Wave propagation in a seven-story reinforced concrete building: III. Damage detection via changes in wavenumbers

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Abstract

This paper presents low frequency wavenumbers in a seven-storey reinforced concrete building estimated from its recorded response to eleven earthquakes, one of which (1994 Northridge) caused visible structural damage, and two of which are its aftershocks. The wavenumbers, $K_i(f)$, are estimated from pairs $(i,j)$ of records at neighboring recording sites in the building, distributed vertically or horizontally. Changes in $K_i(f)$ from one event to another are compared in the undamaged (lower) and in the damaged (upper) part of the building, with the aim to find whether trends in $K_i(f)$ can indicate damage. The results suggest significant and permanent increase of the wavenumbers in the damaged parts for the 1994 Northridge earthquake and its aftershocks, which is not the case for the other events in the damaged parts, and for all eleven events in the undamaged parts of the building. This increase in wavenumbers in the damaged parts can be explained by reduced wave velocities through the damaged structural members, and by scattering of waves from the discontinuities created by the damage. It is concluded from this qualitative analysis that wavenumbers estimated from strong motion recordings in a building can indicate location of damage, and that it would be useful to refine further this method (extend it to higher frequencies, and add the capability to quantify the damage). However, this would require more dense strong motion instrumentation in buildings than currently available. Deployment of dense arrays in selected buildings would provide data for further work on this subject.

Keywords: Damage detection; Wave propagation in structures; Strong motion data; 1994 Northridge earthquake

1. Introduction

The aim of damage detection and health monitoring of structures is first to find out whether damage has occurred, and, if it has, to determine its location and severity, and to find what is the remaining strength (life) of the structure. The methods for damage detection involve visual inspection and experimental measurements. The interest to develop global damage detection methods to identify damage of structure during and following strong earthquake shaking has often resulted in some adaptation or generalization of methods used in mechanical, nuclear, aerospace and astronautical engineering \cite{1,2}. By far the most common approach has been to examine the changes in the vibrational characteristics of the structure \cite{3}.

There are two aspects of damage detection and structural health monitoring in earthquake engineering which are very different from those in the mechanical, aerospace and aeronautical engineering. The first one is that buildings, bridges, dams, etc. are built on or near the ground surface, and are supported through the contact with the soil, over large and multiple foundation areas. Consequently, their dynamic response is affected by soil–structure interaction, which requires advanced modeling and analysis methods \cite{4}. Ignoring the interaction with the soil leads to assigning observed changes in stiffness of the soil–foundation system to the structure, and consequently leads to erroneous inference about damage in the structure when the observed changes in stiffness are primarily due to changes in the soil \cite{5,6}. The changes of the soil stiffness (time and amplitude dependent) can be (and usually are) much larger than the changes of the structural stiffness caused by ‘minor’ or ‘partial’ damage of the structural members \cite{5–8}. This requires damage identification schemes such that it is possible to separate (isolate) the changes in structural stiffness from the changes of the soil–foundation stiffness. The second important difference is that space structures and
many mechanical systems are finite in all three dimensions, and their vibrational characteristics are determined completely by their discrete frequencies and modes of vibration. In earthquake engineering, the soil supporting the structure can be assumed to be extending to infinity. This results in mode shapes with continuously distributed frequencies, which calls for methods of identification based on wave propagation.

Moderate and minor damage of the structure itself leads to small changes of its characteristic frequencies (e.g. less than 10%), which are difficult to identify. Therefore, it is useful to search for new damage detection methods based on wave propagation techniques rather than on changes in the characteristic frequencies. After damage has occurred, the velocities of propagation of shear waves, within the damaged zones are reduced. These changes can be detected from changes in propagation time [9], or from changes of frequency dependent wavenumbers, \( k = \omega c(\omega) \), where \( \omega \) is circular frequency and \( c(\omega) \) is frequency dependent phase velocity [10,11]. Methods for detection of damage in well instrumented single columns have been proposed [12], but extension and verification of such methods to full-scale structures will become possible only when dense arrays for recording structural response are deployed and strong motion data from these arrays becomes available.

This paper presents wavenumbers of shear waves in a seven-story reinforced concrete building in Van Nuys, California, estimated (for low frequencies) from recorded response to 12 earthquakes, one of which caused visible structural damage (1994 Northridge), and two of which (Northridge aftershocks) occurred before the building was repaired. The aim is to show that the phase velocities through the damaged portions of the building were reduced (i.e. the wavenumbers increased), and that such changes are measurable from recorded strong motion response. At present, no two- or three-dimensional theories exist that can interpret and quantify changes in wavenumbers in a structure. To develop and test such theories, data from dense arrays of strong motion sensors in structures are needed. This paper, hence, shows only the feasibility of such theories and methods for damage detection.

This paper (Paper III) is a continuation of our papers on wave propagation in the same building [10,11]. Paper I described the building, presented the theoretical wave propagation models of the building represented as anisotropic homogeneous or layered continuum, and estimated the shear wave velocities of the homogeneous anisotropic models from ambient vibration test data. Paper II presented the strong motion data used in the analysis, the methodology used to estimate wavenumbers of vertically and horizontally propagating shear waves through the building from the recorded earthquake response, and results for the experimentally estimated wavenumbers from pairs of recordings in the part of the building that was not damaged by the

![Fig. 1. The site of the seven-story reinforced building in San Fernando Valley of the Los Angeles metropolitan area, relative to the fault planes of the 1971 San Fernando [13] and 1994 Northridge [14] earthquakes, and relative to the major freeways. The arrows show the directions towards the epicenters of the earthquakes recorded in the building that are outside the area shown in this figure.](image)
Fig. 2. (a) Typical floor plan. (b) The foundation plan. (c) Typical transverse section. (d) Log of typical soil boring.
Northridge earthquake. The results showed that the empirical wavenumbers are consistent from one earthquake to another and are of the order of magnitude of those estimated from ambient vibration tests and from design details. In this paper, using the same methodology, we present the empirical wavenumbers for the damaged part of the building and compare those with the wavenumbers for the undamaged part. We also present a brief description of the building and strong motion data, and the methodology as a background for this paper.

2. The building and the data

2.1. The building

The instrumented seven-story reinforced concrete building is located in the city of Van Nuys in San Fernando Valley of metropolitan Los Angeles. Fig. 1 shows the building site relative to the major freeways and the rupture areas of the 1971 San Fernando [13] and 1994 Northridge [14] earthquakes, which both damaged the building. It was constructed in 1966 [15], and has been serving as a hotel. Fig. 2 shows (a) a typical floor plan, (b) the foundation plan, (c) a typical transverse section, and (d) log of a typical soil boring. Its plan dimensions are 62.7 × 150 ft² and its height is 65.7 ft (Fig. 2(a)). The typical framing consists of columns spaced at 20 ft centers in the transverse direction and 19 ft centers in the longitudinal direction. Lateral forces in each direction are resisted by the interior column-slab frames (NS) and exterior column spandrel beam frames (EW). The structure is essentially symmetric. The foundation system (Fig. 2(b)) consists of 38 in. deep pile caps, supported by groups of two to four poured-in-place 24 in. diameter reinforced concrete friction piles. These are centered under the main building columns. All pile caps are connected by a grid of the beams. Each pile is about 40 ft long and has design capacity of over 100 kips vertical load and up to 20 kips lateral load. The structure is constructed of regular weight reinforced concrete [15]. The site lies on recent alluvium. A typical boring log (Fig. 2(d)) shows the underlying soil to be primarily fine sandy silts and silty fine sands. The average shear-wave velocity in the top 30 m is 300 m/s.

The February 9, 1971, San Fernando earthquake (Fig. 1) caused minor structural damage. Epoxy was used to repair the spalled concrete of the second floor beam column joints.

<table>
<thead>
<tr>
<th>No.</th>
<th>Earthquake</th>
<th>Date</th>
<th>M</th>
<th>∆ (km)</th>
<th>NS $v_{\text{max}}$ (cm/s)</th>
<th>EW $v_{\text{max}}$ (cm/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>S. Fernando</td>
<td>9 February 1971</td>
<td>6.6</td>
<td>10°</td>
<td>26.86</td>
<td>22.63</td>
</tr>
<tr>
<td>2</td>
<td>Whittier-Narrows</td>
<td>1 October 1987</td>
<td>5.9</td>
<td>41</td>
<td>8.42</td>
<td>6.35</td>
</tr>
<tr>
<td>3</td>
<td>W.N. aftershock</td>
<td>4 October 1987</td>
<td>5.3</td>
<td>41</td>
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<td>2.18</td>
</tr>
<tr>
<td>4</td>
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<td>3 December 1988</td>
<td>4.9</td>
<td>32</td>
<td>1.57</td>
<td>0.94</td>
</tr>
<tr>
<td>5</td>
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<td>19 January 1989</td>
<td>5.0</td>
<td>35</td>
<td>1.00</td>
<td>0.96</td>
</tr>
<tr>
<td>6</td>
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<td>34</td>
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<td>0.85</td>
</tr>
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<td>7</td>
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<td>5.8</td>
<td>44</td>
<td>4.49</td>
<td>2.78</td>
</tr>
<tr>
<td>8</td>
<td>Landers</td>
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<td>7.5</td>
<td>186</td>
<td>11.40</td>
<td>10.63</td>
</tr>
<tr>
<td>9</td>
<td>Big Bear</td>
<td>28 June 1992</td>
<td>6.5</td>
<td>149</td>
<td>3.90</td>
<td>3.58</td>
</tr>
<tr>
<td>10</td>
<td>Northridge</td>
<td>28 June 1992</td>
<td>6.4</td>
<td>4°</td>
<td>38.38</td>
<td>50.93</td>
</tr>
<tr>
<td>11</td>
<td>N. Aftershock</td>
<td>20 March 1994</td>
<td>5.2</td>
<td>1</td>
<td>7.79</td>
<td>4.83</td>
</tr>
<tr>
<td>12</td>
<td>N. Aftershock</td>
<td>6 December 1994</td>
<td>4.3</td>
<td>11</td>
<td>3.16</td>
<td>2.41</td>
</tr>
</tbody>
</table>

* Horizontal projection of the closet distance to fault surface.
on the north side and east end of the building. The nonstructural damage was extensive. The building was damaged again by the 17 January, 1994, Northridge earthquake (Fig. 1). Fig. 3 shows a schematic representation of the damage and of the location of the sensors. The structural damage was extensive in the exterior north (D) and south (A) frames, designed to take most of the lateral load in the longitudinal (EW) direction. Severe shear cracks occurred at the middle columns of frame A, near the contact with the spandrel beam of the fifth floor. Those cracks significantly decreased the axial, moment and shear capacity of the columns. The shear cracks, which appeared in the north (D), frame on third and fourth floors, and the damage of columns D2, D3 and D4 on the first floor were minor. No major damage of the interior longitudinal (B and C) frames was noticed. There was no visible damage in the slabs and around the foundation [16].

2.2. The data

The first digitized strong motion accelerograms in the building were recorded on the February 9, 1971, during San Fernando earthquake (Fig. 1; Table 1) [17]. We do not use the records of this earthquake for this analysis because of inadequate location and number of sensors [16]. We use records from eleven other events, listed in Table 1 (for each event, the date, magnitude, epicentral distance and peak horizontal velocity at the ground floor are included). All of these responses were recorded by a CR-1 system, with the sensor locations shown in Fig. 4. The records of the Whittier Narrows, Landers, Big Bear and Northridge earthquakes were processed by the California Division of Mines and Geology [16,18]. The directions of wave arrivals for all of the eleven events is shown in Fig. 1. The 1987 Whittier-Narrows, 1992 Landers and 1992 Big Bear earthquakes occurred at epicentral distances of 41, 186 and 149 km, respectively, and caused wave arrivals from E27°S, East, and E1.5°S. During the 1994 Northridge earthquake, the first motions started to arrive from the West, with the last arrivals coming from N42°W, about 7–10 s later [14].

A discussion on the accuracy of the processed data for the purposes of this analysis can be found in Ref. [11].

3. Methodology

3.1. Estimation of wavenumbers from earthquake response data

Because it takes time for wave interference to occur, the motion in the building consists of standing waves and propagating waves. Todorovska et al. [11] used the following equation to estimate the wavenumbers of propagating waves in the building between two recording points, $i$ and $j$, from strong motion data

$$K_{ij}(\omega) = \left| \frac{\mathcal{F}\left(\frac{v_i(t) - v_j(t)}{D}\right)}{\mathcal{F}\left(\frac{v_i(t) + v_j(t)}{2}\right)} \right|$$

where $\mathcal{F}(\cdot)$ indicates Fourier Transform, $\omega$ the circular frequency, $v_i$ the displacement recorded at channel $i$, and $D$ the separation distance between the two recording points. For long wavelengths compared to the separation distance, and for frequencies away from the modal frequencies $K_{ij}(\omega)$ will be equal, approximately to the wavenumber of the propagating shear wave between points $i$ and $j$, $k(\omega) = \omega c(\omega)$, and near the modal frequencies, $K_{ij}(\omega)$ will be the average slope of the mode-shape between the two points. For small $\omega$ and/or slowly changing $c(\omega)$, $K_{ij}(\omega)$ for propagating waves can be estimated from the slope of $K_{ij}(\omega)$. For example, for a propagating wave with constant $c$, $K_{ij}(\omega)$ will be a straight line with slope $1/c$.

The approximation of wavenumbers by Eq. (1) is valid only for wavelengths $\lambda > 4D$ which implies

$$K_{ij}(\omega) \leq \frac{\pi}{2D}$$

For horizontal wave propagation along the floor slabs or at the ground floor, $D = L = 40$ m implying $K_{ij} \leq 0.04$ m$^{-1}$. 

Fig. 4. Locations of the sensors for channels 1–16.
For vertical wave propagation between the first and second floors, \( D = h = 4.1 \text{ m} \) implying \( K_{ij} < 0.4 \text{ m}^{-1} \). Similarly, for vertical wave propagation between the second and third floors, \( D = h = 2.7 \text{ m} \) implying \( K_{ij} < 0.6 \text{ m}^{-1} \).

The accuracy of computing \( K_{ij} \) by Eq. (1) depends on the accuracy of relative timing of the recorded motions at channels \( i \) and \( j \). A discussion on this for film records can be found in Ref. [11].

4. Results

In this section we first review results for the frequencies and shear wave velocities and phase velocities for this building from our earlier work, and then present new results for the wavenumbers in the damaged part of the building.

4.1. Summary of results of previous studies

Todorovska et al. [10] calibrated the parameters of an equivalent anisotropic shear plate model of the building using the ambient vibration tests data for this building [7, 8], and obtained, for vibrations in the NS direction, vertical shear wave velocity \( \beta_z = 112 \text{ m/s} \) and anisotropy factor \( \beta_x/\beta_z = 0.55 \), and for vibrations in the EW direction, vertical shear wave velocity \( \beta_z = 88 \text{ m/s} \) and anisotropy factor \( \beta_x/\beta_z = 1 \). These are lower bounds because the ambient vibration tests gave the frequencies of
the soil–structure system which are smaller than the fixed base frequencies of the building. The shear-wave velocity through the slabs was estimated to be about 2000 m/s.

Todorovska et al. [11] computed wavenumbers in the lower part of the building (between the first and second floor and between the second and third floors) using Eq. (1), which implied phase velocities between 50 and 150 m/s. These estimates are valid for small frequencies (0–6 Hz for the space between the first and second floor, and 0–9 Hz for the space between the second to third floors), and are in qualitative agreement with the vertical shear wave velocities estimated from ambient vibration data [10]. The lower part of the building did not suffer major structural damage.

Application of the same procedure to wave propagation in the horizontal direction, along the stiff floor slabs, implied phase velocities in the range between 500 and 2000 m/s, also consistent with the estimate in Ref. [10]. On the ground floor, along the longitudinal axis of the building, analysis of the motions recorded by channels 1 and 13 (Fig. 4) implied apparent EW phase velocities for the ground floor in the range 0.5–2.5 km/s. These values are of the same order as the phase velocities of Love surface waves in the shallow sediments of San Fernando Valley. Ivanovic et al. [9] showed that the observed time delays between motions at channels 1 and 13 were also consistent with expected direction of wave approach during the same earthquakes. Trifunac et al. [19] analyzed amplitude ambient vibration test data on the ground floor and around the building. Their study suggested horizontal and vertical warping of the foundation, during the passage of ground noise waves.

<table>
<thead>
<tr>
<th>Earthquake</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>Landers 1992</td>
<td></td>
</tr>
<tr>
<td>Big Bear 1992</td>
<td></td>
</tr>
<tr>
<td>Northridge 1994</td>
<td></td>
</tr>
<tr>
<td>Northridge aft. March 1994</td>
<td></td>
</tr>
<tr>
<td>Northridge aft. Dec. 1994</td>
<td></td>
</tr>
<tr>
<td>Whittier Narrows 1987</td>
<td></td>
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<tr>
<td>Whittier Narrows aft. 1987</td>
<td></td>
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<tr>
<td>Pasadena 1988</td>
<td></td>
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<tr>
<td>Malibu 1989</td>
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<tr>
<td>Montebello 1989</td>
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<tr>
<td>Sierra Madre 1991</td>
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</tbody>
</table>

Fig. 6. Wavenumber estimates $K_{ij}$ in cm$^{-1}$, computed by Eq. (1), for EW response for eleven earthquakes (Table 1). The vertical grey bands show the frequency limits computed from by Eq. (2). The short vertical bars at 1.0, 3.5, 5.7 and 8.1 Hz. show the apparent frequencies of EW translation identified from ambient vibration tests.
believed to consist primarily of high frequency Rayleigh waves, implying apparent surface phase velocities of about 300 m/s.

4.2. Results of this study

Figs. 5 and 6 show $K_{ij}(f)$ in cm$^{-1}$ ($f$ is frequency in Hertz) for various combinations of neighboring channels, for frequencies that satisfy the inequality (2), and for 11 earthquakes. Fig. 5 corresponds to transverse (NS) vibrations and Fig. 6 to longitudinal (EW) vibrations. The results for combinations of channels below the third floor, and for the 1987 Whittier-Narrows, 1992 Landers, 1992 Big Bear, and 1994 Northridge earthquakes were discussed previously by Todorovska et al. [11]. The results for $K_{ij}(f)$ below the third floor are included in these figures because of interest for this paper is comparison between $K_{ij}(f)$ in the parts of the building which did not suffer major structural damage (below the third floor) and the parts that did suffer structural damage (above the third floor).

It is seen that the overall trend of all $K_{ij}(f)$ is an increase with $\omega$ approximately following a straight line, with slope $(2\pi c/c)$ implying $c$ between $\sim 50$ and $\sim 100$ m/s for vertical wave propagation, and between 500 and 2000 m/s for horizontal wave propagation. Many local peaks are superimposed to the general linear trend. Some of these peaks could be interpreted to be associated with the soil–structure system frequencies, which are standing waves. These frequencies are 1.4, 1.6, 4.2 and 4.9 Hz for NS translations and 1.0, 3.5, 5.7 and 8.1 Hz for torsion. The group of peaks near and below 1 Hz corresponds to the first system frequency, which shifts towards lower frequencies for larger excitation amplitudes, indicating nonlinear system response. The shifts of the system frequency were interpreted by Trifunac et al. [5,6] to be mostly due to nonlinear soil response.

Now we look for features in $K_{ij}(f)$ in the upper part of the building, for channel pairs $(2,5)$, $(3,4)$ and $(4,6)$, which we know was damaged (the structural damage from the 1994 Northridge earthquake could be seen by a naked eye; Fig. 3), that would indicate damage. We expect these features to be different for different response amplitudes, and to vary with time, during the strong motion phase of response. We recall that our analysis of data from two ambient vibration tests of this building, performed 2 weeks and 2 months after the damage occurred, proved unsuccessful in detecting the location of the observed damage [7]. These test were far more detailed than most ambient vibration tests of such a simple and symmetric buildings. We were able to identify some features in the modeshapes which could be associated with damage of columns at fifth floor (Fig. 3), but this interpretation was possible only because we knew where

Fig. 7. Enlargement of $K_{3,4}$, $K_{4,6}$, $K_{8,13}$ and $K_{2,5}$ in cm$^{-1}$ from Fig. 5 (NS response). It is seen that $K_{ij}$ for Northridge earthquake and its two aftershocks are different and larger then those for the eight smaller and nondamaging events.
the damage was. The lessons of our investigation lead to the conclusion that only more advanced work and more detailed ambient vibration tests in future may be able to do better.

Figs. 7 and 8 show enlargements of Figs. 5 and 6 for the upper parts of the building. These figures show systematic departures from the straight line trend of $K_{ij}$ for the 1994 Northridge earthquake and its two aftershocks (i.e. when the building was damaged), in contrast to $K_{ij}$ for the smaller amplitude responses during the seven events preceding the Northridge earthquake. It is striking how smooth and repeatable $K_{ij}$ can be for the seven events preceding the Northridge earthquake (see $K_{3,4}$, $K_{4,6}$ and $K_{9,10}$). This obvious difference indicates that wave propagation methods may be far superior to modal methods in identification of damaged zones.

5. Discussion and conclusions

Figs. 5–8 showed that, for moderate and small ground motion amplitudes (in this study for peak ground velocities between 1 and 10 cm/s), the waves propagate through the structure in essentially a linear manner. This leads to $K_{ij}$ which tends to be similar from one excitation to another, and which on the average increases linearly with frequency.

During Northridge earthquake of 17 January 1994, the building experienced significant damage. This resulted in increased values of $K_{ij}$ (i.e. slower phase velocities), in selected frequency bands and for station pairs surrounding the observed damage. These changes are similar for excitation by the main event and by two of its aftershocks, suggesting that once the damaged zones were created, both ‘small’ (two aftershocks) and ‘large’ (main event) wave amplitudes experienced similar delays in propagation time and similar reflections from the damaged zones. The building was braced prior to March 20, 1994 aftershock [16] to prevent progression of major damage during aftershocks of Northridge earthquake, but the braces may have been disengaged during small shaking, thus ‘allowing’ the incident waves during both aftershocks to sample essentially the ‘same damage’ as during the main event. Overall, the changes of $K_{ij}$ following the Northridge
earthquake are consistent with the interpretation that these changes indicate permanent damage. This is indicated by similarity of $K_{ij}(\omega)$ for excitations by the main event and by the two aftershocks. This also suggests that $K_{ij}$ are mostly sensitive to local changes between the two recording stations $i$ and $j$. In contrast, the changes of overall rocking amplitudes of $\theta_x$ and $\theta_y$ of the whole building, which display significant changes of the predominant system frequency, $f_p$, (Figs. 9 and 10) are governed by the overall excitation amplitudes and show ability of the soil–structure system to regenerate its rigidity during shaking by small events. Trifunac et al. [5,6] interpreted those changes to result mainly from nonlinear stiffness of the soil–pile foundation, with small to insignificant contribution from damage within the structure. This should help visualize why it is so difficult to identify permanent changes and damage in the building from overall changes in system frequency.

In this paper, we used the elementary low frequency approximation for computing wavenumbers within the building. Our aim was to explore whether the observed changes in $K_{ij}$ can be used to identify the presence of damage which is located between the two recording stations $i$ and $j$. We conclude that this attempt was successful. The next step, therefore, will be to refine calculation of $K_{ij}$ by developing more advanced methods which will be able to describe the wavenumbers in broader frequency band and which may be capable of showing their time dependent changes.

We conclude that the relative changes in the observed wavenumbers may become a useful tool for locating and interpreting the effects of damage, as well as for monitoring the structural health, provided, of course, that adequate instrumentation is available [20,21].

References


