequency of this building is identified decreasing 7.9% and 8.31% during the Oct 24 and Dec 10 aftershocks by means of the WETFS method, and that means structural damage of this building further deteriorated in these two aftershocks.

Introduction

The seismic response data of real structures measured by the seismic arrays in it can help monitor the health of the structure, detect damage as it occurs, and issue an early warning after the earthquake, before physical inspection is possible. Furthermore, better understanding of damage and its evolution in buildings during earthquake can be obtained. Then engineers can better design new buildings and strengthen existing buildings to survive further quakes

1. Institute of Engineering Mechanics, China Earthquake Administration, 29 Xuefu Road, Harbin, 150080, China. Email: maochenxi@iem.ac.cn
2. School of Civil Engineering, Harbin Institute of Technology, Harbin, 150090, China
3. School of Civil & Hydraulic Engineering, Dalian University of Technology, Dalian, 116024, China

(Todorovska and Trifunac 2008). To this end, a typical 6-story masonry structure in the Guangyuan city, Sichuan province, China was selected and a monitoring system was installed on it to record the seismic responses of this building during aftershocks of the Wenchuan earthquake. This building was built in 1991 and was slightly damaged in the Wenchuan earthquake. The seismic array on it consisted of 7 accelerometers and total 10 aftershocks were recorded by this array since its installation. The measured structural response data was then used, not only for damage evaluation of this building, but also for investigation on damage mechanics of masonry structures.

Most damage detection and evaluation methods assess damage level of structures according to the changes in the structural modal properties. The commonly used modal parameters as damage indices includes: frequency, mode shape, curvature mode, strain mode and modal strain energy. Among them, structural frequency is the most practical index because it is easy to be measured. Furthermore, its accuracy is much better than other modal indices. The required measurement information for frequency identification is also much less than that needed for other modal parameter identification. However, a big shortage for using frequency as damage index is that its relationship with the damage location in structures is not very clear yet. To solve this problem, much valuable work has been done by many researchers. The first contribution was made by Cawley and Adams (1979) . They found that the ratio between the changing rates of two different modal frequencies is the function of damage location in structures. This method was effectively applied for damage localization on plate structure, but it can not work well if more than one damage location exist in the structure. Based on the method proposed by Cawley and Adams, Hearn and Testa (1991) proposed a new index, i.e. the ratio between the square of variation of two different frequencies (the changes in natural frequencies is normalized with respect to the largest frequency change before the ratio is calculated). This index is independent of severity for small deterioration in structures and can serve to indicate the location of structural damage directly. As for the frame structure, Hassiotis and Jeong (1993) established the relationship of the variation in the frequencies to the localized reductions in the stiffness of the structure, and proposed an optimality criterion to solve the underdetermined simultaneous equations. This method was utilized on a ten-story frame structure and multiple damage sites were detected.

The time-frequency analysis methods, mainly the wavelet transform and the Hilbert-Huang transform, were often used to identify frequency variation of buildings to ambient vibration, wind load and earthquake, and then evaluate structural damage during the last three decades (Kareem and Kijewski 1999). However, the frequency identification results under earthquake excitation are always inaccurate or even inexplicable in many cases. From the signal analysis viewpoint, although the building acts as a natural filter and keeps the frequency components of its own in the seismic response data, part of the earthquake frequency components still can be detected in the seismic response at the building floor. Also, the frequency of earthquake is difficult to be separated by digital filter methods because most earthquake signals are narrow band stochastic processes. To solve this problem, a concept of wavelet energy transfer function is proposed in this paper, which is similar to the concept of Fourier transfer function. The frequency variation of structures during earthquakes is estimated by this method.

In this paper, the new method identifying structural frequency evolution during earthquake by using Gabor transform is proposed. This new method is referred as the wavelet energy transfer function spectrum (WETFS) method. In this method, the Gabor transform of structural seismic responses and of the earthquake excitation are performed respectively first. The Gabor wavelet spectra of structural responses and excitation are then both integrated along time subsequently and they are called the Gabor wavelet energy spectra. Finally, the Gabor wavelet energy transfer function is obtained by the quotient of Gabor wavelet energy spectra of response and excitation. The WETFS method is applied in analyzing the seismic responses data of the 6-story masonry building measured during 10 aftershocks. The structural damage evolution during each aftershock is identified and the damage level of this building is evaluated through changes in structural frequencies.

Description of the Building

The building investigated in this study, constructed in 1991, is a 6-story masonry structure in the Guangyuan City, Sichuan Province of China (Fig. 1). The building is an apartment for the staffs working in the Guangyuan Earthquake Administration. It was slightly damaged during the 2008 Wenchuan earthquake.

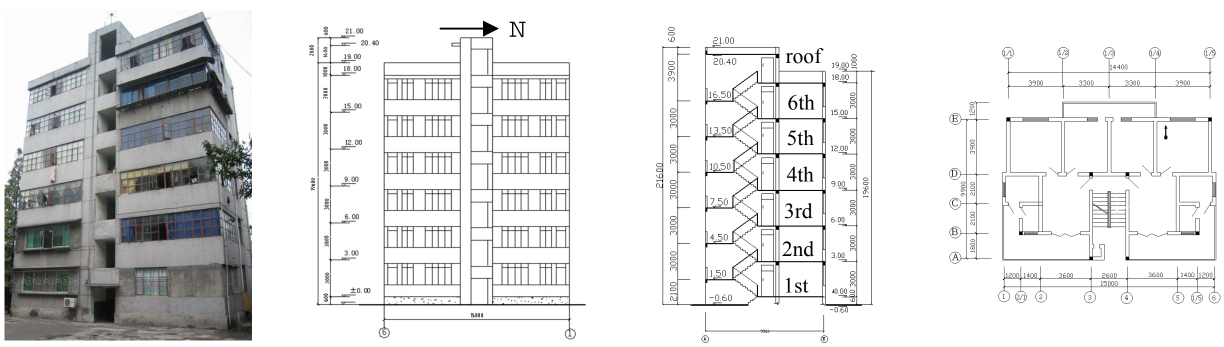


Figure 1. Photo and drawing of the apartment building.

Table 1 Aftershocks recorded in the 6-story masonry building.

No.	Epicenter	Date & time	Magnitude	Latitude	Longitude	Epicentral distance (km)
1	Qingchuan	Oct 3, 2008, 7:08	3.0	32.6	105.3	80.58
2	Maoxian	Oct 4, 2008, 20:10	4.5	31.8	104.1	222.78
3	Qingchuan	Oct 15, 2008, 7:25	3.0	32.5	105.2	83.08
4	Qingchuan	Oct 16, 2008, 3:25	3.0	32.4	105.1	85.35
5	Qingchuan	Oct 16, 2008, 5:10	3.1	32.4	104.9	162.31
6	Qingchuan	Oct 18, 2008, 16:40	3.2	32.4	105.1	86.24
7	Maoxian	Oct 21, 2008, 15:25	4.1	31.7	104.1	224.76
8	Qingchuan	Oct 24, 2008, 5:56	4.0	32.5	105.2	83.98
9	Qingchuan	Oct 25, 2008, 23:05	3.3	32.7	105.4	79.46
10	Qingchuan	Dec 10, 2008, 02:53	5.0	32.6	105.4	80.29

The building was then instrumented by Harbin Institute of Technology after September, 2008. The instrumentation system consisted of an array of 6 uniaxial and 2 triaxial force-balance accelerometers provided by the Harbin Caomu Electronic Technique Co., Ltd. Among them the 6 uniaxial accelerometers were located from the second to the roof floor, while the 2 triaxial accelerometers were installed on the first (ground) and roof floor respectively. The installed accelerometers were characterized by a frequency bandwidth from DC to 120 Hz, an amplitude range of ±2.0g and a dynamic range of 120 dB. The sensors' signals were continuously recorded with a sampling rate of 100 Hz, using a 16-bit data acquisition card NI-6034E, made by National Instrument Inc. Data acquisition software was developed by LabVIEW. All the real-time information and data can be watched and downloaded by a remote controlled computer in the Harbin Institute of Technology through internet. Actually, all above sensors, transmission cables, data acquisition system and network constitute one integrated Online Structural Health Monitoring System (SHMS). The SHMS was operated since October 8, 2008. The instrumentation has recorded 10 aftershocks since its installation. All the aftershocks recorded are summarized in Table 1.

Methodology

Wavelet Theory

Mathematically, wavelet transforms are inner products of the signal $y(t)$ and a family of wavelets. Let $\psi(t)$ be the mother wavelet, it should satisfy the two conditions expressed by Eqs. 1 and 2 as follows:

$$\int_{-\infty}^{\infty} \psi(t)\mathrm{d}t = \mathbf{0} \tag{1}$$

and it is square integrabel or, equivalently, has finite energy, i.e.,

$$\int_{-\infty}^{\infty} \left|\psi(t)\right|^{\mathbf{2}} \mathrm{d}t < \infty \tag{2}$$

The corresponding family of wavelets consists of a series of son wavelets, which are generated by dilatation and translation from the mother wavelet $\psi(t)$ shown as follows:

$$\psi_{a,b}(t) = \frac{\mathbf{1}}{\sqrt{a}}\psi\left(\frac{t-b}{a}\right), \quad a > \mathbf{0}, \quad b \in R \tag{3}$$

where a = the dilatation or scale parameter defining the support width of the son wavelet and b = the translation parameter localizing the son wavelet function in the time domain. The wavelet transform of $y(t)$ is expressed by the following inner product in Hilbert space:

$$W_{\psi}(a,b) = \left\langle y(t), \psi_{a,b}(t) \right\rangle = \int_{-\infty}^{+\infty} y(t)\psi^{*}_{a,b}(t)\mathrm{d}t \tag{4}$$

where the asterisk stands for complex conjugate. Eqs 3 and 4 show that the wavelet transform is a linear scalar product normalized by the factor $\mathbf{1}/\sqrt{a}$ and this scalar product is a measure of the fluctuation of the signal $y(t)$ around the point b at the scale a. The value of the scale a is proportional to the reciprocal of the signal frequency ω. The scale a can be converted to frequency using the following relationship:

$$\omega = \frac{\omega_s}{\sqrt[3]{\mathbf{2}}}\frac{\mathbf{1}}{a} \tag{5}$$

where ω_s = sampling frequency.

WETFS and Instantaneous Frequency of a Structure

As indicated in many literatures, the scalogram of wavelet transform describes variation of frequency component in signal with time. The ridge of wavelet transform scalogram is just the time-varying frequency of the signal. The wavelet transform can then be used to identify time-varying frequency of infrastructure when it is subjected to impulse load, or even wind load (wide-band random process). However, wavelet transform was never used to identify frequency variation of infrastructure when it is subjected to strong earthquake. The reason is that structural seismic response contains frequency component of the earthquake excitation. It is difficult to distinguish frequency of the structure itself from the frequency of earthquake because most earthquakes are narrow-band stochastic process.

To solve this problem, the concept of WETFS is proposed in this paper. Since the dilation parameter α and translation parameter b have clear relationship with frequency ω and time t of the signal, then the wavelet coefficient can also be expressed as $W_\psi(\omega,t)$. First, the continuous wavelet transform are performed for the structural seismic response and earthquake excitation, respectively, as expressed in Eqs. 6 and 7.

$$W_\psi(a,b)_{output} = \int_{-\infty}^{+\infty} y(t)_{output}\psi^*_{a,b}(t)\mathrm{d}t \tag{6}$$

$$W_\psi(a,b)_{input} = \int_{-\infty}^{+\infty} x(t)_{input}\psi^*_{a,b}(t)\mathrm{d}t \tag{7}$$

where $y(t)_{output}$ and $x(t)_{input}$ denote structural seismic response and earthquake excitation signal respectively; $W_\psi(a,b)_{output}$ and $W_\psi(a,b)_{input}$ are wavelet transform coefficients of $y(t)_{output}$ and $x(t)_{input}$. The Gabor wavelet function is adopted in this paper because it has the optimum time and frequency resolution. The Gabor wavelet can be written as

$$\psi(t) = \frac{\mathbf{1}}{(\sigma^2\pi)^{1/4}} e^{-t^2/(2\sigma^2)} \cdot e^{i\eta t} \tag{8}$$

where parameter σ and the initial scale define the time and frequency spread of the Gabor wavelet function, and η is the parameter of frequency modulation. The scalogram of $W_\psi(a,b)_{input}$ contains frequency components of earthquake, while the scalogram of $W_\psi(a,b)_{output}$ involves not only the natural frequencies of the structures but also frequencies of earthquake. Therefore, it is difficult to get natural frequencies of structures by simply performing wavelet transform on the output response data only.

Similar to the concept of Fourier transfer function, the transfer function in the wavelet domain is defined as the ratio of the wavelet coefficients of output response to that of input excitation of a system. However, the wavelet coefficients of a signal are so susceptible to noise. Consequently, the transfer function in the wavelet domain is also infected by noise. A reasonable transfer function in wavelet domain cannot be directly obtained through the wavelet coefficients of input excitation being divided by that of the output responses. To solve this problem, the wavelet coefficients $W_{\psi}(\omega,t)_{output}$ and $W_{\psi}(\omega,t)_{input}$ are first integrated along time to eliminate the effect of noise on the wavelet coefficients for small amplitude signal (ratio of signal to noise is small for this case). Finally, the wavelet energy spectra of the output and input are obtained, as expressed in Eqs. 9 and 10.

$$E_i(\omega, \; T_i)_{output} = \int_0^{T_i} W_{\psi}(\omega,t)_{output}\,\mathrm{d}t \tag{9}$$

$$E_i(\omega,T_i)_{input} = \int_0^{T_i} W_{\psi}(\omega,t)_{input}\,\mathrm{d}t \tag{10}$$

where T_i = duration from the initial time to sampling point i and $T_i = [\mathbf{0},T]$, T is the entire duration of earthquake excitation; $W_{\psi}(\omega,t)_{output}$ = wavelet coefficient of structural seismic response; $W_{\psi}(\omega,t)_{input}$ = wavelet coefficient of earthquake input; $E_i(\omega,T_i)_{output}$ = wavelet energy of structural seismic response up to sampling point i and $E_i(\omega,T_i)_{input}$ = wavelet energy of earthquake excitation up to sampling point i. The wavelet energy spectrum $E_i(\omega,T_i)_{output}$ still contains frequency information of structural seismic response and earthquake excitation, The ration of $E_i(\omega,T_i)_{output}$ to $E_i(\omega,T_i)_{input}$ is defined as wavelet energy transfer function, as expressed in Eq.11.

$$H(\omega,T_i) = \frac{E_i(\omega,T_i)_{output}}{E_i(\omega,T_i)_{input}} \tag{11}$$

where $H(\omega,T_i)$ is called the WETFS. Obviously, it is a function of time and structural natural frequencies because the frequency components of earthquake excitation have been removed already. Only the frequency information of structure itself is reserved. With the same meaning of the ridge of the wavelet scalogram, the ridge of the WETFS also describes the frequency variation of a structure. Detailed explanations of the various methods for ridge extraction can be found in (Ruzzene et al. 1997).

Since structural damage induces decrease in frequency, the change in structural freuqency is adopted as the index for assessing damage level of the building, as expressed in Eq. 12

$$D = \frac{\Delta f_{\max}}{f_{initial}} \times \mathbf{100\%} \tag{12}$$

where D = damage index; $\Delta f_{\max}$ = max value of the frequency variation and $f_{initial}$ = the initial frequency before damage occurs.

Application to The 6-story Masonry Building

Among 10 aftershocks the monitoring system on the 6-stroey masonry building has recorded, the seismic responses of the building on Dec 10 and Oct 24, 2008 aftershocks were strongest. The peak values of the measured ground acceleration (y direction) during these two aftershocks are 0.057 and 0.041 m/s2 respectively. While the peak values of acceleration response (y direction) on the roof floor are 0.296 and 0.135m/s2. For the other 8 aftershocks, the seismic responses of this building were much smaller. In order to assess structural damage and its evolution during these 10 aftershocks, the WETFS method are performed in analyzing the measured structural acceleration responses and ground accelerations. The frequency variation of this building during each aftershock is identified and analyzed. The WETFS methods are performed on the y directional data. The ratio of frequency variation is adopted as the index assessing the damage level of this building. The damage identification and assessment results for the Dec 10 aftershock are given first in the following subsections. After that, the damage evolution during the total 10 aftershocks is assessed.

Results for Dec 10 aftershocks

The aftershock occurred on Dec 10 in Qingchuan was the most severe one the building ever suffered until now (December 2008). Fig. 2 plots the structural seismic responses at each floor. Because the accelerometers on the sixth floor could not work well during the 10 aftershocks, then the accelerations at this floor is not displayed in this Figure. In Fig. 2, part (a) shows the absolute accelerations at each floor, while part (b) shows displacements that are obtained by double integration of the measured accelerations (part (a)).

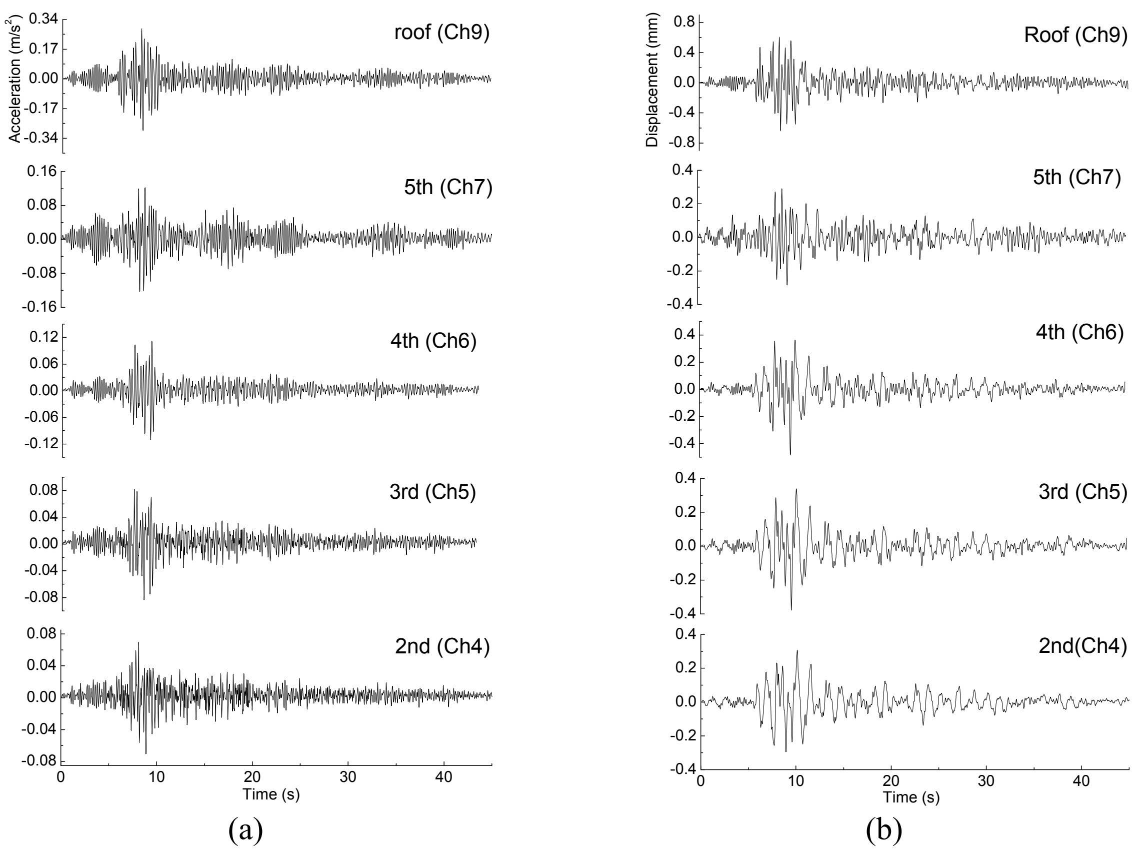
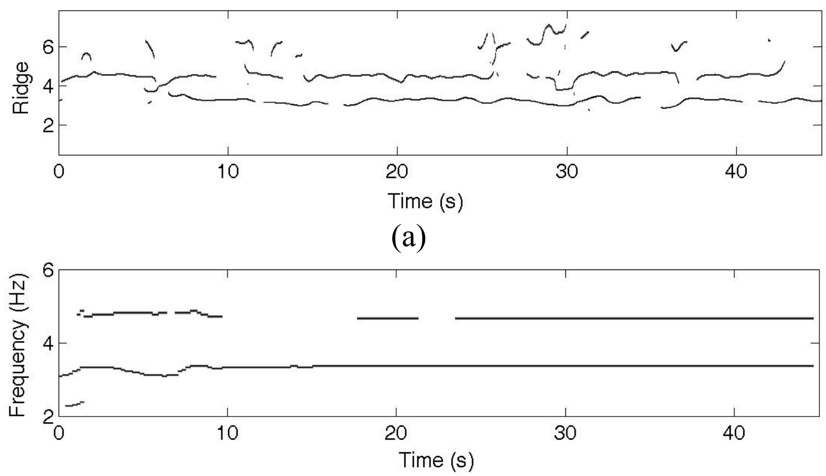


Figure 2. Structural seismic responses at each floor under the Dec 10 aftershock: (a) accelerations; and (b) displacements.

The WETFS method is used here to analyze the measured accelerations and identify the frequency variation process. Fig. 3 depicts the identified frequency by the WETFS method. Fig.3 (a) shows the ridges of the wavelet scalogram of the roof acceleration. It can be noted that the ridges fluctuate around the first two modal frequencies. But this fluctuation can not be interpreted as the results of stiffness variation due to structural damage. It is mainly due to the frequency interference of the aftershock excitation, so the structural frequency can not be identified accurately by the wavelet transform of the structural response only. Fig. 3 (b) displays the frequencies identified by the WETFS method, in which the effect of aftershock excitation has been removed. The first modal frequency decreases from 2s to 10s, and the structural response is also bigger during this period. The second modal frequency does not change evidently. According to the Eq. 10, the damage index for this aftershock is 8.31%. Then we can conclude that although the structural frequencies (i.e. the structural stiffness) recover to their original values due to the closing of cracks, the earthquake resistance ability of this building already degraded.



(b)

Figure 3. The identified frequencies under the Dec 10 aftershock: (a) ridge extracted from the wavelet scalogram of the roof acceleration; and (b) the identified structural natural frequency during the Dec 10 aftershock.

Damage Evolution during All the 10 Aftershocks

In order to address the damage evolution of this building during the 10 aftershocks above mentioned, the measured accelerations in these aftershocks are treated by the traditional Fourier transfer function method and the WETFS method respectively. Fig. 4 shows the accelerations at the roof floor of this building during all the 10 aftershocks (part (b)), along with the Frequency spectra derived by the Fourier transfer function (part (a)) and the frequency time histories identified by the WETFS method (part (c)). One can note that the structural responses in the Oct 4 aftershock are also relatively severe except for that in the Oct 24 and the Dec 10 aftershocks. For the convenience of comparison, the amplitude of the frequency spectra in part (a) is normalized with respect to the peak value of it. It is evident that the structural seismic responses under the five aftershocks (from Oct 15 to Oct 21) are dominated by the second modal shape. Then the variation rule of the first modal frequency in these spectra is not credible because of the small corresponding response components. While the seismic responses during the other five aftershocks (the Oct 3, Oct 4, Oct 24, Oct 25 and Dec 10 aftershocks) are dominated by the first and second modal shapes. The first two modal frequencies are both decreased during the Oct 24 and the Dec 10 aftershocks due to the intense seismic responses. The similar rule can also be observed in the frequency time histories in part (c). According to Eq. 10, the damage index during the Oct 24 aftershock is 7.9%, which is smaller than that of the Dec 10 aftershock. During the five aftershocks occurred from the Oct 15 to the Oct 21, the first modal frequency is hardly detected and its changing rule is not correct. In the other five aftershocks, however, the first two modal frequencies are both clear. The first modal frequency decrease evidently during the Oct 24 and the Dec 10 aftershocks when the acceleration amplitude is big. In the Oct 3, Oct 4 and Oct 25 aftershocks, the first modal frequency almost keeps constant because the structural seismic responses are relatively small. As for the second modal frequency in part (c), it does not change evidently.

Conclusions

A new method identifying frequency variation of buildings under earthquakes based on the new concept of wavelet energy transfer function is proposed in this paper. This method is then applied on the damage detection and evaluation of an actual 6-story masonry structure during 10 aftershocks. The main conclusions of this study are summarized as follows:

(1) The WETFS method can eliminate frequency contamination of earthquake excitation from the seismic responses of structures and can identify structural frequency variation of buildings during earthquake accurately.

(2) Frequency evolution of the 6-story masonry building during the 10 aftershocks is identified by the WETFS method. The natural frequency of this building decreased 7.9% and 8.31% respectively in the Oct 24 and Dec 10 aftershocks, which means structural damage further deteriorated in these two aftershocks.

Acknowledgments

This study is financially supported by NSFC with Grant Nos 50178025, 50538020 and 50278029, the National Distinguished Youth Funds with Grant No 50525823, the National Basic Research Program of China (973 Program) with Grant No. 2007CB714204 and the Basic Research Program of Institute of Engineering Mechanics, China Earthquake Administration with Grant No. 2007B15.

References

Cawley, P., and R. Adams, 1979. The location of defects in structures from measurements of natural frequencies, *Journal of Strain Analysis* 14(2), 49-57.

Hassiotis, S., and G. D. Jeong, 1991. Assessment of structural damage from natural frequency measurement, *Computers and Structures* 49(4), 79-691.

Hearn, G., and R. B. Testa, 1991. Modal analysis for damage detection in structures, *Journal of Structural Engineering* 117(10), 3042-3063.

Kareem, A., and T. Kijewski, 2002. Time-frequency analysis of wind effects on structures, *Journal of Wind Engineering and Industrial Aerodynamics* 90, 1435-1452.

Ruzzene, M., A. Fasana, L. Garibaldi, and B. Piombo, 1997. Natural frequencies and dampings identification using wavelet transform: application to real data. *Mechanical Systems and Signal Processing* 11(2), 207–218.

Todorovska, M. I. et al. 2008. Impulse response analysis of the Van Nuys 7-story hotel during 11 earthquakes and earthquake damage detection, *Structural Control and Health Monitoring* 15, 90-116.

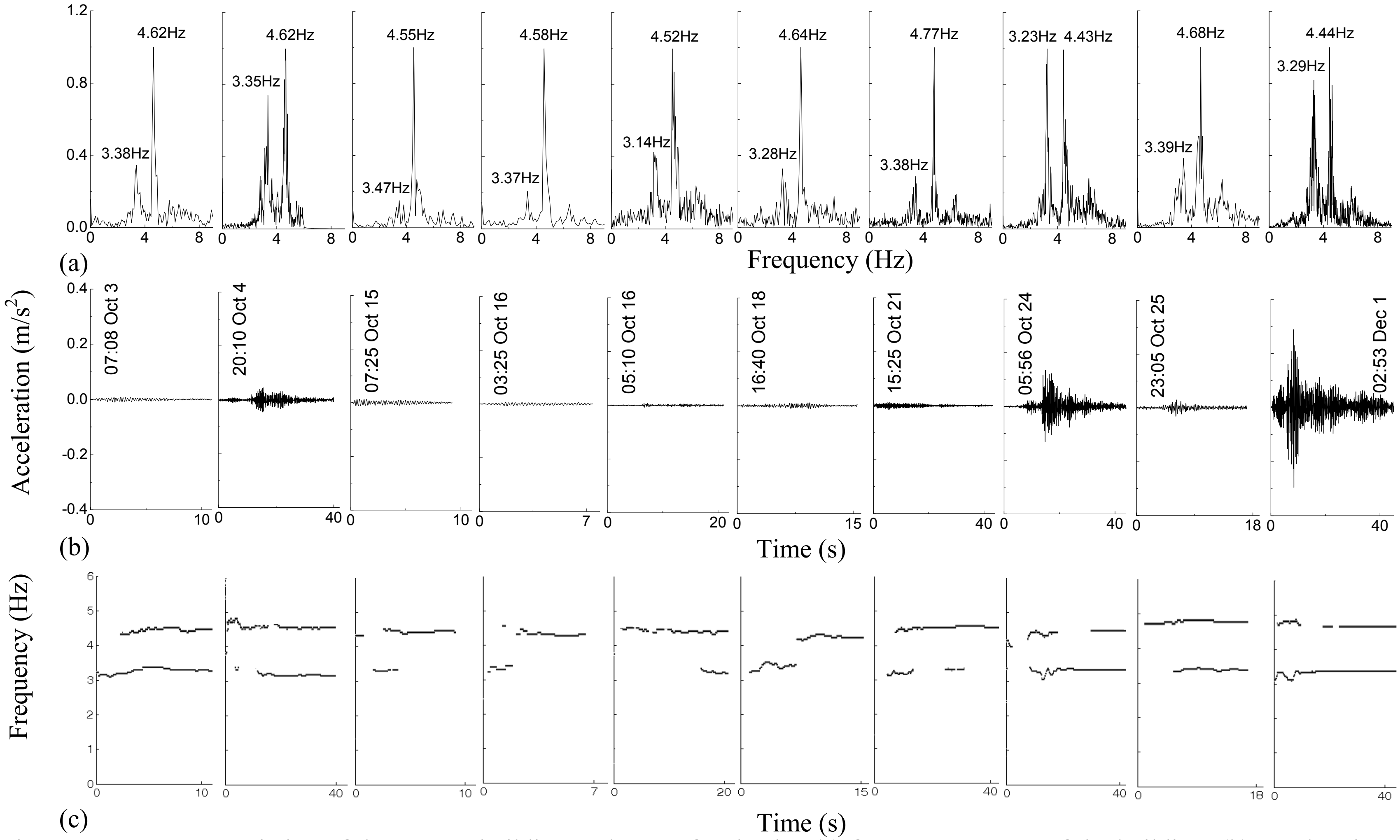


Figure 4. Frequency variation of the 6-story building under 10 aftershocks: (a) frequency spectra of the building; (b) accelerations at the roof floor; and (c) frequency variation identified by the WETFS method.