VARIABILITY OF THE FIXED-BASE AND SOIL-STRUCTURE SYSTEM FREQUENCIES OF A BUILDING – THE CASE OF BORIK-2 BUILDING

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ABSTRACT

Borik-2 is an IMS type prefabricated 14-story reinforced concrete building located in Banja Luka, Republic of Srpska (Bosnia, former Yugoslavia), and is a rare example of an instrumented building in Europe shaken by a significant number of earthquakes. This paper presents an analysis of its response to 20 earthquakes recorded in this building, and a comparison with results from previous full-scale tests and analyses. Only one of the 20 earthquakes (8/13/1981, M=5.4) could possibly have caused damage, but no structural damage was reported. For each of these earthquakes, the building fundamental fixed-base frequency $f_1$ was computed from wave propagation travel times estimated by impulse response functions, and the soil-structure system frequency $f_{sys}$ was estimated from the peaks of the Fourier spectra of the response. The analysis suggests consistency of the estimates of $f_{sys}$ from the earthquake response data, from the forced vibration test before the earthquakes and ambient vibration tests conducted near the end of the earthquake sequence. The results suggest nonlinear but essentially “elastic” behavior of the building for the amplitudes of motion covered by the data, and essentially linear soil-

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structure interaction. During the largest event, $f_1$ and $f_{sys}$ decreased respectively by about 16% and 18% for EW motions, and by about 22% and 31% for NS motions, compared to the values before the earthquake from the small amplitude response. Comparison of $f_1$ and $f_{sys}$ during the smaller events before and after EQ 11 event shows that $f_1$ did not change, but $f_{sys}$ reduced permanently, by about 10%.

**Keywords**: Earthquake damage detection; structural health monitoring; wave propagation in structures; impulse response analysis; variation of building frequencies of vibration; Borik-2 building in Banja Luka; IMS type prefabricated construction.

**INTRODUCTION**

Full-scale tests of structures and records of their response to repeated earthquake excitation over an extended period of time are invaluable for testing and calibration of structural identification and health monitoring methods, and for understanding how the damage sensitive features, in real structures and under real life conditions, vary due to factors other than earthquake damage (e.g., age of the structure, prior seismic exposure, level of response, various environmental factors such as temperature and rainfall, and changes in the soil supporting the structure) [1-3]. The latter is referred to as “operational variability” of the damage sensitive features, and has been recognized as one of the critical unresolved issues that need to be addressed before structural health monitoring systems can be routinely deployed in practice [3]. Despite the obvious value of full-scale data, literature reviews on this topic reveal that the majority of the vibrational methods are tested only on analytical models [1-4].

The number of buildings for which records are available of response to significant earthquake shaking (such that has caused damage or could have caused some damage) is quite small worldwide, due to the fact that only a fraction of buildings are instrumented, and because such
shaking is a rare event. The number of structures for which *multiple* earthquake records are available is even smaller, because the strong motion instrumentation programs typically release only the data beyond some threshold level. Another problem is that detailed supporting information is not available for many buildings. In view of this, multiple earthquake records in *well documented* buildings, including significant events, are particularly valuable. This paper presents an analysis of such a case study.

Borik-2 is a prefabricated instrumented 14-story reinforced concrete building located in Banja Luka, Republic of Srpska (Bosnia, former Yugoslavia), which has recorded many earthquakes, and has been studied extensively as an important construction type. Figure 1 shows a photo of a southwest view of the building. This paper presents an analysis of the variations of its frequencies of vibration using records of 20 earthquakes over a period of 12 years (between April of 1974 and October of 1986). The strong motion array in this building was part of the Yugoslav Strong Motion Network, which was operated by the Institute of Earthquake Engineering and Engineering Seismology, Skopje, Macedonia (also known by the acronym IZIIS) [5]. Figure 2 shows the location of the building within (a) the area of former Yugoslavia, and (b) a larger view of the area near Banja Luka, also showing the epicenters of 16 of the contributing earthquakes, which have been identified. The rectangles in part (a) show areas with concentrated earthquake activity between the mid-1970s early 1980s [5-7]. The earthquake records were digitized and processed by the authors in the summer of 2006. During one of the events analyzed (August 13, 1981, $M=5.4$), the response approached a damaging level, but no structural damage was reported after the earthquake. This was the largest shaking that this building experienced over the seismic monitoring period covered by the IZIIS archive (1972 to 1990). Forced vibration tests and one ambient vibration test have also been conducted in this building, and can be used for relative comparison with results from the earthquake response analysis.
The structural system is of the IMS type, which has been used extensively in the former Yugoslavia since the 1960s. It can be found in many major cities like Belgrade (50% of apartments in New Belgrade), Novi Sad, Niš, Banja Luka, Sarajevo, Tuzla, etc., as well as in China, Cuba, Egypt, Georgia, the Philippines, and Russia. It is estimated that to date more than 400,000 housing units (~2.5 million m² of built area) have been constructed using this system. Consequently, it was extensively tested and continues to be developed further at the Institute for Testing of Materials in Belgrade, Serbia. IMS is a prefabricated construction system consisting of precast concrete columns, waffle floor slabs, edge girders, stairs, and wall panels. The frame structure carries gravity loads, while shear walls are the main lateral load-resisting elements. The main characteristic of this technology is that the key structural elements are joined together by pre-stressing in the two orthogonal horizontal directions.

Two types of analyses were carried out in this paper: (1) of travel times of seismic waves propagating through the building, and (2) of energy distributions of the response (Fourier or windowed Fourier). The wave travel times were estimated by tracing impulses radiated from virtual sources created by deconvolution of the recorded seismic response [8-11]. As the speed of propagation of seismic waves through the structure (hence the time it takes for the virtual pulse to propagate from one level of the building to another) physically depends only on the properties of the building itself, and not on the properties of the foundation soil, structural health monitoring by detecting changes in these wave travel times eliminates the soil-structure interaction as a factor producing the same effect on the damage sensitive feature as damage. This is a significant advantage of such methods over methods based on detecting changes in the frequencies of vibration as estimated from energy distributions of the response. The energy of the response of a structure on flexible soil is concentrated around the frequencies of the soil-foundation-structure system, i.e. the peaks of the roof response as well as of foundation rocking occur at the system frequencies [12,13]. These frequencies are different from the fixed-base
frequencies, the deference being more significant for stiffer structures compared to the dynamic stiffness of the foundation soil. The frequency associated with the fundamental mode of vibration of the building, referred to herein as \( f_{sys} \), is typically most affected. Consequently, methods for estimating the structural frequencies from the peaks in the Fourier spectra or transfer functions of the response give the soil-structure system frequencies. An important implication of this fact for structural health monitoring is that detected changes in these frequencies may as well be due to changes in the foundation soil, which is a granular material exhibiting nonlinear response during earthquake shaking. Another important consequence is that calibrating stiffness of analytical models of a structure using \( f_{sys} \) in lieu of the fundamental fixed base frequency \( f_1 \) underestimates the actual stiffness of the structure as \( f_{sys} < f_1 \).

In this paper, we use the wave travel times to estimate the building fundamental fixed-base frequency, based on an equivalent shear beam approximation of the building response, which implies \( f_1 = 1/(4 \tau_{tot}) \), where \( \tau_{tot} \) is the travel time of seismic waves from the point of fixity (ground level) to the roof. Based on this, \( f_1 \) can be estimated using data from only two horizontal sensors. While the goodness of this approximation of \( f_1 \) may vary from one building to another, the changes in \( f_1 = 1/(4 \tau_{tot}) \) will still depend only on changes in the building itself, and not on changes in the soil, and monitoring of changes in such an estimate of \( f_1 \) can be used as a global indicator of damage in a building [9,10]. Our recent study on earthquake damage detection in a 7-story reinforced concrete hotel in Van Nuys of the Los Angeles metropolitan area, using data from records of 11 earthquakes showed that \( f_1 = 1/(4 \tau_{tot}) \) was practically constant during the events that produced small amplitude response, while \( f_{sys} \) dropped significantly during two of these events [10]. We concluded that a change in \( f_1 = 1/(4 \tau_{tot}) \) is a
more reliable indicator of damage than a change in $f_{sys}$, which may lead to false alarms that damage has occurred.

The purpose of this study is: (1) to contribute a detailed case study augmenting the knowledge base on the variations of the frequencies of vibration of real buildings and the threshold change associated with structural damage, and (2) to test further the structural health monitoring method based on changes in wave travel times. The specific questions addressed and objectives are as follows. One objective of this study is (i) to use the Borik-2 data to find out if the observations for previously studied buildings hold for other buildings in general, or are specific to the buildings studied. Another objective is (ii) to further investigate, for a new case study, the relationship between $f_1 = 1/(4\tau_{tot})$ and $f_{sys}$, and their changes with time, and as functions of the level of response and prior exposure to earthquake shaking. The third objective is (iii) to obtain a calibration point for the threshold change in the building fundamental frequency that is associated with structural damage (here damage refers to substantial reduction of stiffness of the structure, beyond what is expected from normal service over its life). The fourth objective is (iv) to further investigate the robustness of measuring wave travel times in a building using pulses from virtual sources created by deconvolution of recorded earthquake response (same as impulse response functions). The last objective is (v) to examine the agreement of the results obtained for $f_{sys}$ and $f_1 = 1/(4\tau_{tot})$ using the earthquake response data from the 20 events studied in this paper with results from all known investigations of the same building. Because of the importance of the IMS system, the Borik-2 building, which was also instrumented, has been tested and studied extensively.

The authors can trace the wave approach of representing the seismic response of a building back to the 1930s [14-16]. There was a revived interest in the wave approach in the late 1980s
and early 1990s [17-19], late 1990s and early 2000s [20-29], and since the mid 2000s [8-11]. Although the vibrational approach is still prevailing, the interest in the wave methods is steadily increasing. The method of analysis of deconvolved earthquake response (giving impulse response functions) to obtain wave travel times has been applied to several buildings excited by small [8,11] as well as large amplitude earthquake shaking which has caused structural damage [9,10], and is closely related to measuring wave travel times and its changes using cross-correlation analysis, applied to earthquake damage detection in the Van Nuys building [22]. It is also closely related to the NIOM method, which essentially gives the impulse response function at a level in a building to a virtual input impulse at the roof, applied to both small earthquake response records [27,29] and to earthquake records in damaged buildings [28]. It is also related to the method of measuring wave travel times using dereverberated waves, applied to damage detection in a laboratory tests structure [26]. Damage detection based on changes in wave travel times was proposed by Safak [20] and demonstrated on numerically simulated data. All of these studies have shown that the wave travel times reflect the true distribution of stiffness in the structure, and their changes reflect the degree and spatial distribution of the observed damage.

Following this introduction, a brief description of the building is presented, a summary of prior forced and ambient vibration tests conducted on this building, and a summary of all known studies is presented to justify the agreements and to explain the discrepancies between the results from different studies. Such detailed comparisons of results by full-scale experiments, analytical models and earthquake observations can provide invaluable lessons for improvements of both the design of future full-scale experiments, and for the development of analytical models. Then, the strong-motion data used in this analysis is described, followed by a brief description of the methods of analysis used in this paper. Finally, the results of this study are presented and compared with results of all previous studies, and conclusions are drawn.
DESCRIPTION OF THE BUILDING AND SUMMARY OF PREVIOUS STUDIES

The Building and the Site

Borik-2 is a 14-story apartment building located in the settlement “Borik”, at 44° 46’ 15.35´´ North, and 17° 12´ 20.02´´ East, in the city of Banja Luka, Republic of Srpska (Bosna and Hercegovina). The building is $17.84 \times 17.84$ m in plan and has a basement (2.47 m high), 13 floors (each 2.80 m high), and a roof. The construction on the roof (terrace plus a housing for the lift equipment) is 3.40 m high. Figure 3 shows (a) the foundation layout, (b) a plan view of a typical floor, and (c) a cross-section view of the building frame. The foundation is a strip footing of uniform height, connected in the two orthogonal directions by a grid of beams. The foundation level for all strip footings is at 4.24 m depth relative to ground level. The typical framing consists of columns spaced at 4.20 m centers in both the transverse and longitudinal directions.

The building was constructed in 1972 using the IMS (Institute for Testing of Materials, Belgrade, Serbia) system of construction [30-33]. This system consists of prefabricated reinforced-concrete columns and floor diaphragms and cast-in-place reinforced-concrete shear walls. The connection between the columns and floor diaphragms is attained solely by the friction due to horizontal pre-stressing of the floor diaphragms. The floor diaphragms are reinforced-concrete prefabricated cassette ceilings with the total height of 22 cm at all floors. Spandrel beams surround the perimeter of each slab and comprise the cables for the horizontal pre-stressing of the whole floor diaphragm system. The shear walls have a constant thickness of 15 cm, from the basement to the roof, and are made of the relatively constant quality of the reinforced concrete, with the cylinder compressive strength measured in these walls being in the range of 33 to 48 MN/m$^2$. The percent of reinforcement in the shear walls ranges from 0.92 to 2.54% in the E-W direction and from 2.74 to 4.73 % in the N-S direction. The columns also have a constant cross section of $38 \times 38$ cm, with the prefabricated continuation at each third floor. All
columns are reinforced with $4\phi 18$, and the cylinder compressive strength measured in the ground floor columns was in the range from 62 to 64 MN/m$^2$. The measured Young’s modulus of elasticity was in the range $3.3 \times 10^4$ to $4.4 \times 10^4$ MN/m$^2$. The “non-structural” elements include light partition walls, a brick masonry lift shaft, and prefabricated reinforced concrete parapets, face elements, and staircase. The roof diaphragm is a cast-in-place reinforced-concrete structure.

The building is situated on soil with favorable (in the engineering sense) conditions, with considerable gravel deposit. The geotechnical soil profile beneath the building is shown in Fig. 4. The equivalent shear wave velocity in the top 30 m is near 475 m/s, which is about 60% higher than the typical value of 300 m/s in Southern California. Further, the footing is only about 4 m above the Marley clay layer, which has shear wave velocity of 650 m/s, and, consequently, the representative shear modulus of the soil around the foundation is 3 to 5 times that of the sites of the two reinforced concrete building in Southern California damaged by an earthquake that we studied [9,10]. Therefore the nonlinearities in the soil response at the site of the Borik-2 building occur at relatively higher levels of shaking.

**Results of Previous Forced and Ambient Vibration Tests**

Two forced-vibration tests were carried out by staff of the Institute of Earthquake Engineering and Engineering Seismology (IZIIS), Skopje, using GSV-101 GEOTRONIX vibrators [34]. The first test was conducted in July 1972, prior to the installation of most of the partition walls and the other nonstructural elements. The second test was conducted in October 1972, after almost all nonstructural elements had been installed. The mass of the building during these tests was estimated to be 59% and 81% of the final mass. Following the second test, when the state of the building was considered to be similar to the expected state during its service, its dynamic characteristics were defined. The measured natural frequencies are listed in Table 1.
The results of the forced-vibration tests indicate pronounced nonlinear response and some coupling between the translational and torsional responses. Table 2 shows our summary of the frequencies from the second test as function of the amplitudes of the harmonic force applied based on information in [35], which describes the second test. It is noted that the transition of the resonant curves from low to high forcing functions is not continuous for this building, as one would expect for a gradually softening system [36]. For both EW and NS excitation, the measured resonant curves show a clear peak at 1.38 Hz for the smallest periodic excitation (“force” of 115 kg). With increasing force, this peak disappears and a new one emerges at 1.34 Hz for EW response (“force” of 330 kg), and at 1.35 Hz for NS response (“force” of 200 kg). This behavior suggests opening and closing of some “gaps” at different force (deformation) amplitudes, and a complex dynamic system. It is of interest to determine whether such behavior occurs also during transient earthquake excitation, but currently available data is not sufficiently detailed for further studies of this effect.

In June 1983, almost two years after  $M=5.4$ Banja Luka earthquake of August 13, 1981, an ambient vibration test was conducted in the building, also by IZIIS staff members, to determine the dynamic characteristics of the structure after the earthquake [37]. The frequencies determined from this test are listed in Table 1 as reported in [37], and [38], which differ. The values quoted in subsequent work [39,40] are apparently based on the values reported in [38], and small differences appear to be due to conversions from period to frequency and different significant figures used.

Summary of Other Previous Studies

In this summary, for consistency and continuity we will use the same notation as used by the authors of the original studies, which use same notation ($f_1$ and $f_2$) both for the experimentally measured values (from Fourier spectra of the recorded response), and for the values computed
from analytical models of the structure assuming fixed-base response. We note here that, as discussed in the introduction of this paper, the experimental estimates represent the frequencies of the soil-structure system, while the analytical estimates represent the fixed-base frequencies. Later in this paper, we use a different notation for the different type of frequencies.

Jurukovski et al. [38] present results of parametric system identification for the NS response of the building, based on a one-dimensional lumped-mass model for the structure with horizontal degree of freedom for each mass, supported by a flexible foundation with horizontal and rocking degrees of freedom. They fitted this model to the forced and ambient vibration test data, and to the response to the August 13, 1981, earthquake based on the assumption that the response of the building was linear during both the tests and the earthquake (as no damage was reported following the earthquake). Further, they considered the effects of the soil structure interaction only in fitting the earthquake response data, obtaining first the building parameters by fitting the tests data to a fixed base model, and then used the obtained fixed-base model of the superstructure as input in fitting the earthquake data to include the foundation response. The latter was performed by iteration, with the objective to minimize the square error between the recorded and predicted motions at the 7th and 13th floors accumulated over a period of excitation. While their fit produced a reasonably good agreement of the first and second system frequencies, they provide no justification, reasoning or reference in support of their assumption that the effects of soil-structure interaction were significant only during the earthquake and not during the forced and ambient vibration tests. Finally, they conclude “a certain time after the earthquake, the dynamic system is strengthened and brought to its original position again,” but provide no evidence or references to support their claim, and they do not make it clear whether this “strengthening” occurs in the structure, in the soil, or in both.
Jurukovski et al. [38] is the first paper we found that quotes frequencies for the ambient vibration test conducted after the earthquake that are different from those reported earlier by Taškov and Krstevska [37]. These values are shown in brackets in column 4 of Table 1. Jurukovski et al. [38] also report values of torsional frequencies for the same test, which were neither measured (by the design of the experiment) nor mentioned in Taškov and Krstevska [37]. The values they quoted were later used by other investigators.

In Section 1.3.3 of “Selected Chapters on Earthquake Engineering”, Petrović [41] develops an equivalent continuous Bernoulli beam theory for analysis of natural periods of a symmetric building fixed at its base, illustrated on an example with geometric and material properties that correspond to the Borik-2 building. The author shows how, using the presented tables, one can compute \( f_1 = 1.27 \) Hz for this building, while the “exact” value, computed by numerical analysis using a digital computer and matrix representation of the stiffness gives \( f_1 = 1.20 \) Hz, the difference being only 5.5%.

Fajfar et al. [39] present a dynamic response analysis of the Borik-2 building using a fixed-base, discrete, linear model, with properties calibrated in terms of the results of: the 2\(^{nd}\) forced-vibration experiment conducted in 1972 before the earthquake, the earthquake response in 1981, and the ambient vibration test in 1983. They justify their assumption that the effects of soil-structure interaction for this building can be neglected, based on the appearance of the mode shapes as determined from the forced and ambient vibration tests. Such an argument, however, would be valid only if the foundation rocking were equal to zero, which was never observed to occur for real buildings. Their results imply “practically linear structural behavior” during the 1981 earthquake, with only minor excursions to the nonlinear range in the deformations of the shear walls (in the first story and from the 7\(^{th}\) to the 10\(^{th}\) floors). Table 3 summarizes the observational data (system frequencies in Hz) they use to calibrate their models, and Table 4
shows the corresponding frequencies of their models. The only reference to the source of the quoted frequencies for the ambient vibration test was that the test was performed by IZIIS, Skopje. Fajfar at al. [39] reach the following conclusions. (1) The differences among different experiments and earthquake response resulted almost entirely from changes in masses and from the influence of the non-structural elements in the response. The stiffness was significantly higher during small amplitudes of response and was much smaller during the larger earthquake response. (2) Large earthquake excitation amplitudes near 5 Hz contributed to strong excitation of the second mode, and this had significant influence on all internal forces. The displacement time history, however, was not significantly influenced by the second mode. (3) Without experimental data, only rough predictions of structural behavior during earthquakes are possible.

Borik-2 appears to be the “building of IMS type in Banja Luka” described in the book “Earthquake Engineering” by Anićić et al. [40] on pages 247–252, in what appears to be a summary of work by Fajfar et al. [39]. The authors describe their model, which includes walls as cantilevers, with T cross-sections, and a frame consisting of columns and inter-story floor slabs. They assume that the soil-structure interaction can be neglected because “the experimental results have shown that it plays a minimal role” but present no reference to experimental observations that would confirm this. Further, in their model, they assume that the nonstructural members accounted for 30% (for EW) to 40% (for NS) of the total stiffness of the building for weak motions (i.e. during the ambient and forced vibration tests), and only 8% to 18% (for both NS and EW motions) during the earthquake of August 13, 1981. Finally, they conclude that the behavior of the building was essentially linear during the earthquake, with the exception of “a few places” in the structure where slight nonlinearity may have occurred, but do not specify the location. Anićić et al. [40] summarize their results in two tables, which are reproduced here as Tables 3 and Table 4. Anićić et al. [40] conclude that the modeling tools they describe are capable of determining the dynamic characteristics of buildings shaken by small and moderate
earthquakes. They state that during small amplitude excitation (forced- and ambient vibration tests) the nonstructural members and partition walls contribute significantly to the stiffness of the models, and that the good agreement between the observed and predicted response shows the adequacy of their simple models. They note, however, that, in the absence of recorded full-scale response, this may be possible only approximately. In connection with their last comment, the reader may wish to peruse Ref. [42].

We conclude this section with Table 5, which summarizes the quoted values for the building (system) frequencies for the same ambient vibration test of June 1983 by different investigators, for convenience in their comparison. It appears that Fajfar et al. [39] and Aničić et al. [40] adopted the values reported by Jurukovski et al. [38], except for the 2nd system frequencies. All of the 1st system frequencies are essentially the same, and the minor differences appear to result from rounding off and conversion from period to frequency. What is remarkable in this comparison is (1) how different the values used in Ref. [38-40] are from those reported by Taškov and Krstevska [37], and (2) that the former quote torsional frequencies, while the latter do not. So far, from the published literature we have been able to gather on this building, we have not been able to determine the source for the quoted torsional frequencies in [38-40].

**EARTHQUAKE RECORDS AND OBSERVED DAMAGE**

The Borik-2 building was instrumented in October 1972, and was part of the Yugoslav strong motion network, which was operated by IZIIS [5]. The instrumentation consisted of three SMA-1 tri-axial accelerographs, stand alone, but with common triggering mechanism, installed at the foundation level, 7th, and 13th floors. The location of the instruments is shown in Fig. 3. Table 6 shows a list of 31 earthquakes recorded in the building during the operational period of the seismic array (October 1972 to end of 1980s). The first column shows the chronological order number of the 31 events, column 2 shows the code name by which the events digitized for this
study are referred to in this paper (EQ 01 to EQ 20). The records of the remaining 11 events were not digitized as they were considered too small to provide reliable information for this work. Columns 3 through 8 show respectively the available information about the event date, time, epicentral coordinates, focal depth, earthquake magnitude and epicentral intensity. Not all of the listed events have been identified. The film records of the selected 20 events used in this study were digitized and processed by the authors during the summer of 2006 using a flatbed scanner and the procedure described in [43].

It can be seen from Table 6 that the earthquake records we used in this study cover the period of 12 years, from 1974 to 1986. The largest earthquake is the Banja Luka, Yugoslavia, earthquake of August 13, 1981, which had magnitude $M = 5.4$ and epicenter in the Banja Luka seismic source area (Fig. 2b). This is event EQ 11 in Table 6, and is the same earthquake that was mentioned in the summary of the previous studies of this building.

Several days after the earthquake, detailed inspection of the building was carried out, and neither structural nor nonstructural damage (except for minor damage on the terrace, at the top of the building [44]) was observed, which shows that the building has worked essentially in the “elastic range” during all recorded strong earthquakes thus far. Fajfar et al.[39] suggested that local cracking might have occurred in some shear walls (at the 1st story and from 7th to the 10th stories) and in some columns (10th to 14th stories). Inspection of the soil conditions in the vicinity of the building showed no traces of nonlinear soil behavior close to the building.

Figure 5 shows plots of the relative horizontal displacements of the 13th floor relative to the basement versus time for the 20 earthquakes considered in this analysis. Part a) and b) correspond to the NS and EW components of motion. These relative displacements were computed by integrating twice the recorded accelerations after correction for a wavy baseline (by high pass filtering) and for the instrument transfer function, and filtering out the digitization
noise [43]. Hence, they represent band-pass filtered relative displacements. For convenience in comparing the motions from different earthquakes, the amplitude scale in Fig. 5 is the same for all events (-0.5 to 0.5 cm) except for EQ 11 (-5 to 5 cm). It can be seen that, during event EQ 11, the peak relative displacement in the NS direction reached about 4.2 cm, while they were less than 0.1 cm for most of the other events (except EQ 05). If we neglect the contribution from rigid body rocking to the relative displacements plotted in Fig. 5, than the earthquake records imply that the average drift between the basement and the 13th floor did not exceed 0.005% during the 19 smaller events, and did not exceed 0.1% for the NS response to the EQ 11. Because for typical reinforced concrete buildings structural damage begins to occur for drift amplitudes comparable to and exceeding about 1% [45], it is not surprising that event EQ 11 did not result in any apparent structural damage. Plots of Fourier spectra for the absolute acceleration of the 13th floor, and of the relative displacement of the 13th floor with respect to the basement can be found in [46].

**METHODOLOGY**

As the focus of this paper is on the case study rather than on introducing a new method, and due to the space limitations, the methodology is described only briefly. The reader is referred to other cited studies for further details.

**Fourier and Time-frequency Analysis**

For all events, the soil-structure system frequencies, \( f_{sys} \), were estimated from the peaks of the Fourier transform amplitudes of the relative displacement of the 13th floor with respect to the basement computed from the total length of the record. These frequencies were then checked against the spectra of the absolute acceleration response at the 13th floor.
For the largest event, EQ 11, time-frequency analysis was carried out to examine possible variations of the fundamental system frequency with time using both Gabor transform and zero crossing analysis. The Gabor transform analysis is described in detail in [47,48], along with a discussion about the preferences for the choice of transform, and how the Gabor transform is related to wavelet transform and to moving window analysis, and is described here very briefly. Essentially, the Gabor transform represents a smeared projection of the signal onto the time frequency plane, using a wavelet with a well localized frequency and a Gaussian amplitude modulation. The ridge of the transform gives the frequency versus time, while the skeleton of the transform gives the amplitude of the “modal” response. Once these two are determined, the variations of the frequency can be traced, with time and as function of the amplitude of the response. This method essentially gives the frequency in moving time windows assuming that the system is linear within the window. In the zero crossing analysis, the frequency and amplitude are estimated from the zero crossings of the band-pass filtered relative displacement, assuming that the time between two crossings is half of the system period [49]. The windowed Fourier methods generally give smoother results than the zero crossing analysis [50].

**Impulse Response Analysis**

The building fundamental fixed-base frequency was estimated from the travel time of seismic waves propagating from ground level to the roof (the boundary where total reflection occurs), and assuming that the building as a whole deforms primarily in shear. Then, if the travel time from ground to roof (or from the roof to the ground) is $\tau_{tot}$, the fundamental fixed base frequency is $f_j = 1/(4\tau_{tot})$. We measure $\tau_{tot}$ by tracing a virtual impulse that is either input at ground level or at roof level. The input and propagating virtual impulses are created simply by taking inverse Fourier transform of different system transfer functions. This is based on the fact that the system transfer function is a Fourier transform of the system impulse response function.
The relationship between the physical problem (propagating pulse) and linear system theory is as follows. Let the motion at ground level, \( u_{\text{ref}}(t) \), be the input and the output be the motion at the roof, or at any floor, \( u(t) \). Then the transfer function of \( u_{\text{ref}}(t) \) with respect to itself is unity \( (\hat{u}_{\text{ref}}(\omega)/\hat{u}_{\text{ref}}(\omega) = 1) \) and its inverse Fourier transform is the Dirac delta function, \( \delta(t) \), which physically represents an impulse. For some upper floor, the transfer function is \( \hat{h}(\omega) = \hat{u}(\omega)/\hat{u}_{\text{ref}}(\omega) \), and its inverse Fourier transform is the system function \( h(t) \), which represents physically the response at that floor to the input delta function at ground level. Hence, conceptually, the impulse response function can be computed as

\[
h(t) = FT^{-1}\left\{\frac{\hat{u}(\omega)}{\hat{u}_{\text{ref}}(\omega)}\right\}
\]

Practically, for earthquake records, a more stable solution is obtained using

\[
h(t) = FT^{-1}\left\{\frac{\hat{u}(\omega)\bar{u}_{\text{ref}}(\omega)}{[\hat{u}_{\text{ref}}(\omega)]^2 + \varepsilon}\right\}
\]

where the bar indicates complex conjugate, and \( \varepsilon \) is a regularization parameter [8]. In this work, we used \( \varepsilon = 0.1*\bar{P} \) when \( u_{\text{ref}} \) is the ground floor record, and \( \varepsilon = 0.05*\bar{P} \) when \( u_{\text{ref}} \) is the roof record, where \( \bar{P} \) is the average power of \( u_{\text{ref}} \). For more detail on this method, the reader is referred to previous work on its application to buildings [8-11].

Once \( h(t) \) has been computed for different sensors, relative to a reference motion \( u_{\text{ref}} \), the wave travel time between two points is determined by measuring the time of the arrival of the pulse at different locations, and the time delay \( \tau \) at one point relative to another. This is best done if \( u_{\text{ref}} \) is a sensor at the roof or a sensor at the ground floor. The relation between \( \tau \) and damage is based on the fact that \( \tau = d/V_s \), where \( d \) is the distance traveled and \( V_s \) is the
equivalent shear wave velocity in the part of the building between the two sensors. The latter is related to the rigidity via the relation $V_s = \sqrt{\mu/\rho}$, where $\mu$ is the shear modulus and $\rho$ is the density. Hence, reduction of rigidity due to damage will produce a reduction of the equivalent shear wave velocity, which will produce an increase in the pulse travel time, relative to the travel time for the undamaged state. In our earlier work, we used as “undamaged” state the initial time window of the earthquake shaking while the response was still small [9,10].

In this paper, we show results only for the global change in the building by monitoring $f_1 = 1/(4\tau_{tot})$. The fundamental fixed-base frequency $f_1$ is related to the first system frequency $f_{sys}$ by [12]

$$f_{sys}^2 = f_1^{-2} + f_H^2 + f_R^2$$

(3)

where $f_H$ and $f_R$ represent the horizontal and rocking frequencies of the system, which depend on the foundation horizontal and rocking stiffnesses. Eqn (3) implies $f_1 > f_{sys}$. In our previous work [9,10], we showed that this was the case for two reinforced concrete buildings in California, which supports our hypothesis that the $f_1 = 1/(4\tau_{tot})$ is approximately the fixed-base frequency. Further, a study of this method conducted recently by the second author on an analytical soil-structure interaction model, in which the building is represented as an equivalent shear beam, showed that the $1/(4\tau_{tot})$ gives $f_1$, while Fourier analysis gives $f_{sys}$, and that the measurement of $\tau_{tot}$ is not affected by the soil-structure interaction even for coupled horizontal and rocking foundation motions (these results will be reported in a separate paper).

RESULTS AND ANALYSIS

Fourier Analysis – Comparison of $f_{sys}$ during the Earthquakes and Forced and Ambient
Vibration Tests

Figure 6 summarizes the information that could be deciphered from the peaks in the Fourier amplitude spectra of the EW and NS records of the 20 earthquakes, which occurred between 1974 and 1986 (plots of the spectra can be found in [46]). The dots represent frequencies of all unambiguous spectral peaks plotted versus the event number (1 through 20). The full dots correspond to peaks for NS response, and the open dots to peaks for EW and torsional response. The dashes on the left show the frequencies determined from the 2\textsuperscript{nd} forced-vibration test, conducted in October 1972 [34], and the dashes on the right correspond to the frequencies measured from the ambient vibration test, conducted June 1983, and as reported in [37]. The trends we observe in these data are shown by a continuous line for the NS response, and a dashed line for the EW response, drawn by hand through the data points. These lines are interrupted at the time of EQ 11, which caused the largest amplitude response. A comparison of the pre and post EQ 11 trends shows a minor drop in $f_{sys}$ for the fundamental mode (for both NS and EW motions), from near 1.3 Hz before EQ 11 to about 1.15 Hz after EQ 11. The drop of $f_{sys}$ for the 2\textsuperscript{nd} mode is more apparent, for NS response from about 4.6 Hz to 4.0 Hz, and for EW response from about 5.0 Hz to 4.7 Hz. For both modes, $f_{sys}$ begins essentially at the frequencies measured during the forced-vibration tests in October 1972, and then gradually decrease and level off during events 5 through 10. After what appears to be a permanent drop, during event 11, these frequencies are again nearly constant during the remaining events, 13 through 20. The gradual decrease during events 1 through 5, and the largest drop during event 11, are apparently associated with cracking of structural concrete, of non-structural elements, and of partition walls.

The ambient vibration test in 1983 shows frequencies lower than those preceding event EQ 11 but consistent with those measured during events 19 and 20. Only for the second EW mode $f_{sys}$ is slightly smaller than the values we obtained for events 19 and 20. The difference, which is
about 5-6\% is of the order of magnitude that can be associated with changes due to environmental factors (e.g. rainfall and temperature) or changes in mass [51,52].

Figure 6 also shows recurring spectral peaks near 2 Hz and 4 Hz, which are not changed by event 11. At this time we do not have a plausible interpretation of what these peaks represent.

**Impulse Responses analysis and Variations of $f_1$ and Comparison with $f_{sys}$**

We computed impulse response functions for input impulse at the basement and for an input impulse at the 13\textsuperscript{th} floor. Plots of these impulse response functions and tabulated arrival and travel times between floors for all of the 20 events can be found in [46], and will not be repeated here. In this paper, we comment on some peculiarities of the analysis for this building, show the results for the fundamental fixed-base frequency $f_1 = 1/(4\tau_{tot})$, and focus on the analysis of its variations from one earthquake to another and in relation to $f_{sys}$.

A disadvantage for the impulse response analysis of this 14-story building is that the top instrument was not on the roof but on the 13\textsuperscript{th} floor (see Fig. 3c). For input impulse at the basement propagating up, reflections from the roof interfere with the upward propagating wave, and make reading the arrival time at the 13\textsuperscript{th} floor difficult and ambiguous. To avoid this problem, we used only the results for a virtual source at the 13\textsuperscript{th} floor, and measured the wave travel time by tracing the acausal wave propagating from the 13\textsuperscript{th} floor to the basement (see [8-10] for a discussion on the acausal wave). We also adjusted the measured wave travel time between the basement and the 13\textsuperscript{th} floor to get $\tau_{tot}$, for estimation of $f_1 = 1/(4\tau_{tot})$, as follows. Because the instrument on the 13\textsuperscript{th} floor is 36.07 m above the instrument in the basement, and the roof is 38.87 m above the instrument in the basement, we prorated the travel times by a factor $38.87/36.07 = 1.08$. We note that we are not considering the additional 3.4 m to the top of the building, which includes the terrace and the elevator equipment on the roof. While certainly
adding to the complexities of the impulse response functions at the 13th floor, these elements are not part of the structural system and have smaller plan dimensions. Consequently, we expect that the upward propagating waves effectively reflect off the roof surface.

Table 7 lists our estimates of $f_1$ for the 20 earthquakes along with the estimates of $f_{sys}$ for the first mode obtained by Fourier analysis. The corresponding values of peak relative displacements $d_{max}$ of the 13th floor relative to the basement are also listed. Figure 7 shows plots of $f_1$ and $f_{sys}$ for earthquakes EQ 01 through EQ 20, which occurred between 1974 and 1986, and also $f_{sys}$ during the forced vibration test in 1972 and the ambient vibration test in 1983. The lines connecting the point estimates for each event are drawn to help emphasize the trends.

It can be seen from Fig. 7 that for all earthquakes $f_{sys} < f_1$, which is consistent with eqn (3), and their ratio is approximately constant, even during EQ 11 when a significant drop occurs, which corresponds to essentially linear soil-structure interaction. During EQ 11, both $f_1$ and $f_{sys}$ drop, by about 18% and 31%, respectively, for NS motions, and by about 16% and 22%, respectively, for EW motions. Clearly, the nonlinearities during the seismic response of this building between 1974 and 1986 were relatively small, and no damage occurred in the building. These trends differ from our previous observations for the 7-story hotel in Van Nuys, and the Imperial County Services Building in El Centro, both in California [9,10], where we found that the ratio between $f_1$ and $f_{sys}$ changed significantly from one earthquake to another during the earthquakes that produced larger response. Figs. 7 also shows that the estimates of $f_{sys}$ from the forced-vibration test in 1972, and from ambient vibration test in 1983 are consistent with our
results for the 19 small earthquakes, which occurred between the tests. Our estimates of $f_{sys}$ and
the ambient tests in 1983 both suggest a slight drop in the system frequency, in the range of 10%.

Figure 7 shows that $f_1$ and $f_{sys}$ both essentially and consistently follow all small fluctuations
of frequency from one event to the next. As they were estimated completely independently, this
consistency assures us that the simple and subjective procedure we used to trace the pulses and
“read” their arrival times is adequate for the purpose of this analysis, and, in qualitative terms,
accurate, consistent from one reading to the next, and hence reproducible. Small fluctuations that
show up consistently in $f_1$ and $f_{sys}$ might be caused by changes in occupancy, environmental
factors (e.g. temperature and rainfall [51,52]), or a combination of these factors.

**Time-frequency Analysis and Variations of $f_{sys}$ during EQ 11**

Figure 8 shows results of time-frequency analysis of event EQ 11 using Gabor transform for
the fundamental mode only. Parts a) and b) correspond to the NS and EW components of
motion. In each part, the plots on the right hand side show, from top to bottom: the basement
acceleration, relative displacement of the 13th floor band-pass filtered around the 1st mode,
relative displacement amplitude associated with the 1st mode (estimated from the skeleton of the
Gabor transform, and from the amplitude of the associated analytic signal), and instantaneous
$f_{sys}$ versus time. The square in the bottom plot, with dimensions $2\sigma_t \times 2\sigma_v$ shows the
resolution (uncertainty) of the estimate of $f_{sys}$ [23,48]. The plot on the left shows the
instantaneous $f_{sys}$ versus amplitude. The numbered points in each plot correspond to selected
points in time.

Because of the fast growth of the response within only 2–3 s following trigger time and the
finite resolution of the method, this analysis does not show the decrease in $f_{sys}$ with growing
amplitudes of response at the beginning, but does show a small recovery after the largest amplitudes of response (after $t \sim 20$ s), which have been observed for other buildings [9,10,49-53]. From shortly after trigger until $t \sim 20$ s, $f_{sys}$ is essentially constant. Minor local decrease of $f_{sys}$ can be noticed for the EW response in Fig. 8b during a time interval with low relative response (points 2-3-4), which is consistent with softer system behavior for vibrations within open gaps.

Figure 9 shows the results of the zero-crossing analysis. Only the peaks that are sufficiently close to a sinusoid were selected for the analysis, shown by open circles in the plot of the relative displacement (top plot), and identified by numbers. The bottom plot shows $f_{sys}$ on the horizontal axis and the amplitude of response on the vertical axis for the selected points in the response. The nature of the changes of $f_{sys}$ with amplitude of response we find in these plots is typical of many other such analyses (e.g. [49]), and we interpret it as follows. In the beginning, and again during the arrivals of large strong-motion pulses with the onset of sudden ground motion, the system briefly becomes “stiffer” as it engages all of its constituents (all or most of the model “gaps” become closed) in a pseudo linear fashion of elastic-nonlinear or stiffening equivalent spring (e.g., point 1 for NS response and points 1 and 3-4-5 for EW response). During the “quiescent” intervals of strong motion, as the relative response begins to decay, the system progressively opens some or all of its “gaps” and with decaying amplitudes becomes “softer.” This can be seen, for example, for the sequences of peaks 4-5-6 and 14-15-16-17 for NS response, and for peaks 5-6-7 and 15-16-17 for the EW response. The overall behavior of the frequency vs. amplitude of all the peaks in Fig. 9 is in excellent agreement with the corresponding smoother results using Gabor transform in Fig. 8, with the pre-EQ 11 conditions ($f_{sys}$ during the forced vibration test in 1972, and the average of $f_{sys}$ during the small
earthquakes that preceded EQ 11), and with the post-EQ 11 conditions (the average $f_{sys}$ during the small earthquakes that followed EQ 11, and the average $f_{sys}$ for the ambient vibration tests in 1983).

**Windowed Impulse Response Analysis and Variations of $f_1$ during EQ 11**

For the largest event, EQ 11, we computed impulse responses in four time windows: 0–9 s, 10–21 s, 21–30 s, and 30–40 s, and obtained corresponding estimates of fixed-base frequency as follows: for NS vibrations $f_1 = 1.65$, 1.59, 1.54, and 1.59 Hz, and for EW vibrations $f_1 = 1.85$, 1.71, 1.54, and 1.49 Hz. Before we proceed with the analysis of the trends, we show in Fig. 10 the computed impulse response functions for input impulse at the 13th floor, and the readings of the impulse arrival times. Parts a) and b) correspond to the NS and EW responses, and in each part, the impulse responses are shown at the basement, 7th and 13th floors. The pulse arrival times were determined by visual inspection, based on our past experience with other buildings [9,10], and various trials using filtered data and different values of the regularization parameter $\varepsilon$ in eqn (2). Therefore, the readings of the pulse arrival times are neither unique nor can their accuracy be easily quantified by some simple error analysis. A more reproducible and systematic method of “reading” the pulse arrival times would require fitting a model, which is planned for our future work on refinement of this method.

Figure 11 shows the interval estimates of $f_1$ versus time during EQ 11, plotted by dots at the central times of the four intervals. These dots are connected by lines to help visualize the trends. The value of $f_1$ computed from the entire length of the record of EQ 11 (0 to 40 s), designated by $\bar{f}_1$, is also shown. It can be viewed as a weighted average of $f_1(t)$. The average of $f_1$ for all other 19 smaller events, designated by $\tilde{f}_{19}$ is also shown. Comparison of $\bar{f}_1$ and $f_1(t)$ with $\tilde{f}_{19}$, used as reference, shows that, while on the average $f_1$ during EQ 11 dropped by only 18% and
16% for NS and EW responses respectively, the drops of the interval values of \( f_1 \) were considerably larger, i.e. 21% for the NS and 30% for the EW responses. Interestingly, during these changes, \( f_{sys} \) was almost constant, between 0.9 and 1.0 Hz for the NS response (Fig. 9a), and between 0.97 and 1.10 Hz for the EW response (Fig. 9b). This implies that that the primary cause for the changes of \( f_{sys} \) during EQ 11 was change in \( f_1 \), while the small permanent changes after this event in \( f_{sys} \) appear to result from changes in the soil-foundation system. Compared with our findings for the buildings in Southern California \([9,10,49,50,53]\), this constitutes a new and interesting observation.

Going back to the variations of \( f_{sys} \) and \( f_1 \) on a larger time scale, shown in Fig. 7, we see a permanent drop of \( f_{sys} \) following EQ 11 of about 11% for the NS and 8% for the EW motions. The trends of \( f_1 \) in the same figure show fluctuations but no systematic drop after EQ 11. The systematic change in \( f_{sys} \) is even more pronounced for the 2\(^{nd}\) mode, as shown in Fig. 6. This may be yet another difference in the overall dynamic behavior of this building relative to what we are used to seeing in Southern California, where for small and intermediate levels of shaking \( f_{sys} \) often gradually returns to its pre-earthquake values \([49,50]\). The difference may be caused by the nature of the site conditions, which beneath the Borik-2 building include considerable gravel deposits. It may be of interest to note here that, in our studies of the spectral amplitudes of strong motion in the former Yugoslavia relative to southern California, the sites of typical Yugoslav accelerograph stations appear to be “stiffer” \([54-56]\).

**What is the Threshold Change of Frequency Associated with Damage?**

In our previous work, we proposed that changes of \( f_1 \) could be used as a simple global indicator of damage \([9,10]\). To eliminate the reliance on baseline data measured prior to the
earthquake, we proposed to use as baseline the value of $f_1$ determined from the initial time window of weaker shaking, and set an alarm if $f_1$ drops by more than a certain percentage (about 20% for the Van Nuys building in California). Such an algorithm, implemented in a real-time health-monitoring system, would have indicated that damage had occurred in the Van Nuys building during the San Fernando and Northridge earthquakes as early as about 10 s after trigger. We also noted that such a rule applied to $f_{sys}$ would have resulted in false alarm for some other earthquakes. For the Borik-2 building, we found an average drop of $f_1$ during the 1981 earthquake (over the length of the record) of 18% for the NS and 16% for the EW responses, while the corresponding largest “instantaneous” drops were 21% and 30%, and the building did not experience structural damage.

The behavior of these two buildings is not as different as it may appear based on the above quoted changes. For a meaningful comparison, we need, for the Van Nuys building, to express the change in $f_1$ relative to a different baseline that is comparable to the one used for the Borik-2 building. To do that, we choose as baseline a value of $f_1$ corresponding to one of the small local earthquakes, or the large but distant 1992 Landers earthquake, which produced smaller amplitudes of response than those during the initial time window of the 1994 Northridge earthquake [10]. If we take $f_1 = 1.25$ Hz as baseline, which is a representative value for the EW response to the small shaking before the Northridge earthquake, then the drop is 32% for the initial time window, before the onset of structural damage (from 1.25 Hz to 0.85 Hz), and 47% for the maximum drop in windows considered (from 1.25 Hz to 0.66 Hz) associated with structural damage. This suggests that a drop in $f_1$ of over 20% and even 30% may not necessarily be associated with structural damage.
Can one Use $f_{sys}$ as a Proxy for $f_1$?

As mentioned earlier, the analysis of $f_{sys}$ and $f_1$ during the 20 earthquakes suggests that, for this building and for the range of amplitudes of response covered by these data, the soil-structure interaction was essentially linear, and the trends in the variations of $f_{sys}$ agree well with the trends in the variation of $f_1$. However, there is a significant systematic difference between $f_{sys}$, which is near 1.3 Hz, and $f_1$, which is close to 2 Hz as estimated from wave travel times, which implies a factor of about 2.5 error in estimating the building stiffness ($\sim f_1^2$) used for calibrating structural models of this building [39-41]. Such an error due to neglecting the effects of soil-structure interaction in the interpretation of the observed dynamic response is significant, in view of the fact that such engineering models are used to verify the adequacy of structural models and design.

SUMMARY AND CONCLUSIONS

This study presented an analysis of small amplitude earthquake records of 20 events recorded in the Borik-2 building in Banja Luka over a period of 12 years (1974-1986). Only for one of these events, EQ 11, the building response approached damaging levels, but no structural damage was reported following an inspection. This 14-story prefabricated IMS type reinforced concrete building is an example of an instrumented building of an important type of construction, and has been tested and studied by many investigators. The previous studies were summarized and the consistency of the results of this study with the previous studies was examined. The earthquake records were analyzed by several types of analysis: Fourier – to estimate the soil-foundation-structure system frequencies, $f_{sys}$, impulse response – to estimate the building fundamental fixed base frequency, $f_1$, time-frequency analysis (essentially windowed Fourier
analysis) – to estimate the variations of \( f_{sys} \) with time during the largest earthquake, and windowed impulse response analysis – to estimate the variations of \( f_1 \) with time during the largest earthquake. The main objectives of this study were to: (1) augment the current knowledge base on the variability of \( f_{sys} \) and \( f_1 \) of real buildings not associated with damage, and the threshold change associated with damage, and (2) find out if the impulse response analysis method for structural health monitoring we are exploring would work for this building.

The main findings of this study are as follows.

1. The trends of \( f_{sys} \) observed during the 19 small earthquakes agree very well with the estimates of \( f_{sys} \) based on forced-vibration tests conducted in 1972 (following construction and prior to the recorded earthquake excitations), and based on ambient vibration test conducted in 1983, about two years after the largest earthquake shaking by the 1981 earthquake (the 11th event analyzed), and before the last small earthquake analyzed, which occurred in 1986.

2. The fixed-base frequency \( f_1 \) observed during the smaller events did not change as measured during the smaller events that followed the 1981 earthquake (EQ 11), which produced the largest response, but \( f_{sys} \) reduced permanently. For the NS response, the reduction was about 15% (from about 1.306 Hz on the average before to about 1.108 Hz on the average after the earthquake), and for the EW response the corresponding reduction was about 10% (from about 1.283 Hz to about 1.162 Hz). This is consistent with the damage inspection following the 1981 earthquake, which reported no structural damage, but in contrast to earlier observations for two reinforced concrete buildings in southern California [9,10]. Besides the
difference in height and type of construction, the difference in soil conditions, which are stiffer for the Borik-2 building, may have contributed to the observed differences.

3. During the 1981 earthquake, $f_1$ dropped temporarily, on the average (over the duration of the record) by 18% for the NS and 16% for the EW responses, respectively, while the largest “instantaneous” (i.e. within the four time windows) drops were 21% and 30%, respectively. This suggests nonlinear but essentially “elastic” response of the structure itself during the 1981 earthquake. It also suggests that a drop of $f_1$ in a building of as much as 20 – 30% may not necessarily lead to damage. This suggests that the threshold change in $f_1$ needs to be carefully quantified, and that true nonlinear models of structural response such that can predict the observed effects are needed for reliable structural health monitoring methods that rely on analytical models. Such models can be best calibrated by full-scale earthquake response data recorded in structures.

4. For the range of amplitudes of response during the 20 earthquakes considered and the full-scale tests, the soil-structure interaction for this building was essentially linear, based on the agreement of the trends in the variations of $f_{sys}$ and $f_1$. However, the difference between $f_{sys}$ (near 1.3 Hz), and $f_1$ (close to 2 Hz) is significant, implying factor of about 2.5 error in estimating the building stiffness ($\sim f_1^2$) if structural models are calibrated relative to $f_{sys}$ instead of $f_1$. Such an error is significant when such models are used to verify the adequacy of structural models and design procedures.

5. Despite the simplifying assumptions, due to the fact that the top sensor was not on the roof or top floor, the impulse response analysis yielded physically meaningful wave travel times, and estimates of $f_1$ that are consistent with the independent estimates of $f_{sys}$ using Fourier
analysis. The analysis of the trends of $f_1$ provided useful insight into the seismic response of this building.

We conclude that invaluable information about the dynamic behavior of structures can be extracted from analyses of many small earthquake recordings (the displacement of this building during 18 of the earthquakes studied in this paper were smaller than those during the forced-vibration tests). Such information is much needed for the development and calibration of realistic models for prediction of the seismic response of structures for structural health monitoring and design, and for calibration of empirically based methods. Most structural health-monitoring algorithms are based on detecting changes relative to the conditio quo ante, and their accuracy depends on the accuracy of the knowledge of the prior conditions, which change with site and time (as seen from this analysis). Therefore, it is a responsibility of the agencies in charge of archiving strong-motion data recorded in structures to recognize this fact, and make available on a routine basis also small amplitude response data. Considering that data from full-scale observations in real buildings are much more valuable relative to even the most sophisticated laboratory experiments, that such data is already recorded, and technology exists for accurate digitization and processing even of the analog records, there should be no further delay in the systematic publication and release of such data on all buildings with multiple earthquake recordings. The only way the science of predicting the seismic response of structures can be advanced is through creation of a sound and comprehensive database on actual response of real structures. This will provide an unquestionable—and the only acceptable—basis for testing various theoretical models and will provide a realistic view of the nature and extent of changes in structural behavior over time. Without such a database, it is impossible to develop robust and reliable structural health-monitoring systems and to calibrate the required damage detection thresholds.
ACKNOWLEDGEMENTS

We are grateful to B. Petrović, one of the designers of the IMS system, for the useful discussions and comments about the Borik-2 building. We are grateful to P. Fajfar for sending us his article on dynamic analyses of the Borik-2 building. We are indebted to Lj. Taškov and L. Krstevska for their invaluable help in interpreting the 1983 report on ambient vibration measurements in the Borik-2 building and for many useful discussions and comments. Last but not the least, we thank the director and the staff of the Seismological Observatory in Skopje for their hospitality in August 2006, during the digitization of the records used in this study.

REFERENCES


47. Todorovska MI, Trifunac MD. *Earthquake damage detection in the Imperial County Services Building I: the data and time-frequency analysis*, *Soil Dynam. and Earthquake Engrg.* 2007;27(6):564–576.


TABLE CAPTIONS

Table 1 System frequencies of the Borik-2 building estimated using different experimental techniques.

Table 2. Forces, frequencies, and peak displacements at resonance during the forced-vibration test of October 1972 (based on results in Figs 3.9 and 3.11 in [35]).

Table 3 Experimental (system) frequencies for the Borik-2 building used by Fajfar et al. [39] to calibrate their analytical models.

Table 4 Model (fixed-base) frequencies evaluated by Fajfar et al. [39] that correspond to the conditions for the full-scale data in Table 3.

Table 5 A summary of values for the (system) frequencies of the Borik-2 building from the ambient vibration test of June 1983, as quoted by different investigators (Taškov and Krstevska 1983; Jurukovski et al. 1984; Fajfar et al. 1987; and Aničić et al. 1990), including readings by the authors of this paper directly from the original data.

Table 6. Earthquakes recorded in the Borik-2 building during its seismic observation period (1972 to ~1990).

Table 7 Building fundamental fixed-base frequency \( f_1 = 1/(4\tau_{\text{tot}}) \), system frequency \( f_{\text{sys}} \), and the corresponding peak relative displacements \( d_{\text{max}} \) (of the 13\textsuperscript{th} floor with respect to basement) for the Borik-2 building during the 20 earthquakes (1974-1986).
FIGURE CAPTIONS

Fig. 1  A photo of the Borik-2 building (view from South-West).

Fig. 2  Location of the Borik-2 building within: (a) the area of former Yugoslavia, and (b) a larger view of the area near Banja Luka, also showing the epicenters of 16 of the contributing earthquakes, which have been identified. The rectangles in part a) show areas with concentrated earthquake activity between the mid-1970s early 1980s [5-7].

Fig. 3 Borik-2 building: (a) foundation plan, (b) typical floor plan, and (c) North-South cross-section.

Fig. 4  A geotechnical soil profile for the site of the Borik-2 building.

Fig. 5 Relative displacements of the 13th floor with respect to the basement computed from the recorded accelerograms. (a) NS response. (b) EW response.

Fig. 6 Soil-structure system frequencies for the 1st and 2nd modes of vibration estimated from the earthquake records over a period of 12 years (1974-1986) for the NS (full dots) and EW response (open dots). Published values based on a forced vibration test conducted before the earthquakes (October 1972) [34], and ambient vibration tests conducted in June 1983 [37], after the largest earthquake (event 11), are also shown. The torsional frequencies reported in [38] are also shown for completeness of this presentation.

Fig. 7 System frequency $f_{sys}$ and fixed-base frequency $f_1$ during the 20 earthquakes plotted versus the earthquake event number. The values of $f_1$ and of $f_{sys}$ are interconnected by straight lines to help visualize the trends. The values determined from the forced vibration test in 1972, and from the ambient vibration test in 1983 are also shown. (a) NS response. (b) EW response.

Fig. 8 Results of time-frequency analysis based on Gabor transform. (a) NS response. (b) EW response.

Fig. 9 Results of zero-crossing analysis. (a) NS response. (b) EW response

Fig. 10 Impulse response functions computed for four time windows during event EQ 11: 0–9 s, 10–21 s, 22–30 s, and 31–40 s. (a) NS response. (b) EW response. The right vertical amplitude scale is for segment 0-9 s and the left scale is for all other segments.

Fig. 11 Comparison of the window estimates of $f_1$ during event EQ 11 (interpolated by straight lines), with the “instantaneous” $f_{sys}$ during event EQ 11, and with $\tilde{f}_1 = \text{the average value of } f_1$ for event EQ 11 (estimated from the total length of the record), and $\tilde{f}_{19} = \text{the average value of } f_1$ during the 19 small events. (a) NS response. (b) EW response.
### Table 1. System frequencies of the Borik-2 building estimated using different experimental techniques.

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¹ Values reported in Taškov and Krstevska [37]. It appears that in their report the orientation of the building is off by 90º. Consequently, in this table, we exchanged the reported values for EW and NS motions.

² The values inside the brackets are those reported in [38-40], supposedly based on the same experiment.

### Table 2. Forces, frequencies, and peak displacements at resonance during the forced-vibration test of October 1972 (based on results in Figs 3.9 and 3.11 in [35]).

<table>
<thead>
<tr>
<th>Direction</th>
<th>Approximate force at resonance [lb] ([kg])</th>
<th>Resonant frequency [Hz]</th>
<th>Peak roof displacement [cm]</th>
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<tbody>
<tr>
<td>EW</td>
<td>730 (330)</td>
<td>1.343</td>
<td>0.180</td>
</tr>
<tr>
<td></td>
<td>440 (200)</td>
<td>1.360</td>
<td>0.120</td>
</tr>
<tr>
<td></td>
<td>260 (115)</td>
<td>1.376</td>
<td>0.076</td>
</tr>
<tr>
<td>NS</td>
<td>440 (200)</td>
<td>1.345</td>
<td>0.112</td>
</tr>
<tr>
<td></td>
<td>350 (160)</td>
<td>1.372</td>
<td>0.085</td>
</tr>
<tr>
<td></td>
<td>260 (115)</td>
<td>1.380</td>
<td>0.068</td>
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</table>
Table 3  Experimental (system) frequencies for the Borik-2 building used by Fajfar et al. [39] to calibrate their analytical models.

<table>
<thead>
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<th></th>
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</thead>
<tbody>
<tr>
<td>1st mode</td>
<td>E-W</td>
<td>1.30-1.37</td>
<td>1.28</td>
<td>0.98-1.00</td>
</tr>
<tr>
<td></td>
<td>N-S</td>
<td>1.30-1.39</td>
<td>1.22</td>
<td>0.91</td>
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<tr>
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<td>Torsion</td>
<td>1.49-1.56</td>
<td>1.41</td>
<td>-</td>
</tr>
<tr>
<td>2nd mode</td>
<td>E-W</td>
<td>5.56</td>
<td>5.00</td>
<td>4.35-4.76</td>
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<tr>
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<td>N-S</td>
<td>4.76-5.00</td>
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<td>4.17-4.35</td>
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<td>Torsion</td>
<td>5.88-6.25</td>
<td>5.88</td>
<td>-</td>
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Table 4  Model (fixed-base) frequencies evaluated by Fajfar et al. [39] that correspond to the conditions for the full-scale data in Table 3.

<table>
<thead>
<tr>
<th>Mode</th>
<th>Direction</th>
<th>System frequency [Hz]</th>
</tr>
</thead>
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<tr>
<td>1st</td>
<td>E-W</td>
<td>1.33</td>
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<tr>
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<td>N-S</td>
<td>1.32</td>
</tr>
<tr>
<td></td>
<td>Torsion</td>
<td>1.59</td>
</tr>
<tr>
<td>2nd</td>
<td>E-W</td>
<td>6.25</td>
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<tr>
<td></td>
<td>N-S</td>
<td>5.26</td>
</tr>
<tr>
<td></td>
<td>Torsion</td>
<td>7.69</td>
</tr>
</tbody>
</table>
Table 5  A summary of values for the (system) frequencies of the Borik-2 building from the ambient vibration test of June 1983, as quoted by different investigators (Taškov and Krstevska 1983; Jurukovski et al. 1984; Fajfar et al. 1987; and Aničić et al. 1990), including readings by the authors of this paper directly from the original data.

<table>
<thead>
<tr>
<th>Mode</th>
<th>Direction</th>
<th>System frequency [Hz]</th>
</tr>
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<tbody>
<tr>
<td></td>
<td></td>
<td>Taškov and Krstevska [37]</td>
</tr>
<tr>
<td>1st</td>
<td>E-W</td>
<td>1.12 [1.12]¹</td>
</tr>
<tr>
<td></td>
<td>N-S</td>
<td>1.04 [1.11]¹</td>
</tr>
<tr>
<td></td>
<td>Torsion</td>
<td>-</td>
</tr>
<tr>
<td>2nd</td>
<td>E-W</td>
<td>4.24 [4.46]¹</td>
</tr>
<tr>
<td></td>
<td>N-S</td>
<td>3.84 [4.05]¹</td>
</tr>
<tr>
<td></td>
<td>Torsion</td>
<td>-</td>
</tr>
</tbody>
</table>

¹Average values based on 5 to 7 data points in the report by Taškov and Krstevska [37].

²Average values, based on re-examination by the third author in June 2007 of 5 to 7 recordings, at different floors, with clear local peaks in the Fourier amplitude spectra as plotted in the report by Taškov and Krstevska [37].
Table 6. Earthquakes recorded in the Borik-2 building during its seismic observation period (1972 to ~1990).

<table>
<thead>
<tr>
<th>Order No.</th>
<th>Reference name</th>
<th>Date</th>
<th>Time [GMT]</th>
<th>Epicentral coordinates</th>
<th>Focal depth [km]</th>
<th>( M )</th>
<th>( I_0 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>EQ 01</td>
<td>4/12/1974</td>
<td>06:24</td>
<td>44 27 00N, 17 09 00E</td>
<td>-</td>
<td>2.8</td>
<td>5 MCS</td>
</tr>
<tr>
<td>2</td>
<td>EQ 02</td>
<td>4/23/1974</td>
<td>03:45</td>
<td>44 42 00N, 17 18 00E</td>
<td>-</td>
<td>3.0</td>
<td>5 MCS</td>
</tr>
<tr>
<td>3</td>
<td>EQ 03</td>
<td>2/17/1975</td>
<td>14:24</td>
<td>44 49 29N, 17 00 21E</td>
<td>0</td>
<td>3.3</td>
<td>6 MCS</td>
</tr>
<tr>
<td>4</td>
<td>EQ 04</td>
<td>8/09/1975</td>
<td>08:46</td>
<td>44 56 44N, 17 22 35E</td>
<td>0</td>
<td>-</td>
<td>4 MCS</td>
</tr>
<tr>
<td>5</td>
<td>EQ 05</td>
<td>10/08/1975</td>
<td>12:15</td>
<td>44 48 00N, 17 18 00E</td>
<td>-</td>
<td>2.7</td>
<td>5 MCS</td>
</tr>
<tr>
<td>6</td>
<td>EQ 06</td>
<td>4/20/1977</td>
<td>00:31</td>
<td>44 51 56N, 17 19 40E</td>
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</tr>
<tr>
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<td></td>
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<td>4 MCS</td>
</tr>
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<td>EQ 07</td>
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<td>44 44 07N, 16 56 24E</td>
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<td>3.7</td>
<td>6 MCS</td>
</tr>
<tr>
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<td>EQ 08</td>
<td>9/07/1979</td>
<td>12:57</td>
<td>44 51 24N, 17 35 07E</td>
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<td>4.0</td>
<td>5 MCS</td>
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<tr>
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<td>EQ 09</td>
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<td>44 34 48N, 17 02 24E</td>
<td>0</td>
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<td>-</td>
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<td>8/13/1981</td>
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<td>44 43 12N, 17 13 12E</td>
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<td>-</td>
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<td>3.2</td>
<td>5 MM</td>
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<tr>
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<td>EQ 17</td>
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<td>-</td>
<td>-</td>
<td>-</td>
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<tr>
<td>19*</td>
<td>EQ 18</td>
<td>(8/13/1981–8/21/1981)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>20*</td>
<td>EQ 19</td>
<td>(8/13/1981–8/21/1981)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
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</tr>
<tr>
<td>21*</td>
<td>EQ 20</td>
<td>(8/13/1981–8/21/1981)</td>
<td>-</td>
<td>-</td>
<td>-</td>
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<td>-</td>
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<tr>
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<tr>
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<td>-</td>
</tr>
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<td>-</td>
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<td>3.1</td>
<td>3.7</td>
<td>5 MM</td>
</tr>
</tbody>
</table>

* For these events, only motions at the foundation level and on the 13th floor were recorded (due to malfunction of the instrument on the 7th floor).
Table 7 Building fundamental fixed-base frequency $f_1 = 1/(4\tau_{tot})$, system frequency $f_{sys}$, and the corresponding peak relative displacements $d_{max}$ (of the 13th floor with respect to basement) for the Borik-2 building during the 20 earthquakes (1974-1986)

<table>
<thead>
<tr>
<th>EQ Number</th>
<th>$f_1$ Hz</th>
<th>$f_{sys}$ Hz</th>
<th>$d_{max}$ cm</th>
<th>$f_1$ Hz</th>
<th>$f_{sys}$ Hz</th>
<th>$d_{max}$ cm</th>
</tr>
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<tbody>
<tr>
<td>01</td>
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<td>0.013</td>
<td>1.93</td>
<td>-</td>
<td>0.007</td>
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<tr>
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<td>1.93</td>
<td>1.35</td>
<td>0.014</td>
<td>2.10</td>
<td>1.32</td>
<td>0.012</td>
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<tr>
<td>03</td>
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<td>0.062</td>
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<td>0.039</td>
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<td>0.064</td>
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<td>1.30</td>
<td>0.036</td>
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<td>-</td>
<td>0.040</td>
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<td>0.027</td>
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<tr>
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<td>-</td>
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<td>0.99</td>
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<td>0.99</td>
<td>0.009</td>
<td>2.20</td>
<td>1.15</td>
<td>0.010</td>
</tr>
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<td>13</td>
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<td>2.20</td>
<td>1.19</td>
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<td>0.030</td>
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<td>1.09</td>
<td>0.012</td>
<td>2.20</td>
<td>1.10</td>
<td>0.011</td>
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<tr>
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<td>1.93</td>
<td>1.10</td>
<td>0.037</td>
<td>2.01</td>
<td>1.15</td>
<td>0.045</td>
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<td>0.026</td>
<td>2.10</td>
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<td>0.010</td>
<td>2.10</td>
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<td>0.017</td>
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<td>-</td>
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<td>0.042</td>
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<td>1.19</td>
<td>0.018</td>
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<td>1.11</td>
<td>0.106</td>
<td>2.07</td>
<td>1.18</td>
<td>0.089</td>
</tr>
</tbody>
</table>
Fig. 1. A photo of the Borik-2 building (view from South-West).
Fig. 2. Location of the Borik-2 building within: (a) the area of former Yugoslavia, and (b) a larger view of the area near Banja Luka, also showing the epicenters of 16 of the contributing earthquakes, which have been identified. The rectangles in part a) show areas with concentrated earthquake activity between the mid-1970s early 1980s [5-7].
Fig. 3 Borik-2 building: (a) foundation plan, (b) typical floor plan, and (c) North-South cross-section.

Fig. 4. A geotechnical soil profile for the site of the Borik-2 building.
Fig. 5 Relative displacements of the 13th floor with respect to the basement computed from the recorded accelerograms. (a) NS response. (b) EW response.
Fig. 5 (cont.)
Fig. 6. Soil-structure system frequencies for the 1st and 2nd modes of vibration estimated from the earthquake records over a period of 12 years (1974-1986) for the NS (full dots) and EW response (open dots). Published values based on a forced vibration test conducted before the earthquakes (October 1972) [34], and ambient vibration tests conducted in June 1983 [37], after the largest earthquake (event 11), are also shown. The torsional frequencies reported in [38] are also shown for completeness of this presentation.
Fig. 7. System frequency $f_{sys}$ and fixed-base frequency $f_1$ during the 20 earthquakes plotted versus the earthquake event number. The values of $f_1$ and of $f_{sys}$ are interconnected by straight lines to help visualize the trends. The values determined from the forced vibration test in 1972, and from the ambient vibration test in 1983 are also shown. (a) NS response. (b) EW response.
Fig. 8 Results of time-frequency analysis based on Gabor transform. (a) NS response. (b) EW response.
Fig. 9 Results of zero-crossing analysis. (a) NS response. (b) EW response.
Fig. 10 Impulse response functions computed for four time windows during event EQ 11: 0–9 s, 10–21 s, 22–30 s, and 31–40 s. (a) NS response. (b) EW response. The right vertical amplitude scale is for segment 0-9 s and the left scale is for all other segments.
Fig. 11 Comparison of the window estimates of $f_1$ during event EQ 11 (interpolated by straight lines), with the “instantaneous” $f_{sys}$ during event EQ 11, and with $\bar{f}_1$ = the average value of $f_1$ for event EQ 11 (estimated from the total length of the record), and $\tilde{f}_{19}$ = the average value of $f_1$ during the 19 small events. (a) NS response. (b) EW response.