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Variability of the fixed-base and soil–structure system frequencies of a building—The case of Borik-2 building

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SUMMARY

Borik-2 is an IMS (Institute for Testing of Materials, Belgrade, Serbia)-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 tests 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–structure interaction. During the largest event, f_1 and f_{sys} decreased, respectively, by about 16 and 22% for EW motions, and by about 18 and 31% for NS motions, compared with 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% for EW and 15% for NS. Copyright © 2008 John Wiley & Sons, Ltd.

KEY WORDS: 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

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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 for the 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. The uniformly processed data on accelerations recorded in this building, and the trends of the changes of its characteristic frequencies in time should provide a unique and an invaluable starting point for realistic testing and for verification of those structural health-monitoring methods, which are intended for use in real buildings.

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 1974 and October 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 and 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 (13 August 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 (1972–1990). 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 (named after its developer, the Institute for Testing of Materials, Belgrade, Serbia), which has been used extensively in the former Yugoslavia since

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

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 ($\sim$2.5 million m^2 of built area) have been constructed using this system. Consequently, it was extensively tested and continues to be developed further at the IMS 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

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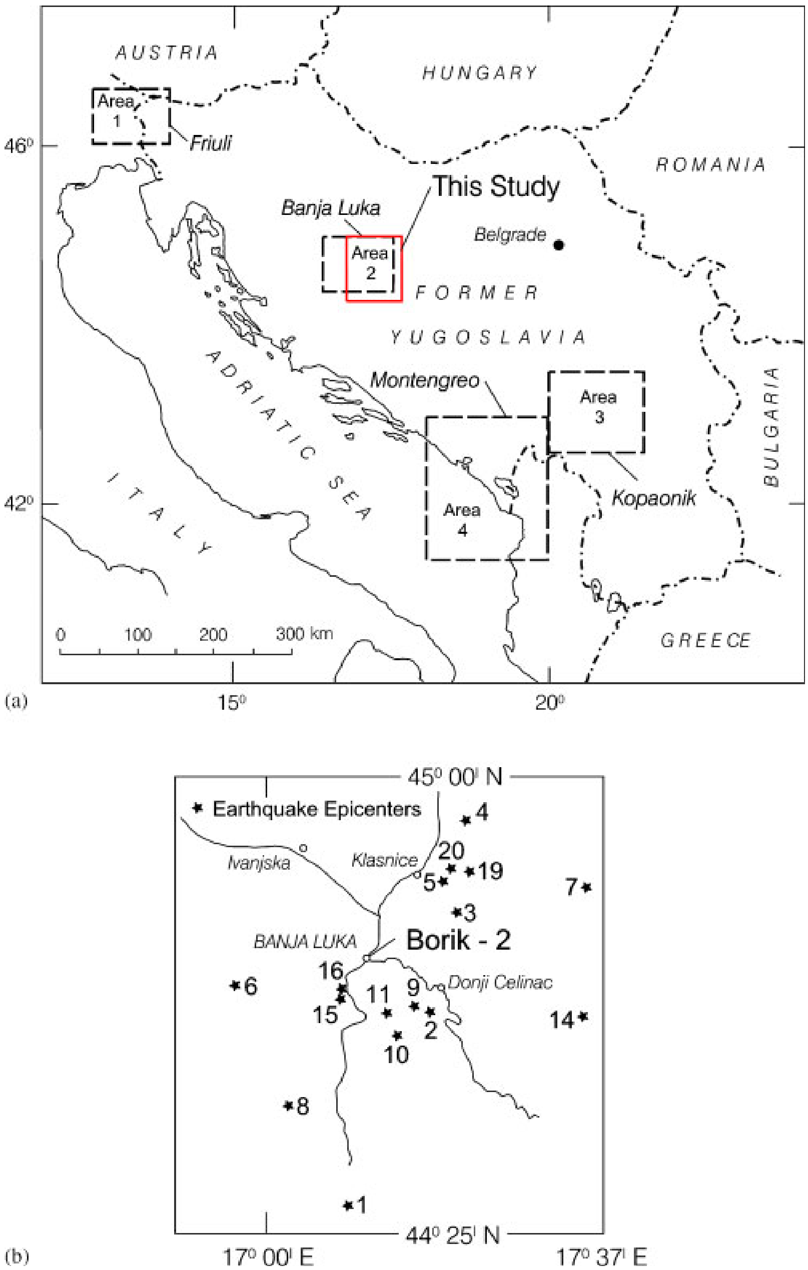

Figure 2. Location of the Borik-2 building within (a) the area of former Yugoslavia and (b) a larger view of the area near Banja Luka, also showing the epicenters of 16 of the contributing earthquakes, which have been identified. The rectangles in part (a) show areas with concentrated earthquake activity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

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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 with the dynamic stiffness of the foundation soil. The frequency associated with the fundamental mode of vibration of the building–soil system, 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 τ_{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 a 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. The objectives of this study are (i) to use the Borik-2 data to find out whether the observations for previously studied buildings hold for other buildings in general or are specific to the buildings studied; (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; (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 in stiffness of the structure, beyond what is expected from normal service over its life); (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

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response (same as impulse response functions); and (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 test 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 are described, followed by a brief summary 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 17.84×17.84 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 the ground level. The typical framing consists of columns spaced at 4.20 m centers in both the transverse and longitudinal directions.

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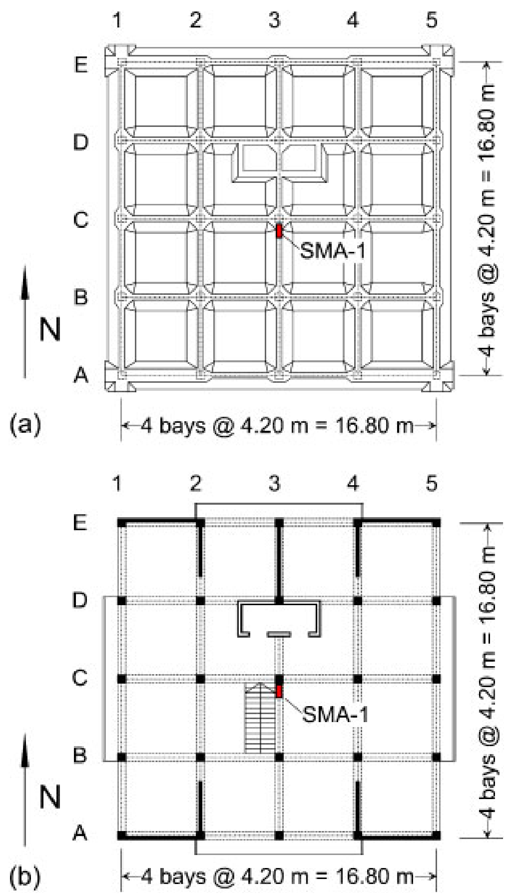

Figure 3. Borik-2 building: (a) foundation plan, (b) typical floor plan, and (c) North–South cross section.

The building was constructed in 1972 using the IMS 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 33–48 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 38×38 cm, with the prefabricated continuation at each third floor. All columns are reinforced with 4ø18 (4 reinforcing bars with 18 mm diameter), and the cylinder compressive strength measured in the ground floor columns

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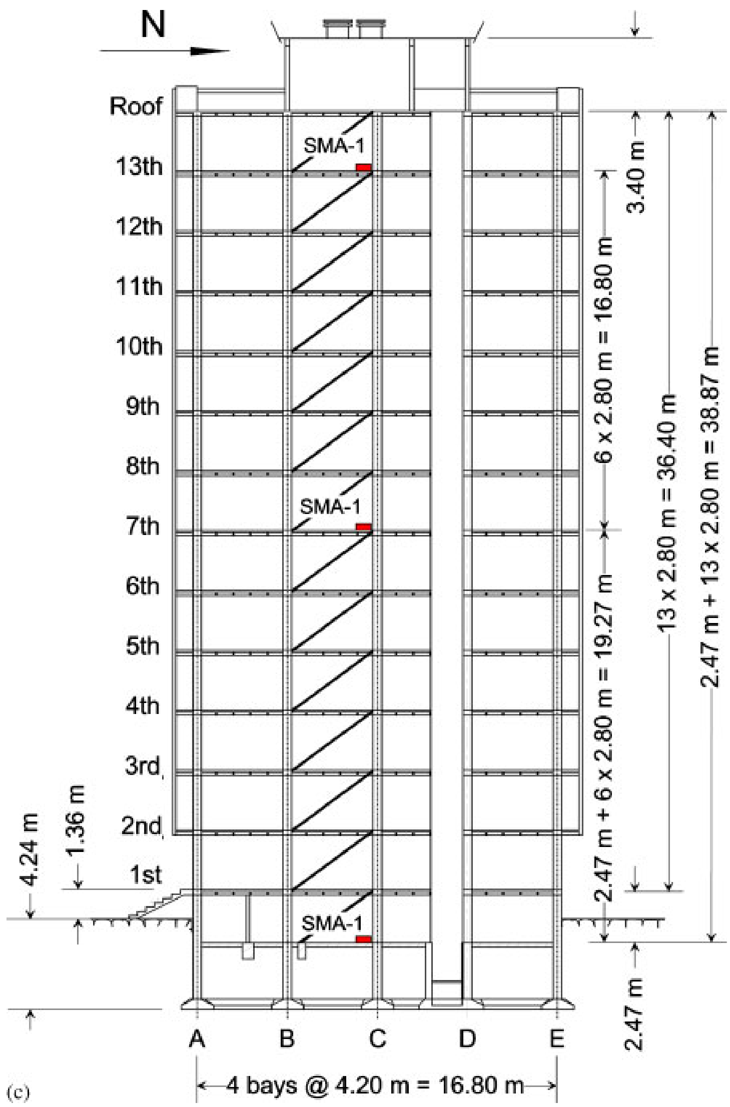

Figure 3. *Continued.*

was in the range from 62 to 64 MN/m^2. The measured Young's modulus of elasticity was in the range 3.3×10^4 to 4.4×10^4 MN/m^2. The 'nonstructural' 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 Figure 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

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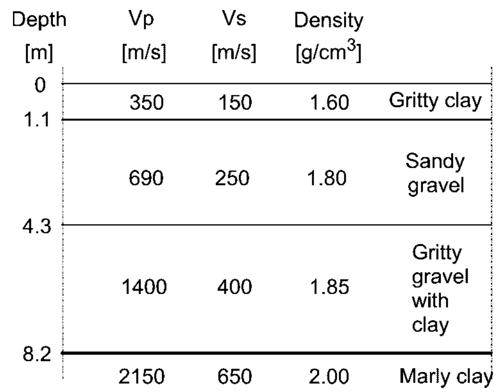

Figure 4. A geotechnical soil profile for the site of the Borik-2 building.

about 4 m above the Marley clay layer, which has a shear wave velocity of 650 m/s, and, consequently, the representative shear modulus of the soil around the foundation is 3–5 times that of the sites of the two reinforced-concrete buildings 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 I.

The results of the forced vibration tests indicate pronounced nonlinear response and some coupling between the translational and torsional responses. Table II shows our summary of the frequencies from the second test as a 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 the EW and the 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 the EW response (‘force’ of 330 kg), and at 1.35 Hz for the 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 are not sufficiently detailed for further studies of this effect.

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

Mode	Direction	System frequency [Hz]		
		Forced vibration test of October 1972 [34]	Ambient vibration test of June 1983 [37,38]	Earthquake response of 13 August 1981 [38]
First	EW	1.31	1.12* (1.28)†	1.00
	NS	1.30	1.04 (1.22)	0.87
	Torsion	1.50	— (1.40)	—
Second	EW	5.57	4.24 (5.12)	4.34
	NS	5.06	3.84 (4.48)	4.00
	Torsion	6.08	— (5.90)	—

*Values reported in Taškov and Krstevska [37]. It appears that in their report the orientation of the building is off by 90°. Consequently, in this table, we exchanged the reported values for EW and NS motions.
†The values inside the brackets are those reported in [38–40], supposedly based on the same experiment.

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

Direction	Approximate force at resonance (lb) (kg)	Resonant frequency (Hz)	Peak roof displacement (cm)
EW	730 (330)	1.343	0.180
	440 (200)	1.360	0.120
	260 (115)	1.376	0.076
NS	440 (200)	1.345	0.112
	350 (160)	1.372	0.085
	260 (115)	1.380	0.068

In June 1983, almost 2 years after $M = 5.4$ Banja Luka earthquake of 13 August 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 I as reported in [37,38], 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 the 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 a 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.

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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 a 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 13 August 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 that '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, which are different from those reported earlier by Taškov and Krstevska [37]. These values are shown in brackets in column 4 of Table I). 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 Jurukovski *et al.* [38] 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 second 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 the 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 7th to the 10th floors). Table III summarizes the observational data (system frequencies in Hz) they use to calibrate their models, and Table IV 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 *et al.* [39] reach the following conclusions: (1) The differences among

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

		System frequency (Hz)		
Mode	Direction	Forced vibration test of October 1972	Ambient vibration test of June 1983 (IZIIS)	Earthquake response of 13 August 1981
First mode	E–W	1.30–1.37	1.28	0.98–1.00
	N–S	1.30–1.39	1.22	0.91
	Torsion	1.49–1.56	1.41	—
Second mode	E–W	5.56	5.00	4.35–4.76
	N–S	4.76–5.00	4.55	4.17–4.35
	Torsion	5.88–6.25	5.88	—

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

		System frequency (Hz)		
Mode	Direction	Forced vibration test of October 1972	Ambient vibration test of June 1983	Earthquake response of 13 August 1981
First	E–W	1.33	1.20	0.97–1.06
	N–S	1.32	1.19	0.89–0.93
	Torsion	1.59	1.43	—
Second	E–W	6.25	5.55	5.26–5.55
	N–S	5.26	4.76	3.85–4.17
	Torsion	7.69	7.14	—

different experiments and earthquake response resulted almost entirely from changes in masses and from the influence of the nonstructural 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 Aničić *et al.* [40, pp. 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 8–18% (for both NS and EW motions) during the earthquake of 13 August 1981. Finally, they conclude that the behavior

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Table V. 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 [37–40], including readings by the authors of this paper directly from the original data.

		System frequency [Hz]				
Mode	Direction	Taškov and Krstevska [37]	Jurukovski *et al.* [38]	Fajfar *et al.* [39]	Aničić *et al.* [40]	This report*
First	E–W	1.12 [1.12]†	1.28	1.28	1.28	1.10
	N–S	1.04 [1.11]†	1.22	1.22	1.22	1.09
	Torsion	—	1.40	1.41	1.41	—
Second	E–W	4.24 [4.46]†	5.12	5.00	5.00	4.33
	N–S	3.84 [4.05]†	4.48	4.55	4.55	3.95
	Torsion	—	5.90	5.88	5.88	—

*Average values based on 5–7 data points in the report by Taškov and Krstevska [37].
†Average values, based on re-examination by the third author in June 2007 of 5–7 recordings, at different floors, with clear local peaks in the Fourier amplitude spectra as plotted in the report by Taškov and Krstevska [37].

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. Aničić *et al.* [40] summarize their results in two tables, which are reproduced here as Tables III and IV. Aničić *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 a recorded full-scale response, this may be possible only approximately. In connection with their last comment, the reader may wish to peruse Reference [42], which emphasizes the same conclusion.

We conclude this section with Table V, 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 second system frequencies. All of the first 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 References [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 a common triggering mechanism, installed at the foundation level, 7th, and 13th floors. The location of the instruments is shown in Figure 3. Table VI provides a list of 31 earthquakes recorded in the building during the operational period of the seismic array (October 1972 to the end of the 1980s). The first column

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

Order no.	Reference name	Date	Time (GMT)	Epicentral coordinates	Focal depth (km)	M	I_0
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1	EQ 01	4/12/1974	06:24	44 27 00N, 17 09 00E	—	2.8	5 MCS
2	EQ 02	4/23/1974	03:45	44 42 00N, 17 18 00E	—	3.0	5 MCS
3	EQ 03	2/17/1975	14:24	44 49 29N, 17 00 21E	0	3.3	6 MCS
4	EQ 04	8/09/1975	08:46	44 56 44N, 17 22 35E	0	—	4 MCS
5		10/08/1975	12:15	44 48 00N, 17 18 00E	—	2.7	5 MCS
6	EQ 05	4/20/1977	00:31	44 51 56N, 17 19 40E	9.8	4.7	6 MCS
7		04/28/1977	03:41	44 55 02N, 17 20 59E	14.0	4.0	4 MCS
8	EQ 06	2/17/1979	22:06	44 44 07N, 16 56 24E	10.0	3.7	6 MCS
9	EQ 07	9/07/1979	12:57	44 51 24N, 17 35 07E	10.0	4.0	5 MCS
10	EQ 08	8/08/1980	16:35	44 34 48N, 17 02 24E	0	3.5	—
11	EQ 09	7/24/1981	02:53	44 42 36N, 17 16 12E	5.0	3.0	—
12	EQ 10	7/24/1981	02:55	44 40 12N, 17 14 24E	10.0	2.9	—
13	EQ 11	8/13/1981	02:58	44 42 00N, 17 13 12E	7.0	5.4	8 MCS
14	EQ 12	8/13/1981	Unkn.	—	—	—	—
15	EQ 13	8/13/1981	Unkn.	—	—	—	—
16	EQ 14	8/13/1981	04:37	44 41 24N, 17 34 48E	7.0	3.5	5 MM
17	EQ 15	8/13/1981	11:13	44 43 12N, 17 13 12E	10.0	2.8	—
18	EQ 16	8/14/1981	04:44	44 43 48N, 17 13 12E	10.0	3.2	5 MM
19*	EQ 17	(8/13/1981–8/21/1981)	—	—	—	—	—
20*	EQ 18	(8/13/1981–8/21/1981)	—	—	—	—	—
21*		(8/13/1981–8/21/1981)	—	—	—	—	—
22*	EQ19	8/21/1981	03:30	44 52 48N, 17 22 12E	10.9	3.5	5 MM
23*		8/30/1981	03:11	44 58 47N, 17 21 14E	10.0	2.8	—
24*		(8/30/1981–11/21/1981)	—	—	—	—	—
25		06/14/1982	18:21	44 38 58N, 17 12 00E	0.0	—	4 MCS
26		07/03/1982	03:41	44 46 49N, 17 08 48E	0.0	3.4	5 MCS
27		09/07/1982	21:22	44 57 24N, 17 24 17E	0.0	—	—
28		10/12/1982	01:34	44 50 37N, 17 21 23E	10.0	3.4	5 MM
29		11/22/1982	18:57	44 35 05N, 16 49 48E	10.0	2.9	—
30		09/02/1984	15:14	44 52 56N, 17 16 32E	0.0	4.2	6 MM
31	EQ 20	10/11/1986	01:09	44 52 56N, 17 20 49E	3.1	3.7	5 MM

*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).

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 to be too small to provide reliable information for this work. Columns 3–8 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 VI 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

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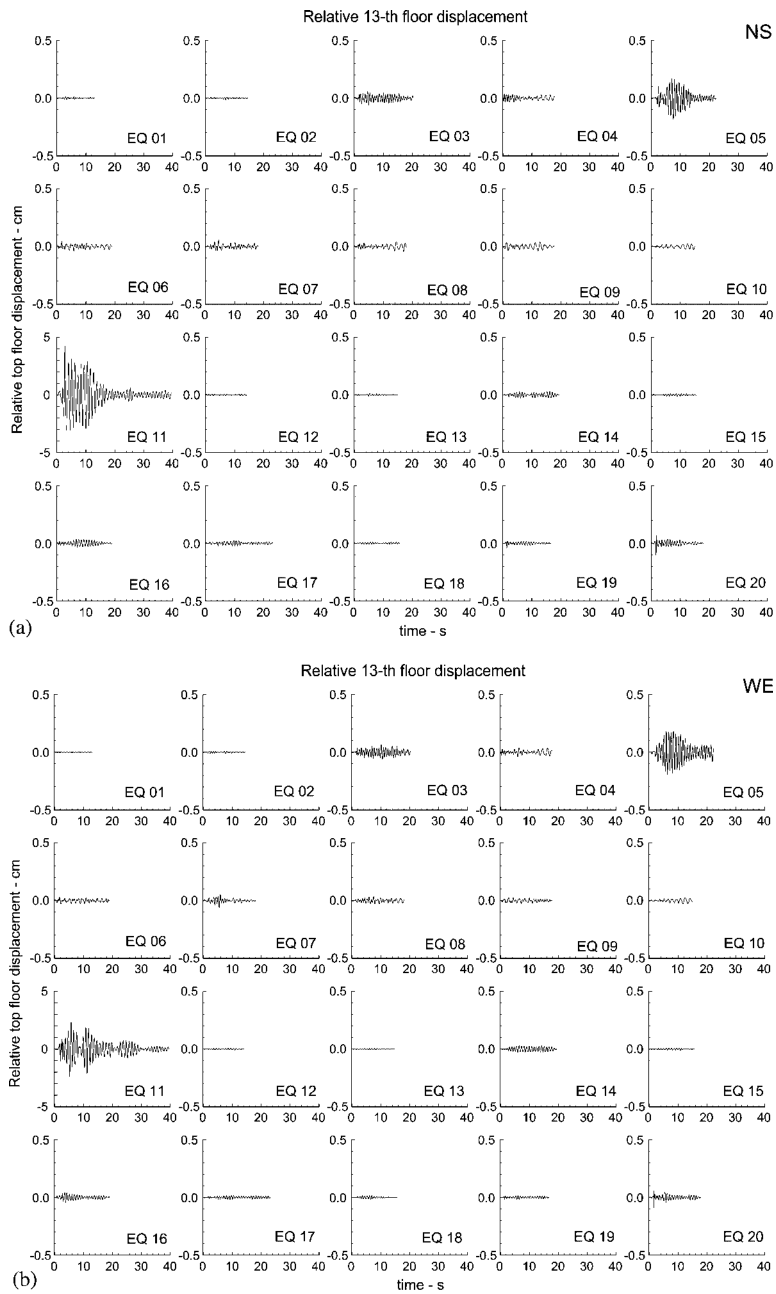

Figure 5. Relative displacements of the 13th floor with respect to the basement computed from the recorded accelerograms: (a) the NS response and (b) the EW response.

seismic source area (Figure 2(b)). This is event EQ 11 in Table VI, and is the same earthquake that was mentioned in the summary of the previous studies of this building.

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Several days after the earthquake, a 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 the 7th to the 10th stories) and in some columns (10th to 14th stories). An 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 Figure 5 is the same for all events (-0.5 to $0.5\,\text{cm}$) except for EQ 11 (-5 to $5\,\text{cm}$). 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 Figure 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].

BRIEF REVIEW OF 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, for completeness of this presentation. 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

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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 the ground to the roof (or from the roof to the ground) is τ_{tot}, the fundamental fixed-base frequency is $f_1 = 1/(4\tau_{\text{tot}})$. We measure τ_{tot} by tracing a virtual impulse that is either input at the ground level or at the roof level. The input and propagating virtual impulses are created simply by taking the 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 the linear system theory is as follows. Let the motion at the 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 the ground level. Hence, conceptually, the impulse response function can be computed as

$$h(t) = \text{FT}^{-1}\{\hat{u}(\omega)/\hat{u}_{\text{ref}}(\omega)\} \tag{1}$$

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

$$h(t) = \text{FT}^{-1}\left\{\frac{\hat{u}(\omega)\bar{\hat{u}}_{\text{ref}}(\omega)}{|\hat{u}_{\text{ref}}(\omega)|^2 + \epsilon}\right\} \tag{2}$$

where the bar indicates the complex conjugate and ϵ is a regularization parameter [8]. In this work, we used $\epsilon = 0.1 * \bar{P}$ when u_{ref} is the ground floor record, and $\epsilon = 0.05 * \bar{P}$ when u_{ref} is the roof record, where $\bar{P}$ is the average power of u_{ref}. For more details 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_{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 τ at one point relative to another. This is best done if u_{ref} is a sensor at the roof or a sensor at the ground floor. The relationship between τ 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 μ is the shear modulus and ρ is the density. Hence, reduction of rigidity due to damage will produce a reduction in the equivalent shear wave velocity, which will produce an increase in the pulse travel time, relative to the travel

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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 $f_1 = 1/(4\tau_{\mathrm{tot}})$. The fundamental fixed-base frequency f_1 is related to the first system frequency f_{sys} by [12]

$$f_{\mathrm{sys}}^{-2} = f_1^{-2} + f_{\mathrm{H}}^{-2} + f_{\mathrm{R}}^{-2} \tag{3}$$

where f_{H} and f_{R} represent the horizontal and rocking frequencies of the system, which depend on the foundation horizontal and rocking compliances. Equation (3) implies that $f_1 > f_{\mathrm{sys}}$. 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 $f_1 = 1/(4\tau_{\mathrm{tot}})$ 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 $1/(4\tau_{\mathrm{tot}})$ gives f_1, while the Fourier analysis gives f_{sys}, and that the measurement of τ_{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 shows 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 (1–20). 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 second 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 in f_{sys} for the second mode is more apparent, for the NS response from about 4.6 to 4.0 Hz, and for the EW response from about 5.0 to 4.7 Hz. For both modes, f_{sys} begin essentially at the frequencies measured during the forced vibration tests in October 1972, and then gradually decrease and level off during events 5–10. After what appears to be a permanent drop, during event 11, these frequencies are again nearly constant during the remaining events, 13–20. The gradual decrease during events 1–5, and the largest drop during event 11, is apparently associated with the cracking of structural concrete, of nonstructural 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].

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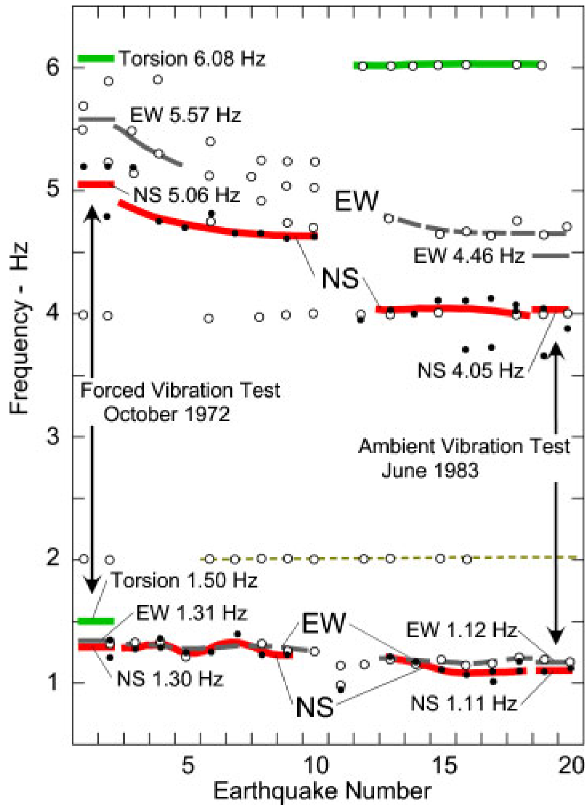

Figure 6. Soil–structure system frequencies for the first and second modes of vibration estimated from the earthquake records over a period of 12 years (1974–1986) for the NS (full dots) and the EW response (open dots). Published values based on a forced vibration test conducted before the earthquakes (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.

Figure 6 also shows recurring spectral peaks near 2 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 response 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 13th 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 $f_1 = 1/(4\tau_{\mathrm{tot}})$, 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 13th floor (see Figure 3(c)). For input impulse at the

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Table VII. 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).

	NS			EW		
EQ number	f_1 (Hz)	f_{sys} (Hz)	d_{max} (cm)	f_1 (Hz)	f_{sys} (Hz)	d_{max} (cm)
01	2.01	1.40	0.013	1.93	—	0.007
02	1.93	1.35	0.014	2.10	1.32	0.012
03	1.85	1.28	0.062	2.14	1.30	0.066
04	2.10	1.30	0.038	2.07	1.33	0.039
05	1.85	1.21	0.185	1.93	1.21	0.194
06	2.01	1.25	0.051	2.07	1.28	0.033
07	2.01	1.38	0.058	2.20	1.30	0.064
08	2.01	1.28	0.044	2.16	1.30	0.036
09	2.01	—	0.040	2.10	1.26	0.027
10	1.98	—	0.027	2.10	1.25	0.028
11	1.65	0.90	4.250	1.78	0.99	2.407
12	1.85	0.99	0.009	2.20	1.15	0.010
13	2.10	1.20	0.014	2.20	1.19	0.008
14	2.01	1.15	0.032	2.10	1.13	0.030
15	1.93	1.09	0.012	2.20	1.10	0.011
16	1.93	1.10	0.037	2.01	1.15	0.045
17	1.93	1.12	0.026	2.10	1.17	0.019
18	2.01	1.11	0.010	2.10	1.20	0.017
19	—	1.10	0.042	2.01	1.19	0.018
20	2.07	1.11	0.106	2.07	1.18	0.089

basement propagating up, reflections from the roof interfere with the upward propagating wave, and make reading the arrival time at the 13th floor difficult and ambiguous. To avoid this problem, we used only the results for a virtual source at the 13th floor, and measured the wave travel time by tracing the acausal wave propagating from the 13th floor to the basement (for a discussion on the acausal wave, see [8–10]). We also adjusted the measured wave travel time between the basement and the 13th floor to get τ_{tot}, for the estimation of $f_1 = 1/(4\tau_{tot})$, as follows. Because the instrument on the 13th 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 assume that the upward propagating waves effectively reflect off the roof surface.

Table VII 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.

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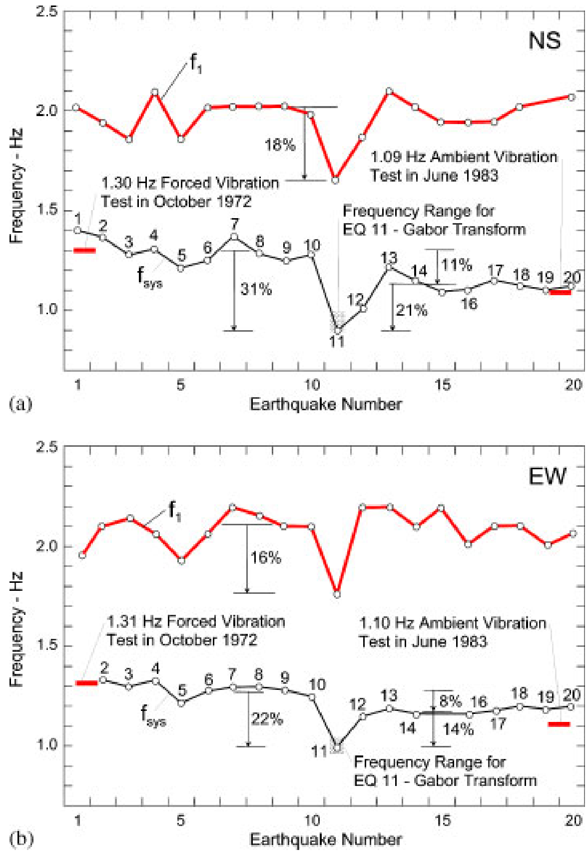

Figure 7. System frequency f_{sys} and fixed-base frequency f_1 during the 20 earthquakes are plotted versus the earthquake event number. The values of f_1 and of f_{sys} are interconnected by straight lines to help visualize the trends. The values determined from the forced vibration test in 1972, and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

It can be seen from Figure 7 that for all earthquakes $f_{sys} < f_1$, which is consistent with Equation (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 the NS motions, and by about 16 and 22%, respectively, for the 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 a larger response. Figure 7 also shows that the estimates of f_{sys} from the forced vibration test in 1972, and from the 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}

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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 first mode, relative displacement amplitude associated with the first 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 $2\sigma_t \times 2\sigma_\nu$, 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 the 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 the trigger until $t \sim 20$ s, f_{sys} is essentially constant. A minor local decrease in f_{sys} can be noticed for the EW response in Figure 8(b) during a time interval with a 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 the NS response and points 1 and 3-4-5 for the 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 the NS response, and for peaks 5-6-7 and 15-16-17 for the EW response. The overall behavior of the frequency versus amplitude of all the peaks in Figure 9 is in excellent

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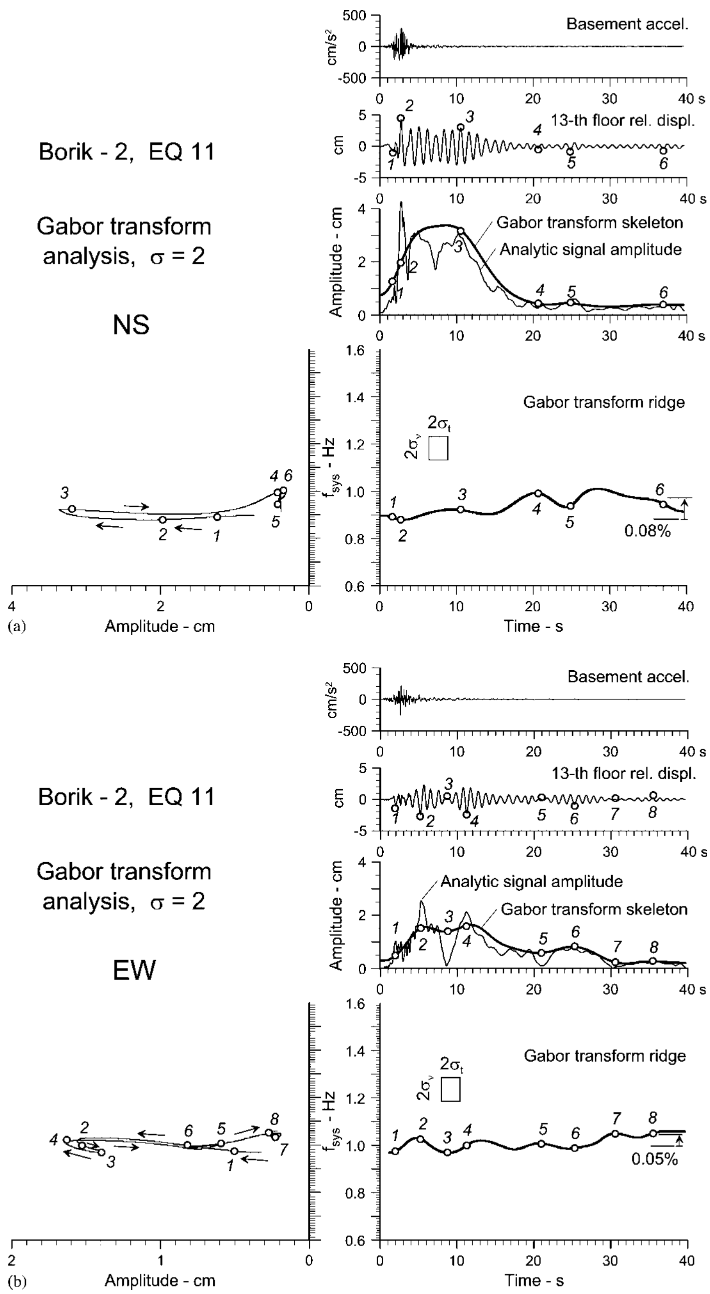

Figure 8. Results of time–frequency analysis based on Gabor transform: (a) the NS response and (b) the EW response.

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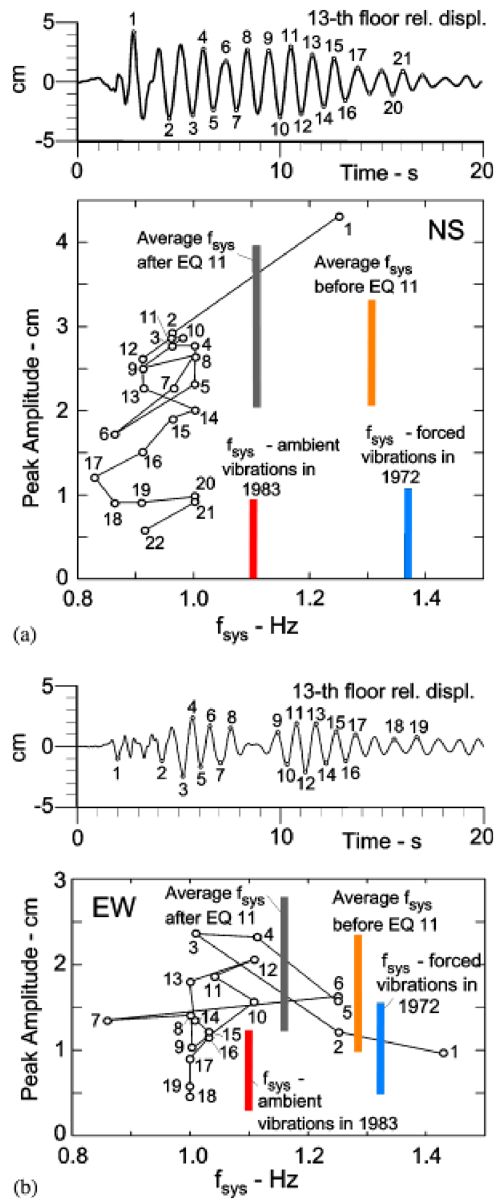

Figure 9. Results of zero-crossing analysis: (a) the NS response and (b) the EW response.

agreement with the corresponding smoother results using the Gabor transform in Figure 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).

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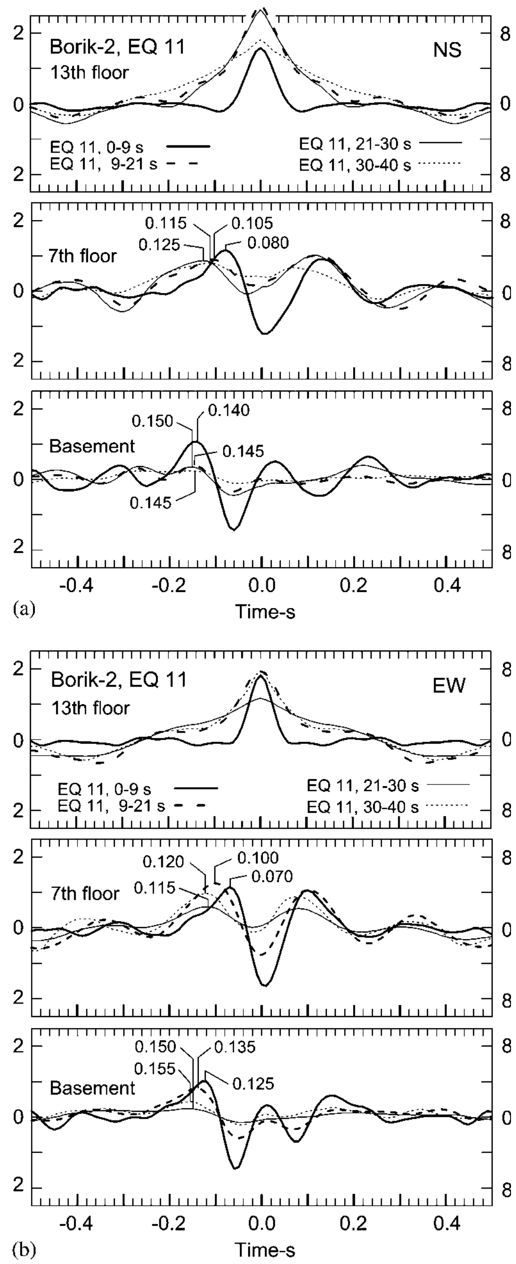

Figure 10. Impulse response functions computed for four time windows during event EQ 11: 0–9, 10–21, 22–30, and 31–40 s: (a) the NS response and (b) the EW response. The right vertical amplitude scale is for segment 0–9 s and the left scale is for all other segments.

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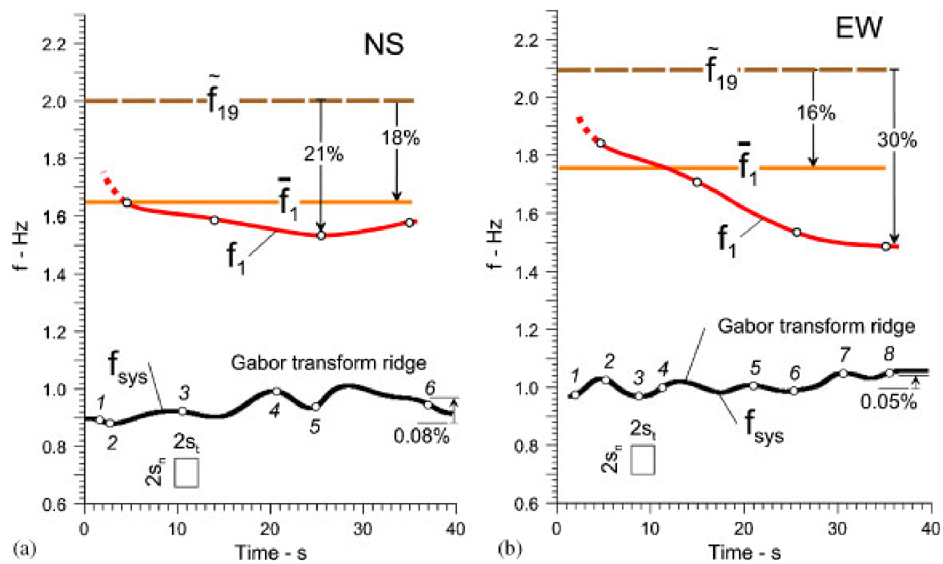

Figure 11. Comparison of the window estimates of f_1 during event EQ 11 (interpolated by straight lines), with the 'instantaneous' f_{sys} during event EQ 11, and with $\bar{f}_1$ = the average value of f_1 for event EQ 11 (estimated from the total length of the record), and $\tilde{f}_{19}$ = the average value of f_1 during the 19 small events: (a) the NS response and (b) the EW response.

Windowed impulse response analysis and variations in f_1 during EQ 11

For the largest event, EQ 11, we computed impulse responses in four time windows: 0–9, 10–21, 21–30, and 30–40 s, and obtained corresponding estimates of the 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 Figure 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, the 7th, and the 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 ϵ in Equation (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 (0–40 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 a 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

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almost constant, between 0.9 and 1.0 Hz for the NS response (Figure 9(a)), and between 0.97 and 1.10 Hz for the EW response (Figure 9(b)). This implies 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 Figure 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 second mode, as shown in Figure 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 the 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 the 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 to 0.85 Hz), and 47% for the maximum drop in windows considered (from 1.25 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.

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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 the 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 analyses: 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 whether 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 2 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 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

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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 a 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 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 a 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 the 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 thus paper was smaller than those during the forced vibration tests). Such information is much needed for the development and calibration of realistic models for the prediction of the seismic response of structures for structural health monitoring and design, and for the 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 the 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

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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.

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