THRESHOLD CHANGES IN BUILDING FREQUENCIES OF VIBRATION ASSOCIATED WITH STRUCTURAL DAMAGE – STUDY OF FULL-SCALE OBSERVATIONS IN THE BORIK-2 BUILDING IN FORMER YUGOSLAVIA

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ABSTRACT :

A critical issue that has to be addressed before structural health monitoring systems can be implemented for real time earthquake damage detection and early warning is that the damage sensitive features may vary due to factors other than earthquake damage. This paper presents a case study of the response of a 14-story prefabricated RC building in Banja Luka, Republic of Srpska, to 20 earthquakes over a period of 12 years. The analysis shows consistency of the estimates of system frequency from the earthquake response data, from forced vibration tests before the earthquakes and ambient vibration test 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.

KEYWORDS: Earthquake damage detection; structural health monitoring; impulse response analysis; Borik-2 building in Banja Luka; IMS type prefabricated construction.

1. 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]. 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 [2]. 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. 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. 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 unique and invaluable starting point for realistic testing and for verification of structural health monitoring methods intended for use in real buildings.

Two types of analyses were carried out in this work: (1) of travel times of seismic waves propagating through the building, and (2) of energy distributions of the response (Fourier or windowed Fourier spectra). The wave travel times were estimated by tracing impulses radiated from virtual sources created by deconvolution of the recorded seismic response [3]. 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 frequency of the soil-foundation-structure system f_{sys} [4]. This frequency is different from the fixed-base frequency f_1, the difference being more significant for stiffer structures compared to the dynamic stiffness of the foundation soil. The important consequence is that calibrating stiffness of analytical models of a structure to f_{sys} in lieu of the fundamental fixed base frequency f_1 underestimates the actual stiffness of the structure since $f_{sys} < f_1$.

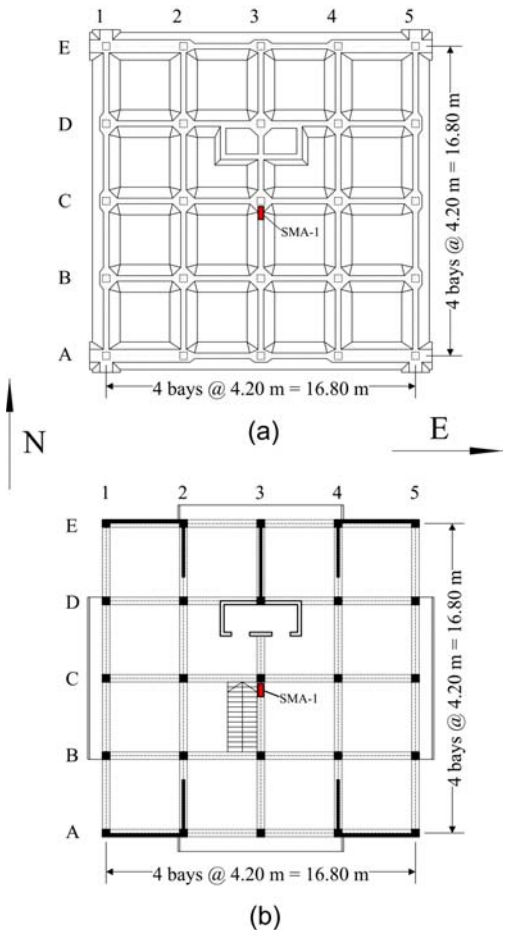

Fig.1a,b. Borik-2 building: (a) foundation plan, and (b) typical floor plan.

2. THE BUILDING AND PREVIOUS STUDIES

2.1 The Building and the Site

Borik-2 is a 14-story apartment building located in the settlement “Borik”, 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 1 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 [5]. It 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 footing is only about 4 m above the Marley clay layer,

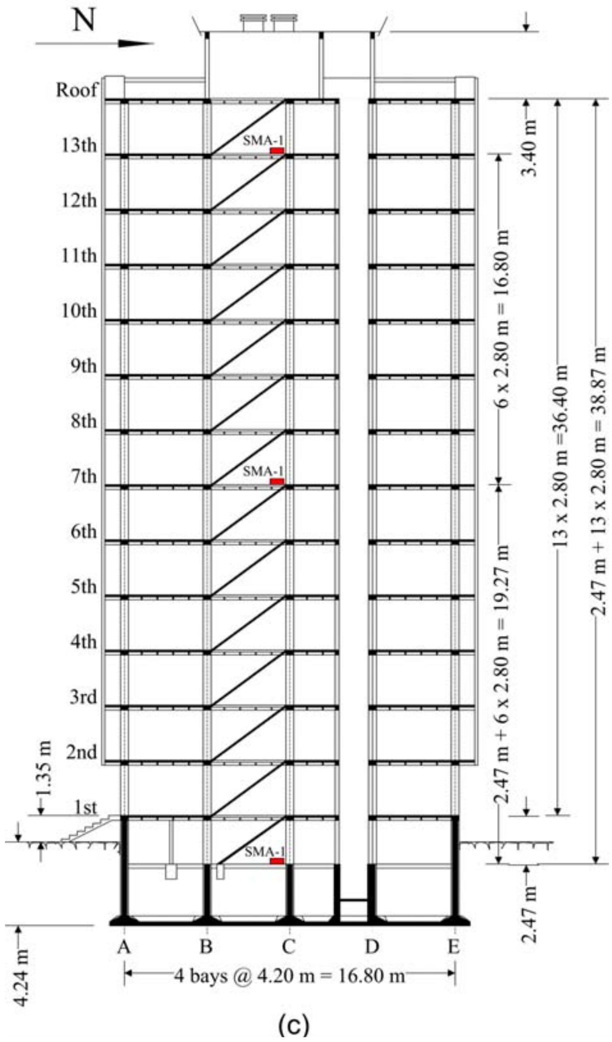

Fig.1c. Borik-2 building: North-South cross-section.

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 a typical building in southern California. Therefore the nonlinearities in the soil response at the site of the Borik-2 building occur at relatively higher levels of shaking.

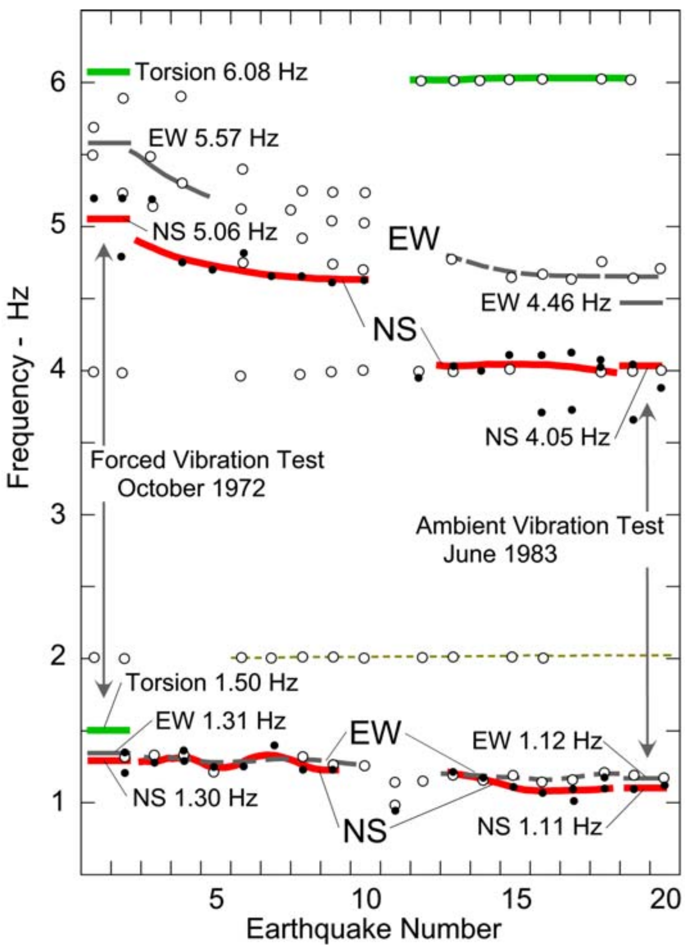

Fig. 2. System frequencies for the 1st and 2nd modes of vibration during 20 earthquakes from 1974 to 1986, for NS (full dots) and EW (open dots) response. Published data on forced and ambient vibration tests, and for torsional frequencies [11] are also shown.

2.2 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, in July 1972, prior to the installation of most of the partition walls and the other nonstructural elements, and 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 [6]. After the second forced vibration test, when the structure was essentially completed, the measured fundamental system frequencies were 1.31 Hz and 1.30 Hz for EW and NS directions respectively. 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 [7].

2.3 Earthquake Records and Observed Damage

The Borik-2 building was instrumented in October 1972, and was part of the Yugoslav strong motion network, operated by IZIIS [8]. Recording was with three SMA-1 tri-axial accelerographs, with common triggering mechanism, installed at the foundation level, 7th, and 13th floors. The location of the instruments is shown in Fig. 1. 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. This is event EQ 11 in the figures. 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 [9]) was observed, which shows that the building has worked essentially in the "elastic range" during all recorded strong earthquakes thus far.

3. METHODOLOGY

3.1 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 [10]. 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.

3.2 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 $\tau_{\rm tot}$, the fundamental fixed base frequency is $f_1 = 1/(4\tau_{\rm tot})$.The relationship between the physical problem (propagating pulse) and linear system theory is as follows. Let the motion at ground level, $u_{\rm ref}(t)$, be the input and the output be the motion at the roof, or at any floor, $u(t)$. Then the transfer function of $u_{\rm ref}(t)$ with respect to itself is unity ($\hat{u}_{\rm ref}(\omega)/\hat{u}_{\rm ref}(\omega)=1$) 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 $\hat{h}(\omega)=\hat{u}(\omega)/\hat{u}_{\rm ref}(\omega)$, 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. Once $h(t)$ has been computed for different sensors, relative to a reference motion $u_{\rm ref}$, 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. In this paper, we show results only for the *global* change in the building by monitoring $f_1 = 1/(4\tau_{\rm tot})$. The fundamental fixed-base frequency f_1 is related to the first system frequency $f_{\rm sys}$ and to $f_{\rm H}$ and $f_{\rm R}$ by $f_{\rm sys}^{-2} = f_1^{-2} + f_H^{-2} + f_R^{-2}$, where $f_{\rm H}$ and $f_{\rm R}$ represent the horizontal and rocking frequencies of rigid structure, and depend on the foundation horizontal and rocking compliances [4]. This implies $f_1 > f_{\rm sys}$.

4. RESULTS AND ANALYSIS

4.1 Comparison of f_{sys} during Earthquakes, Forced and Ambient Vibrations

Fig.2 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 [11]. 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 2nd forced-vibration test, conducted in October 1972, and the dashes on the right correspond to the frequencies measured from the ambient vibration test, conducted in June 1983 [7]. The trends are shown by a continuous line for the NS response, and by dashed line for the EW response. 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 $f_{\rm sys}$ 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 $f_{\rm sys}$ for the 2nd 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, $f_{\rm sys}$ begin essentially at the frequencies measured during the forced-vibration tests in October 1972, and then gradually decrease and level off during events 5 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 $f_{\rm sys}$ 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 [12]) or changes in mass. Fig. 2 also shows recurring spectral peaks near 2 Hz and 4 Hz, which are not changed by event 11. We do not have a plausible interpretation of what these peaks represent.

4.2 Impulse Responses analysis and Variations of f_1

Fig. 3 shows 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 only to help emphasize the trends. It can be seen that for all earthquakes $f_{sys} < f_1$, and that *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. 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% for EW and 15% for NS directions. Finally, Fig. 3 shows that f_1 and f_{sys} consistently follow all small fluctuations of frequency from one event to the next. As they were 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.

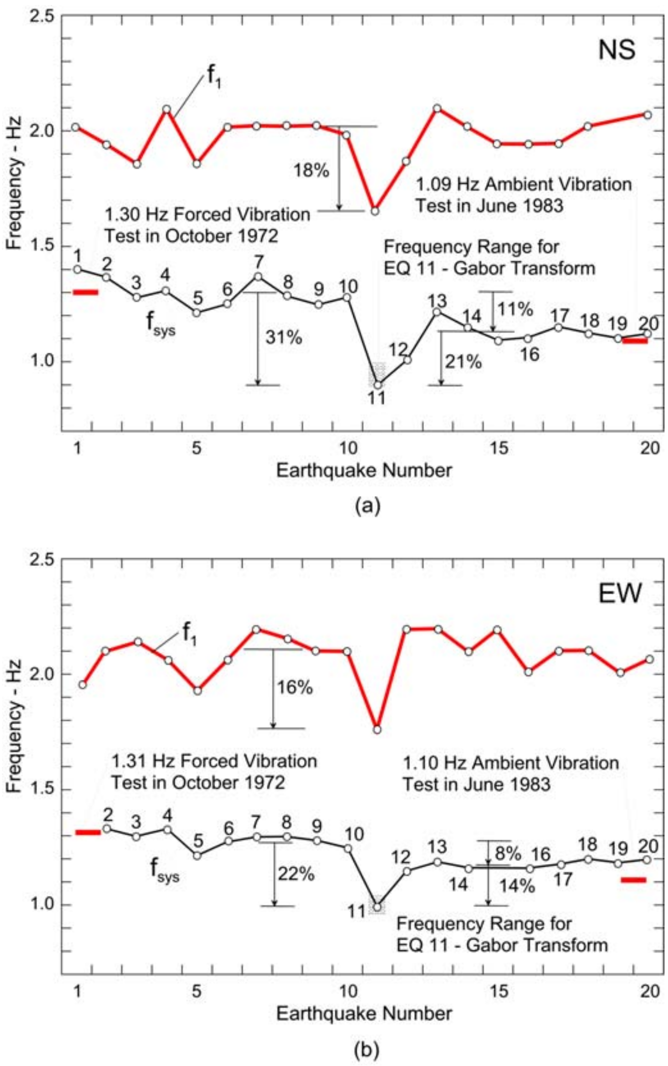

Fig. 3. System , f_{sys}, and fixed-base, f_1, frequencies during 20 earthquakes plotted versus the earthquake event number. The values are interconnected by straight lines to help visualize the trends. The frequencies determined from forced vibration test in 1972, and from ambient vibration test in 1983 are also shown. (a) NS and (b) EW response.

4.3 f_1 and f_{sys} during 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. Fig. 4 shows the interval estimates of f_1 and f_{sys} 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, 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, and between 0.97 and 1.10 Hz for the EW response. 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. Going back to the variations of f_{sys} and f_1 on a longer time scale (Fig. 3), 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 more pronounced for the 2nd mode, as shown in Fig. 2. 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 intermediate and small levels of strong shaking f_{sys} often returns gradually to its pre-earthquake values [3]. 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 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" [13].

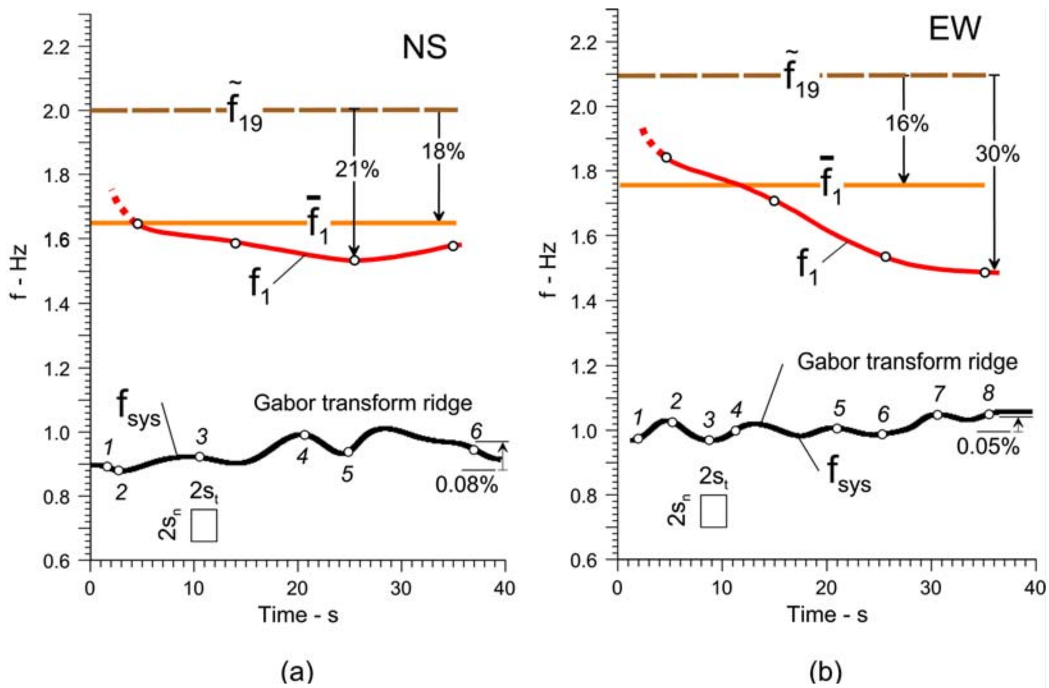

Fig. 4. Comparison of the window estimates of f_1 during event EQ 11 (interpolated by straight lines), with the "instantaneous" f_{sys} during event EQ 11, and with $\bar{f}_1$, the average value of f_1 for event EQ 11 (estimated from the total length f the record), and $\tilde{f}_{19}$ = the average value of f_1 during the 19 small events. (a) NS response. (b) EW response.

4.4 Can one Use f_{sys} as a Proxy for f_1?

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. This implies a factor of about 2.5 error in estimating the building stiffness $(\sim f_1^2)$ when f_{sys} is used instead, as has been the case for calibrating structural models of this building in most of the previous work, which is summarized in [11]. 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.

5. SUMMARY AND CONCLUSIONS

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, and based on ambient vibration test conducted in 1983, about two years after the largest earthquake shaking by the 1981 earthquake (the 11th earthquake event analyzed), and before the last small earthquake analyzed, which occurred in 1986. 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. 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.

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.

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). 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.

The data we presented here for Borik-2 building constitutes an important addition to the database for instrumented buildings, which approached the damage threshold. All other buildings we studied so far exceeded this threshold or experienced serious damage [14-16].

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