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AMBIENT VIBRATION TESTS OF STRUCTURES–A REVIEW

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ABSTRACT

A literature review on the subject of ambient vibration testing is presented, along with a review of a recent study to illustrate state-of-the-art in the application of the ambient vibration method. Reports on testing of full-scale structures by the ambient vibration method began to appear regularly around 1970, with about 3/4 of all contributions devoted to the experiments in buildings, dams, chimneys and silos, and about 1/4 to bridges. Since 1985, there are only about 3 papers per year, world-wide, dealing with this subject. The reviewed study is of a seven-story reinforced concrete building in Van Nuys, California, damaged by the 17 January 1994 Northridge earthquake and its early aftershocks. Two detailed ambient vibration surveys were conducted soon after the earthquake. The apparent frequencies and two- and three-dimensional mode shapes for longitudinal, transverse and vertical vibrations are presented in this paper. The attempts to detect the highly localized damage by simple spectral analyses of the ambient noise data were not successful. It is suggested that very high spatial resolution of recording is required to identify localized column and beam damages. The loss of axial capacity of the damaged columns could be seen in the vertical response of the columns, but similar moderate or weak damage typically would not be noticed in ambient vibration surveys.

KEYWORDS: Ambient Vibration Tests, Full-Scale Experiments, Structural Health Monitoring, Damage Detection, Spatial Resolution

INTRODUCTION

The models used in dynamic analyses of structures are idealizations created to represent the response of real structures to various dynamic loads (strong earthquake shaking, strong winds, explosions etc.). These models can be verified by conducting full-scale ambient and forced vibration experiments (Hudson, 1970a; Hudson et al., 1972; Trifunac and Todorovska, 1999a). Both of these can be used to identify the structural characteristics (frequencies of vibration, damping ratios and mode shapes).

The ambient vibration tests describe the *linear* behavior of structures, since the amplitudes of vibration are small. They can be used also to describe the linear behavior of damaged structures and of their components, and can help in developing time and amplitude-dependent structural models and analysis algorithms, for use in structural health monitoring and in structural control studies. Therefore, the development of experimental methods for in-situ measurement of full-scale partially damaged structures is of considerable interest (Ivanović et al., 1999a, 1999b, Trifunac et al., 2000a, 2000b). An advantage of the ambient vibration over the forced vibration surveys is that usually only light equipment and smaller number of operators are required. The sources of excitation are wind, microtremors, microseisms, and various local random and periodic sources.

The forced vibration tests may require large forces to produce useful (larger) response amplitudes of full-scale structures. The vibration generator (a shaker) is usually located on top of the building. This leads to more prominent excitation of the modes of vibration that have large amplitudes at the higher levels of the structures. Also, the paths of waves propagating through the structure are different from those in case of earthquake ground shaking, ambient noise, or wind excitation, and cautious interpretation of the results is required to take such differences into account (Luco et al., 1975, 1986, 1987, 1988).

LITERATURE REVIEW

In California, ambient and forced vibration tests of structures have been conducted for about 65 years. The U.S. Coast and Geodetic Survey started measuring the fundamental periods of buildings by ambient vibrations tests in the early 1930's (Carder, 1936; Hudson, 1970a). Some 30 years later, Crawford and Ward (1964) and Ward and Crawford (1966) revived the interest in this method and showed that it can be used to determine the lowest frequencies and modes of vibration of full-scale structures. Trifunac (1970a, 1970b) used wind and micro-tremor induced vibrations to test a twenty-two and a thirty-nine story steel frame buildings. Few years later, he compared the results of forced vibration experiments on the same two buildings with the results of ambient vibration surveys (Trifunac, 1972). The results of both tests were consistent and comparable. Udwadia and Trifunac (1973) presented results of ambient vibration tests of four buildings of different type (a twenty-two story steel frame building, a thirty-nine story steel frame building, a nine-story steel frame building and a nine-story reinforced concrete building), and discussed the changes in the ambient vibration response prior and after an earthquake. They analyzed the effects of interaction between soft soil and a stiff structure immediately and long after an earthquake. Throughout the 1970's and the 1980's, ambient and forced vibration tests were used to compare small amplitude (linear) with larger amplitude response and to find the pre- and post-earthquake apparent frequencies of full-scale structures (Mulhern and Maley, 1973; Udwadia and Trifunac, 1974), and to identify the three-dimensional nature of deformations accompanying the apparent frequencies of response (Foutch et al., 1975; Luco et al., 1975, 1977; Moslem and Trifunac, 1986). They were also used to resolve the contradictory interpretations of the significance of the soil-structure interaction and of the causes of non-linearity (in the soil or the structure) in observed response of buildings to strong earthquake excitation (Luco et al., 1986, 1987, 1988; Wong et al., 1988). During the 1990's, ambient vibration tests continued to contribute to in-depth studies of the changes in structural properties (Mendoza et al., 1991) and towards further development of structural identification methods (Kadakal and Yüzügüllü, 1996).

The ambient vibration method was validated by direct comparison with small amplitude forced vibration tests (Bauwkamp and Stephen, 1973; Farrar and James, 1995; Trifunac, 1972), procedures were proposed for its use in structural health monitoring (Beck et al.l 1994a), and its accuracy and required time series analysis procedures were explored (Gerseh and Martinelli, 1979; Taoka, 1976).

One of the most frequent uses of ambient vibration analyses involves identification of natural frequencies, mode-shapes of vibration and equivalent viscous damping parameters of various full scale buildings (Ivanović and Trifunac, 1995; Ko and Bao, 1985; Marshall et al., 1994; McLamore et al., 1971; Midorikawa, 1990; Ristić et al., 1998; Rodriguez-Cuevas, 1989; Serrano, 1974; Slastan and Foissner, 1995; Stubbs and McLamore, 1973; Taskov and Krstevska, 1998), bridges (Abdel-Ghaffar et al., 1984; Abdel-Ghaffar and Housner, 1977, 1978; Brownjohn et al., 1986, 1987; 1989, 1992; Higashihara et al., 1987; Ventura et al., 1994, 1996a, 1996b; Wilson and Lui, 1990), dams (Abdel-Ghaffar and Scott, 1981), and nuclear power plants (Luz et al., 1983). Ambient vibration tests have been used also to improve the parameter estimates and the overall definition and properties of models of full scale structures (Cherry and Topf, 1971; Douglas and Trabert, 1973; Safak and Celebi, 1990; Stephen et al., 1985; Ward and Rainer, 1972; Yang and Liu, 1989), for relative calibration of different small amplitude excitations: ambient and forced vibrations, man-induced excitation and drop of weights (Gates et al., 1990), for modeling bridges (Brownjohn et al., 1999; Jacob et al., 1996; Wahab and DeRoeck, 1998; Wilson et al., 1991), chimneys (Kapsarov and Milicevic, 1986), dams (Daniell and Taylor, 1999; Lu et al., 1986), nuclear power plants (Klasky et al., 1973), water tanks (Housner and Haroun, 1979), and to evaluate radiation damping of existing rockfill dams (Ohmachi and Nakamoto, 1988).

Ambient vibration testing has been used extensively to identify and to monitor changes of system frequencies between small (ambient noise) and large (earthquake shaking) response amplitudes (Gates, 1993; Hart et al., 1972, 1975; Ivanović et al., 2000; Lekidis et al., 1998; Oliveira et al., 1982; Phan et al., 1992; Sandi et al., 1986; Shah et al., 1973; Stark et al., 1991; Marshall et al., 1994), undamaged versus damaged, and versus repaired structures (Carydis and Mouzakis, 1982, 1986; Beck et al., 1994b), changes of system parameters during failure test (Kato and Shimada, 1986), variability of the system during construction (Schuster et al., 1994; Skrinar and Strukelj, 1996; Ventura and Schuster, 1996), variability in response of similar buildings with respect to different soil conditions (Kircher and Shah, 1976), and type of excitation (Kircher, 1977).

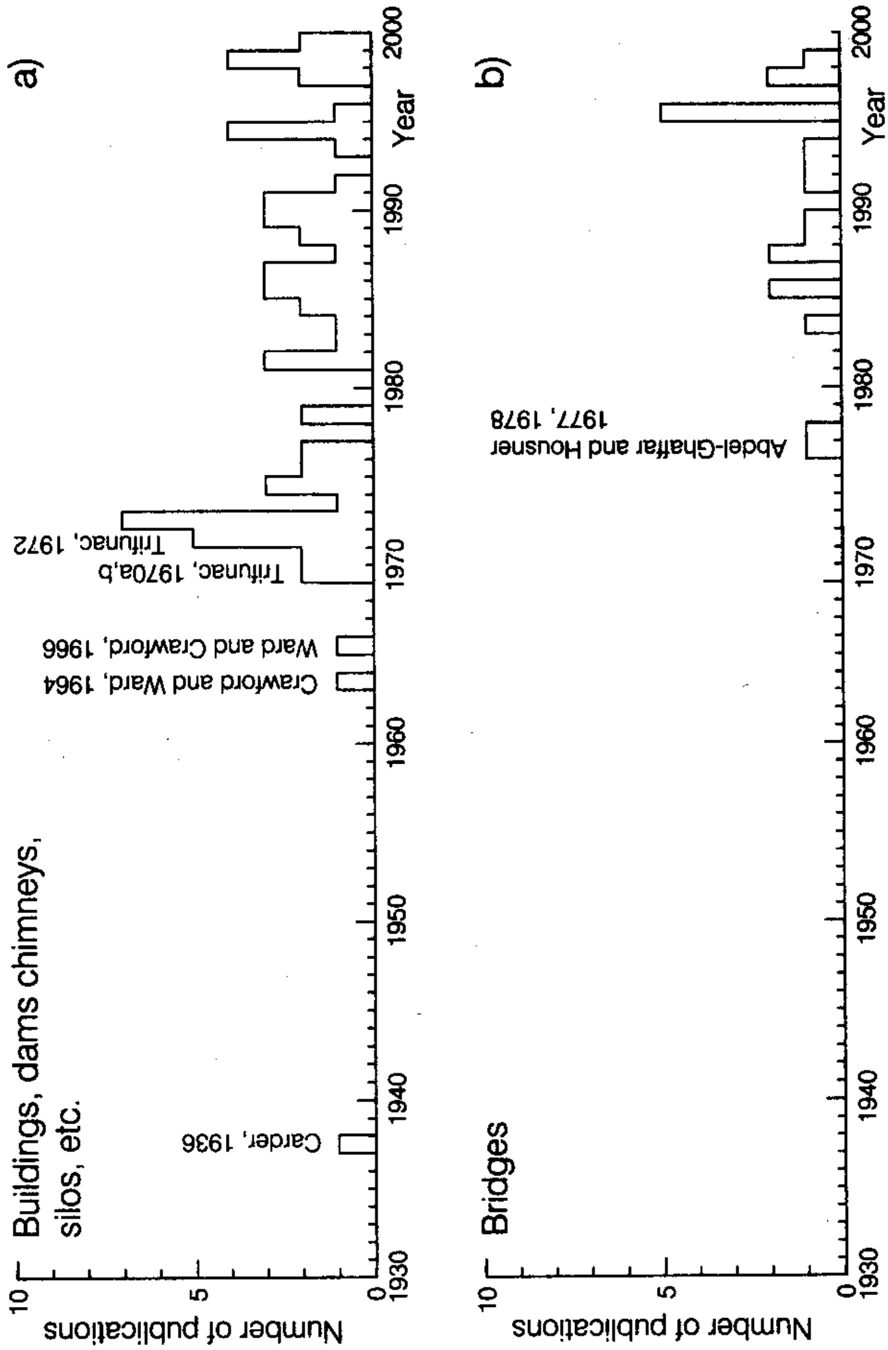


Fig. 1 Frequency of occurrence of published contributions to the subject of ambient vibration testing of full-scale structures: (a) buildings, dams, chimneys, silos,... (b) bridges

Of 87 example papers cited, which are directly related to the subject of testing the full-scale structures, 77 percent describe use of ambient vibration measurements to test buildings, dams, nuclear power plants, chimneys etc. The remaining 23 percent describe the ambient vibration tests of bridges (Figure 1). It is seen that the use of ambient vibration tests essentially begins in 1970, after which about two papers are published per year in the category of buildings and other structures. Even though the first cited papers on ambient vibration tests of bridges are from 1977 and 1978, more frequent tests appear to be conducted only after 1985, producing about one paper per year.

Ambient vibration noise (microtremors and microseisms) has been used extensively in engineering studies of amplification of seismic waves by soft geological and soil layers. Discussion of this subject is outside the scope of this paper. A curious reader may wish to read Lermo and Chavez-Garcia (1994) or Trifunac and Todorovska (2000) for general review and further references.

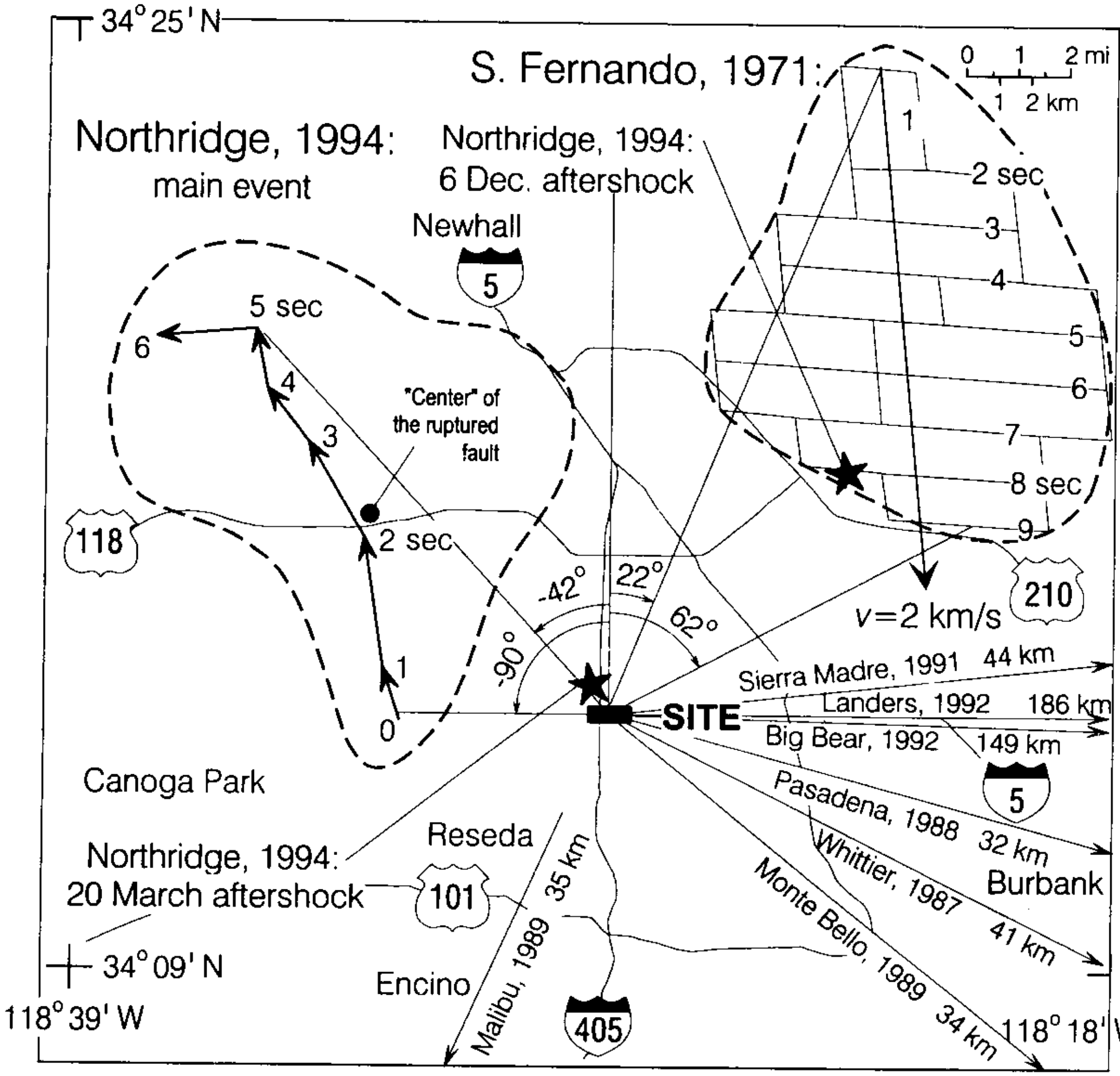


Fig. 2 General setting of the VN7SH building in Central San Fernando Valley: location of the building relative to the fault planes of the 1971 San Fernando and 1994 Northridge earthquakes (their horizontal projections are shown by dashed lines) and the epicenters of two Northridge aftershocks (solid stars) and other earthquakes with epicenters outside the map

ILLUSTRATION OF THE METHOD

The ambient vibration method is next illustrated in the case of an instrumented seven-story reinforced concrete (RC) hotel building in Van Nuys, California (VN7SH). Two ambient vibration surveys were conducted on this building in the spring of 1994, following the Northridge earthquake of January 17, 1994 $(M_L = 6.4, R = 1.5\,\text{km})$ and its early aftershocks that damaged the building severely. The building response to this earthquake as well as to eleven other earthquakes was recorded by permanent strong motion instrumentation. The purpose of the ambient vibration surveys was to find whether the damage could be detected using established analysis procedures, and to search for new procedures of damage detection in structures by using the ambient noise data. Detailed surveys of the damage were conducted at the time of the experiments and are reported in Trifunac et al. (1999b) along with a summary of the recorded strong motion response. In what follows, (1) the building is described, (2) the two ambient vibration surveys are briefly described and their results (reported previously by Ivanović et al., (2000)) are summarized, and (3) results of selected other studies of this building are briefly summarized as they relate to the ambient vibration surveys.

1. Description of the Building

Location: The Van Nuys seven-story hotel (VN7SH) is located in central San Fernando Valley of the Los Angeles metropolitan area (at 34.221°N and 118.471°W), north-west from downtown Los Angeles. Figure 2 shows San Fernando valley and the building location, relative to the major freeways and to the horizontal projections of the fault planes of 1971 San Fernando and 1994 Northridge earthquakes (Trifunac, 1974; Wald and Heaton, 1996), which both damaged the building and were recorded in the building, the epicenters of two Northridge aftershocks, and directions and epicentral distances to seven other earthquakes recorded in the building (Trifunac et al., 1999b).

Design: The building was designed in 1965, constructed in 1966 (Blume et al., 1973; Freeman and Honda, 1973; Mulhern and Maley, 1973), and served as a hotel at the time of the 1994 Northridge earthquake. Figure 3 shows a plan view of a typical floor in (a), a plan view of the foundation layout in (b), a side view of the building frame in (c), and a typical soil-boring log data at the building site in (d). The building is 18.9 × 45.7 m in plan. The typical framing consists of columns spaced at 6.1 m centers in the transverse direction and 5.8 m centers in the longitudinal direction. Spandrel beams surround the perimeter of the structure. Lateral forces in each direction are resisted by interior column-slab frames and exterior column spandrel beam frames. The added stiffness in the exterior frames associated with the spandrel beams creates exterior frames that are roughly twice as stiff as interior frames. The floor system is reinforced concrete flat slab, 25.4 cm thick at the second floor, 21.6 cm thick at the third to seventh floors and 20.3 cm thick at the roof.

Site Conditions: The building is situated on undifferentiated Holocene alluvium, uncemented and unconsolidated, with thickness < 30 m and age < 10,000 years (Trifunac and Todorovska, 1998). The average shear wave velocity in the top 30 m of soil is 300 m/s. The soil-boring log in Figure 3(d) shows that the underlying soil consists primarily of fine sandy silts and silty fine sands. The foundation system (Figure 3(b)) consists of 96.5 cm deep pile caps, supported by groups of two to four poured-in-place 61 cm diameter reinforced concrete friction piles. These are centered under the main building columns. All the pile caps are connected by a grid of beams. Each pile is roughly 12.2 m long and has design capacity of over 444.82 × 10^3 N vertical load and up to 88.96 × 10^3 N lateral load. The structure is constructed of normal weight reinforced concrete (Blume and Association, 1973).

Earthquake Damage: The $M_L = 6.6$ San Fernando earthquake of February 9, 1971 (Figure 2) (Trifunac, 1974) caused minor structural damage (Blume and Association, 1973). Epoxy was used to repair spalled concrete of the second floor beam column joints on the north side and east end of the building. The recorded peak accelerations in the building were: 0.13g (L), 0.24g (T) and 0.18g (V) at the base, and 0.32g (L), 0.39g (T) and 0.22g (V) at the roof, along the longitudinal (L), transverse (T) and vertical (V) axes of symmetry.

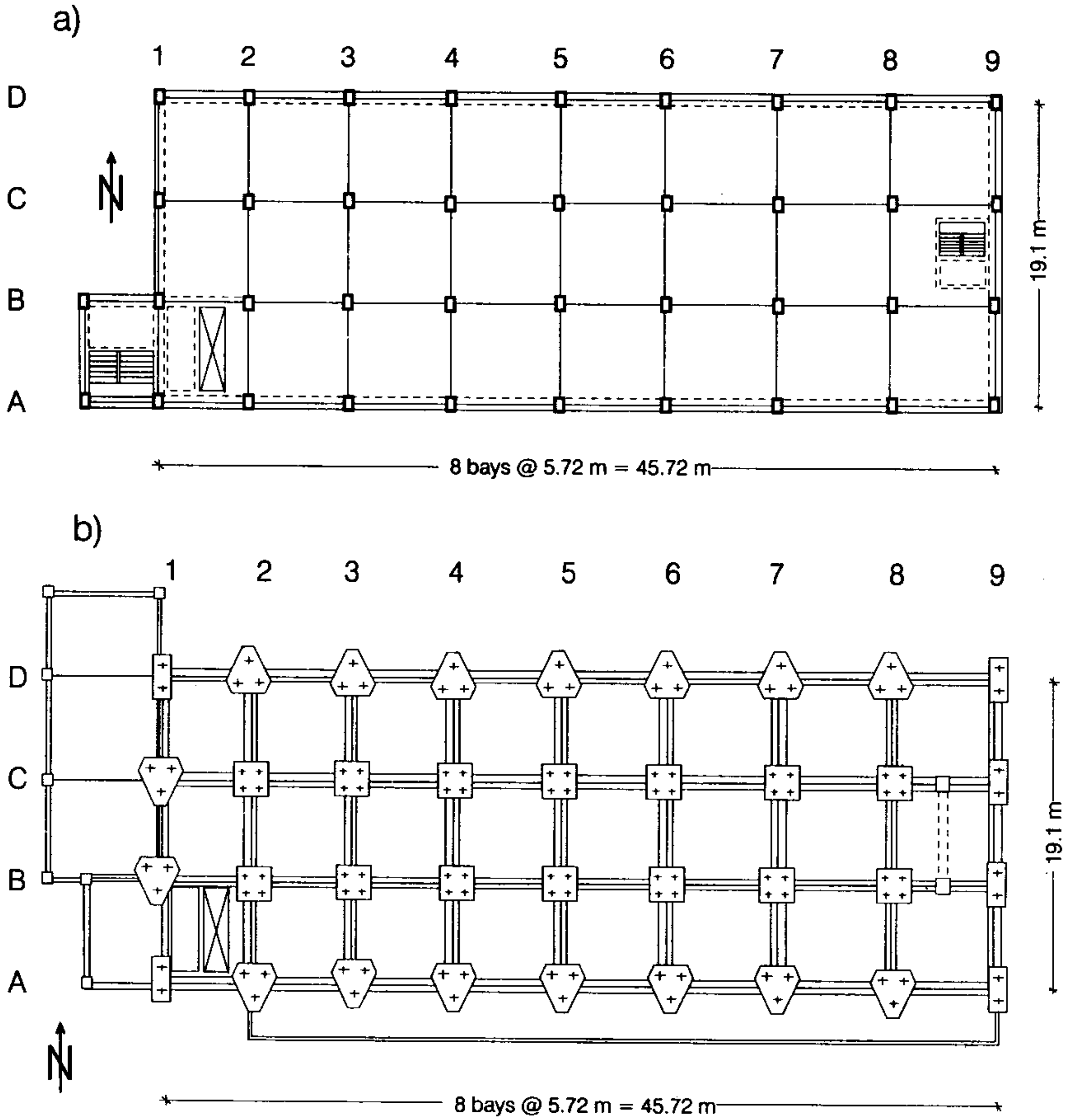


Fig. 3 VN7SH building: (a) typical floor plan (b) foundation plan (Contd.)

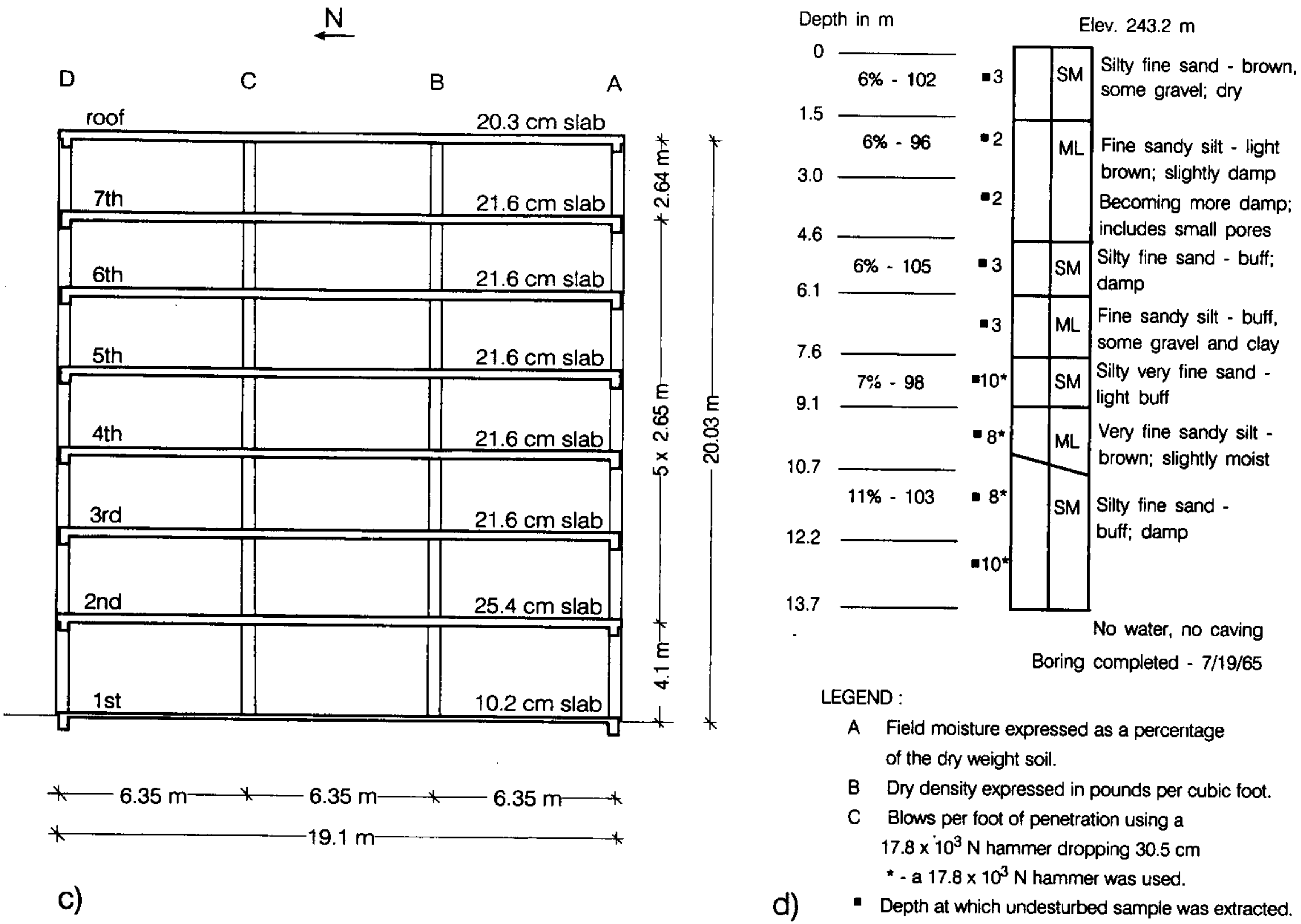


Fig. 3 (Contd.) (c) typical transverse section and (d) soil boring data from 7/17/1965

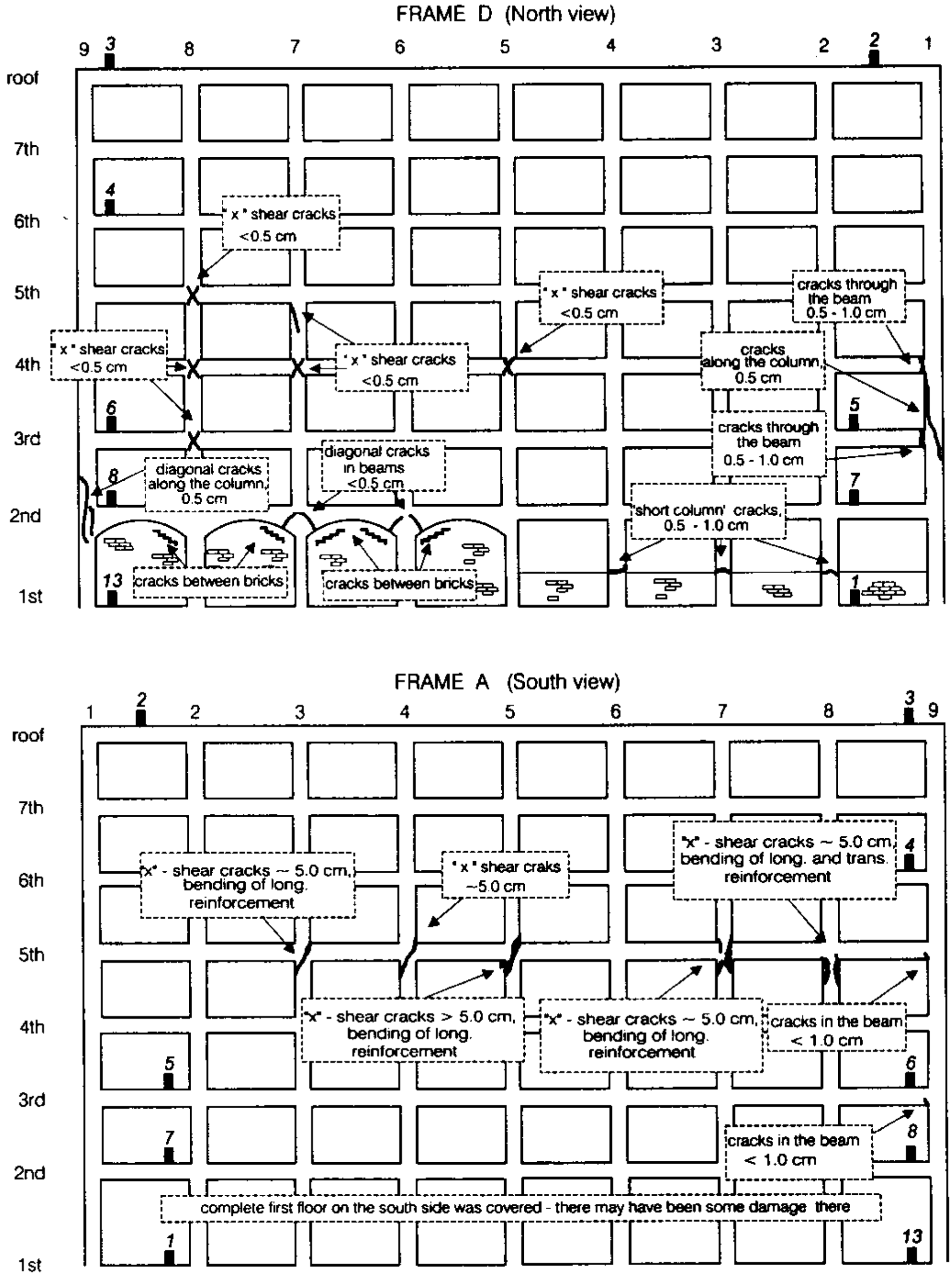


Fig. 4 Schematic representation of the damage: (top) frame D–North view and (bottom) frame A–south view (the sensor locations for channels 1–8 and 13 of strong motion recorders (oriented towards North) are also shown (see Trifunac et al., 2000 a, 2000b)

The $M_L = 6.4$ Northridge earthquake of 17 January 1994 (Figure 2) (Thio and Kanamori, 1996) severely damaged the building, and it was declared as unsafe by the Los Angeles housing authorities. The structural damage was extensive in the exterior north (D) and south (A) frames, designed to take most of the lateral load in the longitudinal direction. Severe shear cracks occurred at the middle columns of frame A, near the contact with the spandrel beam of the 5th floor (Figure 4).

Those cracks significantly decreased the axial, moment, and shear capacity of the columns. The shear cracks which appeared in the north (D) frame on the 3rd and 4th floors, and the damage of columns D2, D3 and D4 on the 1st floor caused minor to moderate changes in the capacity of these structural elements. No major damage of the interior longitudinal (B and C) frames was noticed. There was no visible damage in the slabs and around the foundation. The non-structural damage was significant. The recorded peak accelerations in the building were: 0.46g (L), 0.40g (T) and 0.28g (V) at the base, and 0.59g (L) and 0.58g (T) at the roof, along the longitudinal (L), transverse (T) and vertical (V) axes of symmetry (there were no sensors installed on the roof to measure vertical motions). The motions in the area surrounding the building (determined from smoothed contour maps) had relatively small horizontal transient peak accelerations, a, velocities, v, and displacements, d($a_R = -350$ cm/s^2, $a_T = 225$ cm/s^2, $a_V = 600$ cm/s^2, $v_R = -40$ cm/s, $v_T = 30$ cm/s, $v_V = -25$ cm/s, $d_R = -14$ cm, $d_T = 25$ cm, $d_V = -7$ cm; subscripts R, T and V refer to radial, transverse and vertical components, defined relative to point 34.27°N and 118.55°W in the "center" of the ruptured area (see Figure 2), and positive if away from the fault, clockwise and upward) (Trifunac et al., 1994; Todorovska and Trifunac, 1997a, 1997b). In the vicinity of the building, the peak strain factor was: horizontal ~$10^{-2.6}$ and vertical ~$10^{-3.2}$ (Trifunac et al., 1996; Trifunac and Todorovska, 1997b). The (refined) estimate of Modified Mercalli intensity at the site was VIII (Trifunac and Todorovska, 1997a, 1997b, 1999b).

Photographs and detailed description of the damage from the Northridge earthquake can be found in Trifunac et al. (1999b). Analysis of the relationship between the observed damage and the change in equivalent vertical shear wave velocity in the building can be found in Ivanović et al. (1999a). A discussion on the extent to which this damage has contributed to the changes in the apparent period of the soil-structure system can be found in Trifunac et al. (2001a, 2001b).

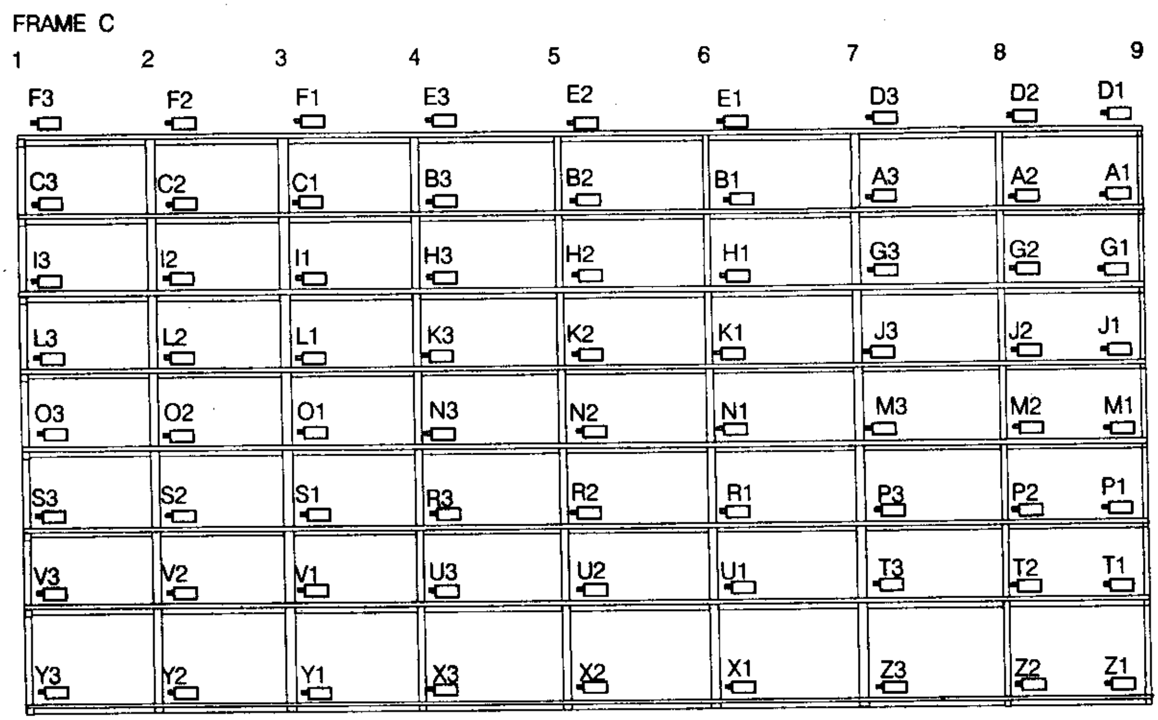


Fig. 5 Experiment I: position of the instruments along frame C

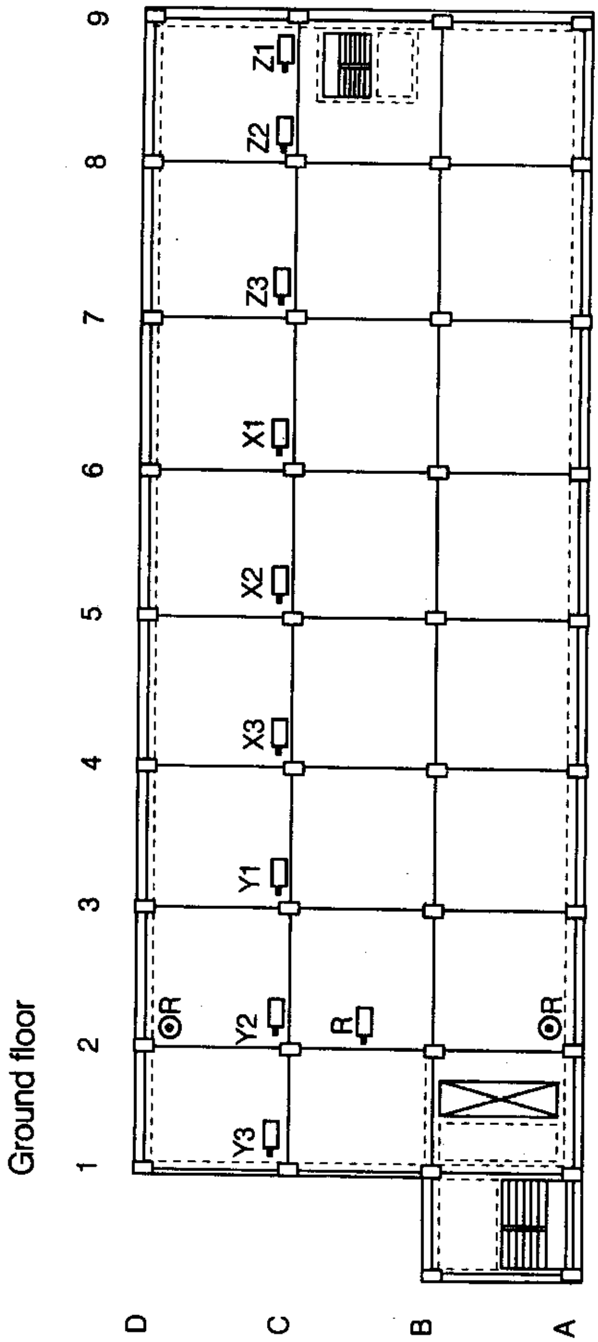


Fig. 6 Experiment I: position of instruments on the ground floor

2. Ambient Vibration Experiments

Two ambient vibration surveys were conducted, both while the building was damaged from the 1994 Northridge earthquake and its aftershocks, and was not in use. The first experiment took place on February 4–5, 1994, approximately two and a half weeks following the Northridge main event of 17 January. The second experiment was on April 19–20, 1994, three months following the Northridge main event, and one month following one of the larger aftershocks (20 March, 1994, $M = 5.2$). The damage observed at the time of each of the experiments was photographed and documented (Trifunac et al., 1999b).

A PC-based data acquisition system and six transducers were used: four Ranger SS-1 and two "old" Earth Sciences Ranger seismometers. This equipment is described in detail in Ivanović and Trifunac (1995). For both experiments, measurements were taken along longitudinal frame C, at all the nine columns, on each floor, and in three directions of motion: longitudinal (E-W), transverse (N-S) and vertical. Three of the Ranger SS-1 seismometers were used to measure the building response, and two of the "old" Earth Sciences Ranger seismometers and the fourth Ranger SS-1 seismometers were used to measure the motions at the reference sites on the ground floor. For each experiment, two calibration tests were conducted (one at the beginning and the other one at the end of the experiment), for two horizontal and for vertical position of the transducers. These tests consisted of placing all the six instruments close to each other, and simultaneously recording. The purpose of these tests was relative comparison of the recorded amplitudes, which differed due to differences in sensitivity and instrument constants. The duration of each of the recordings was about 3 min, and the sampling rate was set to 400 points per second.

2.1 Sensor Locations – Experiment I

Figure 5 shows the location of the measuring points along a longitudinal cross-section of the building. The measuring points were along longitudinal frame C, at all nine columns and on each floor. The order in which the measurements were carried out was from top to bottom and from east to west (A1, A2, A3, B1, B2, B3, ... up to Y1, Y2, Y3, as shown in Figure 5). We will refer to the location of a measuring point in the building by one letter code for the longitudinal frame and one digit code for the transverse frame to which the column closest to the transducer belongs.

Three reference points were used, all at the ground floor, marked by "R" in Figure 6. For the vertical motions, two of the "old" Earth Sciences Ranger seismometers were used (Ivanović and Trifunac, 1995). These were placed near columns A2 and D2 on the ground floor and were oriented always to measure vertical response. As reference instrument for all the horizontal recordings, a Ranger SS-1 seismometer was used, located at transverse frame 2, between longitudinal frames B and C, also on the ground floor, and oriented towards west or towards north.

The first calibration test (at the beginning of the experiment) was done on the S-W stairway, at the 5th floor, and the second one (at the end of the experiment) on the ground floor, between transverse frames 1 and 2 and longitudinal frames B and C. Both calibration tests gave consistent values of the sensitivity ratios (Ivanović and Trifunac, 1995).

The experiment was carried out continuously from the morning of 4 February until the morning of February 5, 1994. During this time, strong wind (about 50 km/h) was blowing intermittently. The temperature was in the range from 8° to 15° C. It was raining the night before, and the rain stopped at about 6:00 am on February 4. It was a week-day (Friday), and typical heavy traffic was moving along the San Diego freeway (I-405), 100–200 m to the west from the building site (see Figure 2). At the roof of the building, the air-conditioning equipment was working continuously. The elevators were not in use. There was no running water in the building, but electricity was on.

2.2 Sensor Locations – Experiment II

This experiment was carried out on Tuesday and Wednesday, April 19 and 20, 1994, three months after the January 17, Northridge, California earthquake, and one month after a strong aftershock with epicenter at 1 km from the building (March 20, 1994, $M = 5.2$, Figure 2). The building was restrained between the two experiments. Wooden braces were installed to increase the structural capacity near the areas of structural damage (Figure 7). Braces were placed in the first three or four stories at selected spans in the exterior longitudinal frames (A and D). Only the first floor of the interior longitudinal frames was restrained. We do not know when the addition of the braces was completed, and whether this preceded the aftershock on 20 March. However, we did observe that the width of the cracks, especially the shear cracks in the south (A) frame, became larger (relative to our first inspection on February 4). No new structural damage was noticed in the building or around its foundation. There were no braces added to the transverse frames. Figure 7 shows the location of structural damage and the braces as observed on April 19, 1994. The size of the "hinges" is roughly proportional to the level of damage.

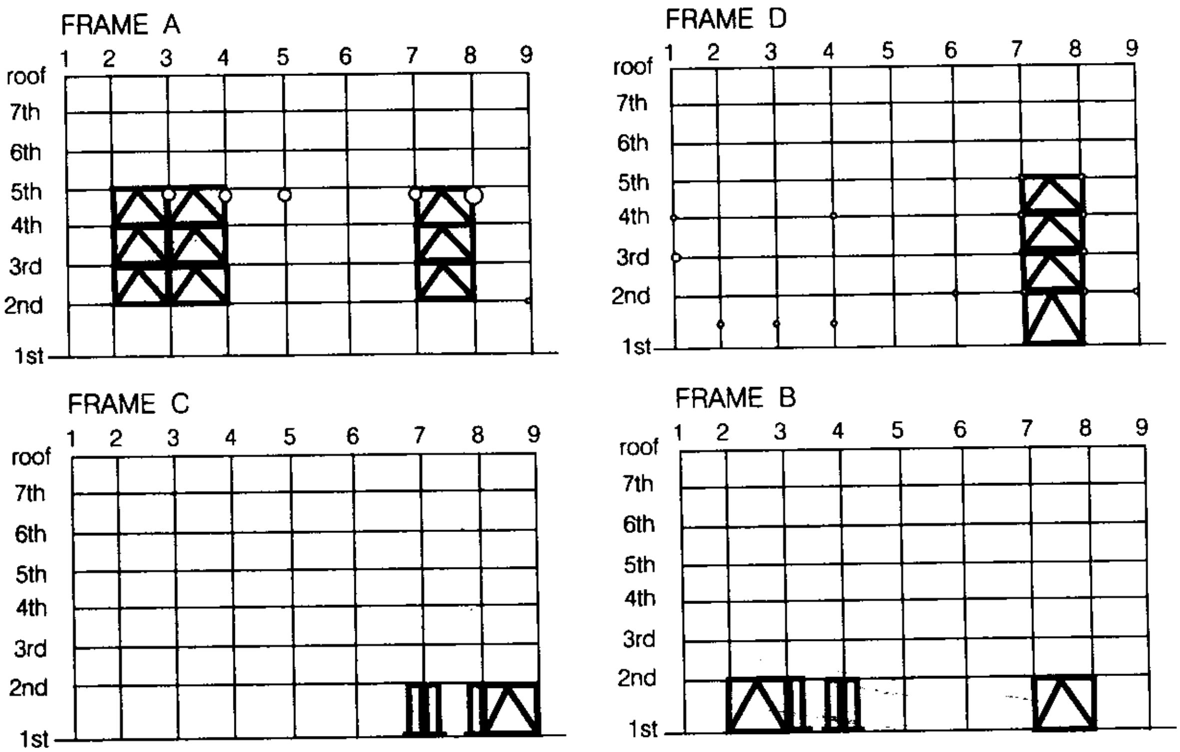


Fig. 7 Schematic representation of the structure and location of damage and of wooden braces, as seen at the time of Experiment II (19 April, 1994)

As in Experiment I, the order of the measurements was from top to bottom and from east to west. The motions on the ground floor were also measured in detail. The measuring points were at each column, and all three components of motion were recorded. At the 3rd, 5th and 7th floors, measurements in vertical direction were carried out along transverse frames 2, 5 and 8 (Figure 8).

The location of the three reference points on the ground floor is marked by "R" in Figure 9. The two Earth Sciences Ranger seismometers were placed at locations A5 and D5, and always recorded vertical motions (up). The reference point for horizontal motions was at the location B2 and was oriented along the longitudinal (E-W) or transverse (N-S) directions.

As part of this experiment, detailed measurements of ambient noise in the parking lot surrounding the building were also carried out. Motions were recorded in all three directions (north, east, and up) at 46 points surrounding the building, within 15 to 20 m around the structure. A detailed description of this part of the experiment and analysis of the recorded amplitudes and phases can be found in Trifunac et al. (1999a) and will not be repeated here.

The experiment was carried out continuously from noon of April 19 (Tuesday), until 9 p.m. on April 20 (Wednesday) 1994. Those were quiet, sunny days with temperature in the range from 12° to 25°C. The building was not in use, and except for electricity, facilities were not available (no elevators, air-conditioning, or running water).

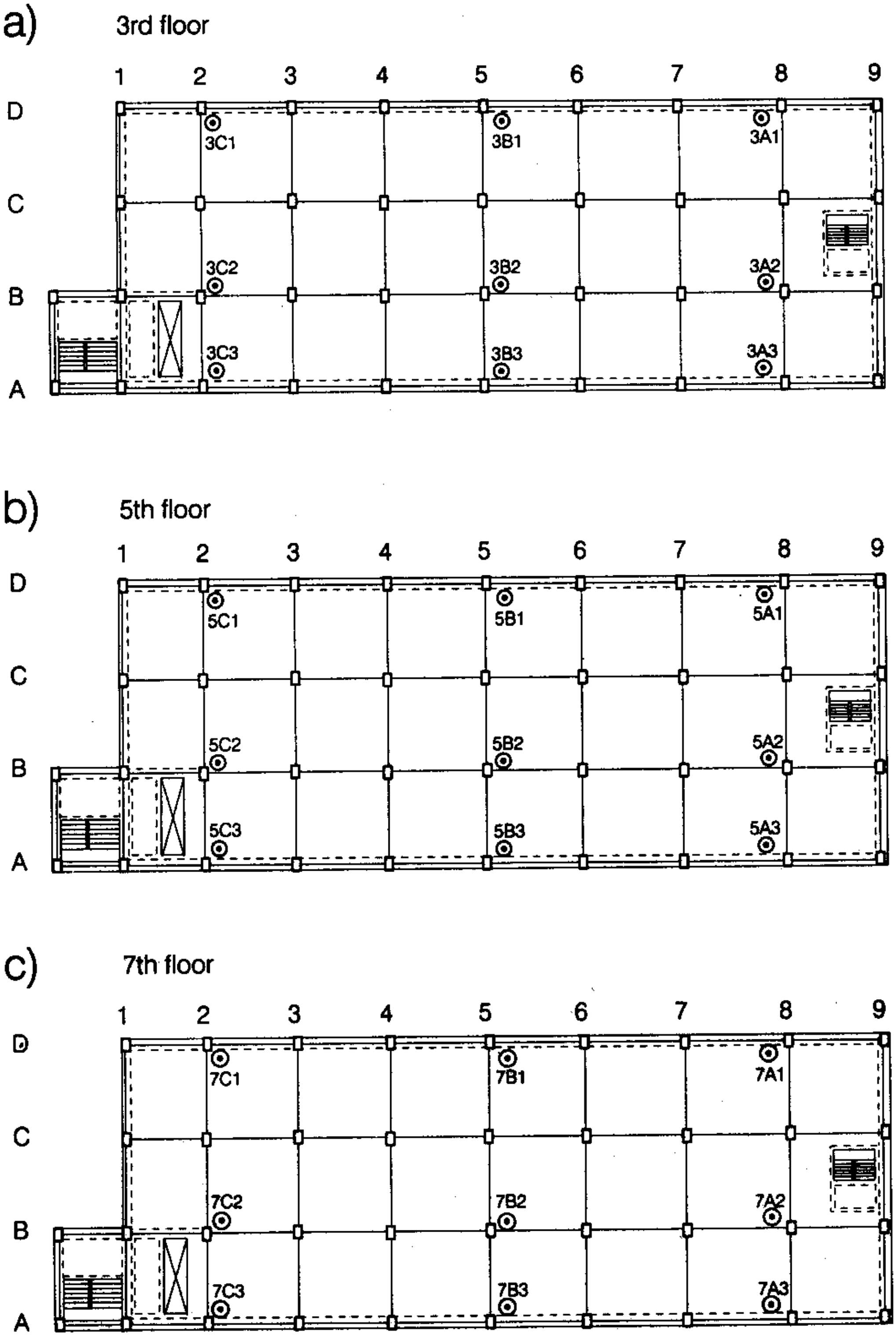


Fig. 8 Experiment II: position of instruments on (a) 3rd, (b) 5th and (c) 7th floors

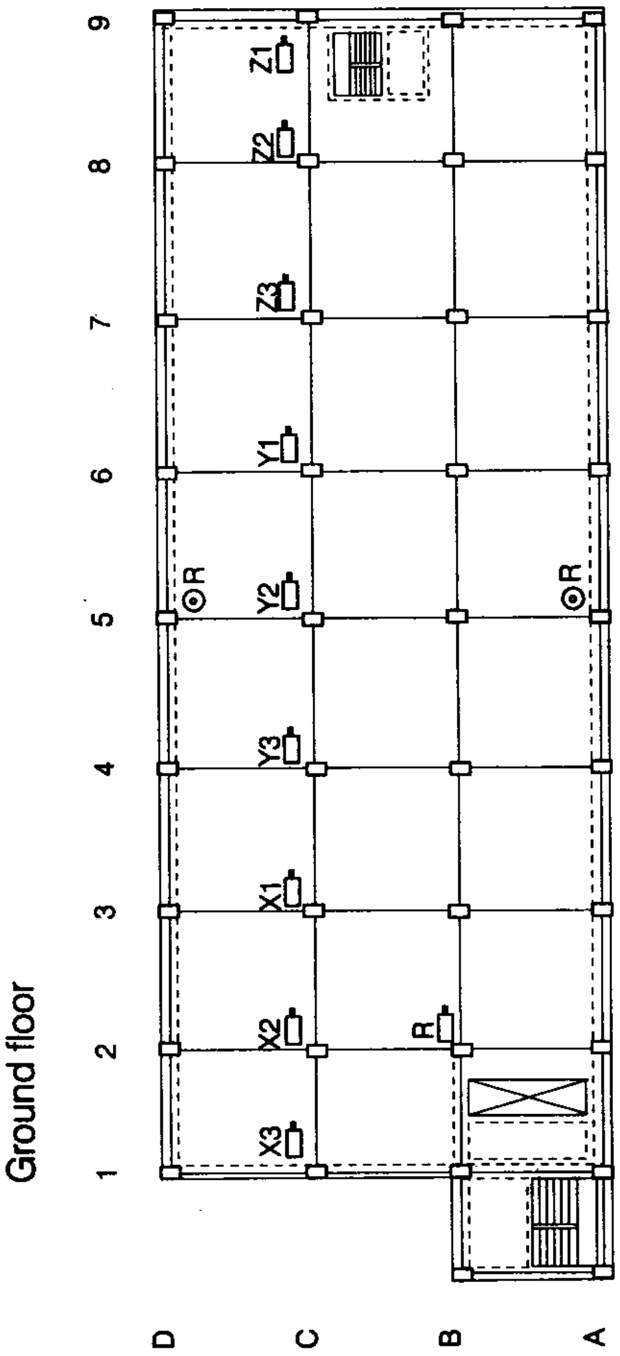


Fig. 9 Experiment II: position of instruments on the ground floor

2.3 Data Processing Methods

The data processing procedure described in Ivanović and Trifunac (1995) was followed. Briefly, describing first Fourier spectra were computed for each record by FFT (following amplitude correction from the calibration tests), and then the transfer-function amplitudes were computed for each measuring point and component of motion, with respect to the appropriate reference point on the ground floor. The Fourier amplitude spectra were smoothed before the division (with $R_s = 2$ for horizontal and $R_s = 5$ for vertical motions (Ivanović and Trifunac, 1995)). Figure 10 shows typical transfer-functions (longitudinal and transverse motions recorded at columns C5 and C9, on the 5th floor (see Figure 3).

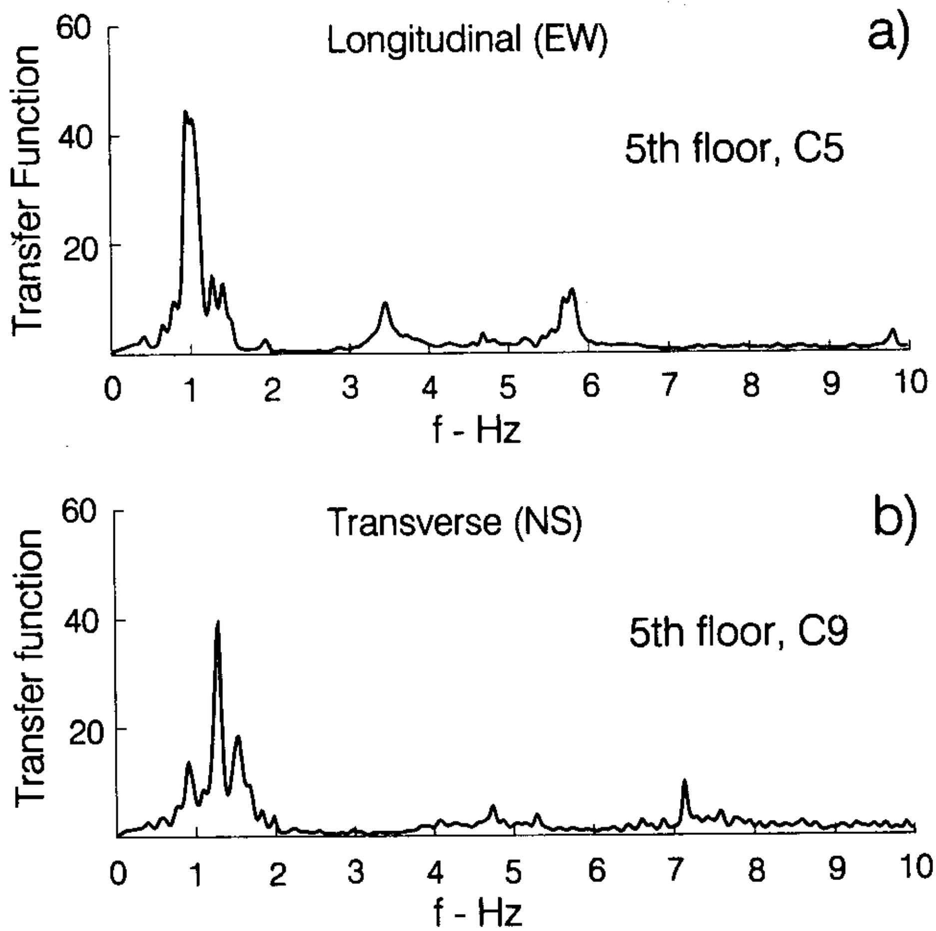


Fig. 10 Sample transfer-functions: (a) west motion recording at 5th floor, column C5, and (b) north motion recording at 5th floor, column C9 (Experiment I)

The transfer-function amplitudes were not smoothed. The phase angles for the transfer-functions were also computed. In drawing the apparent shape-functions, the phase angle was approximated by 0 or by π. To determine the shape functions, in some cases, phase lags were also computed for neighboring measurement points, via the cross-correlation function of band-pass filtered data (0.2 Hz band-width) centered at the modal frequencies (Ormsby filters were used). Cross-correlation functions were also used to check selected amplitudes of the response.

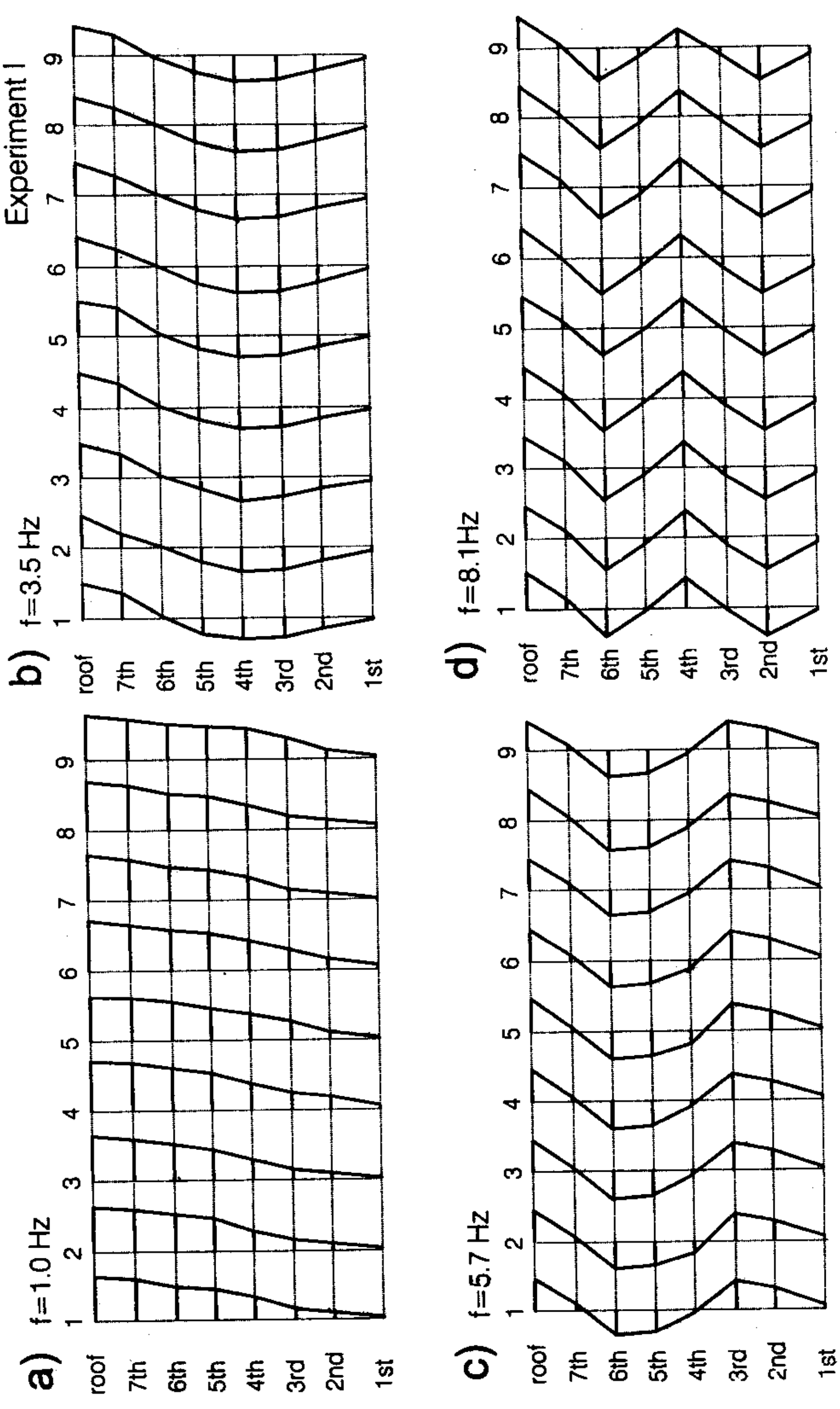


Fig. 11 Experiment I - longitudinal (EW) apparent modes of vibration: (a) first, f = 1.0 Hz (b) second, f = 3.5 Hz (c) third, f = 5.7 Hz and (d) fourth, f = 8.1 H

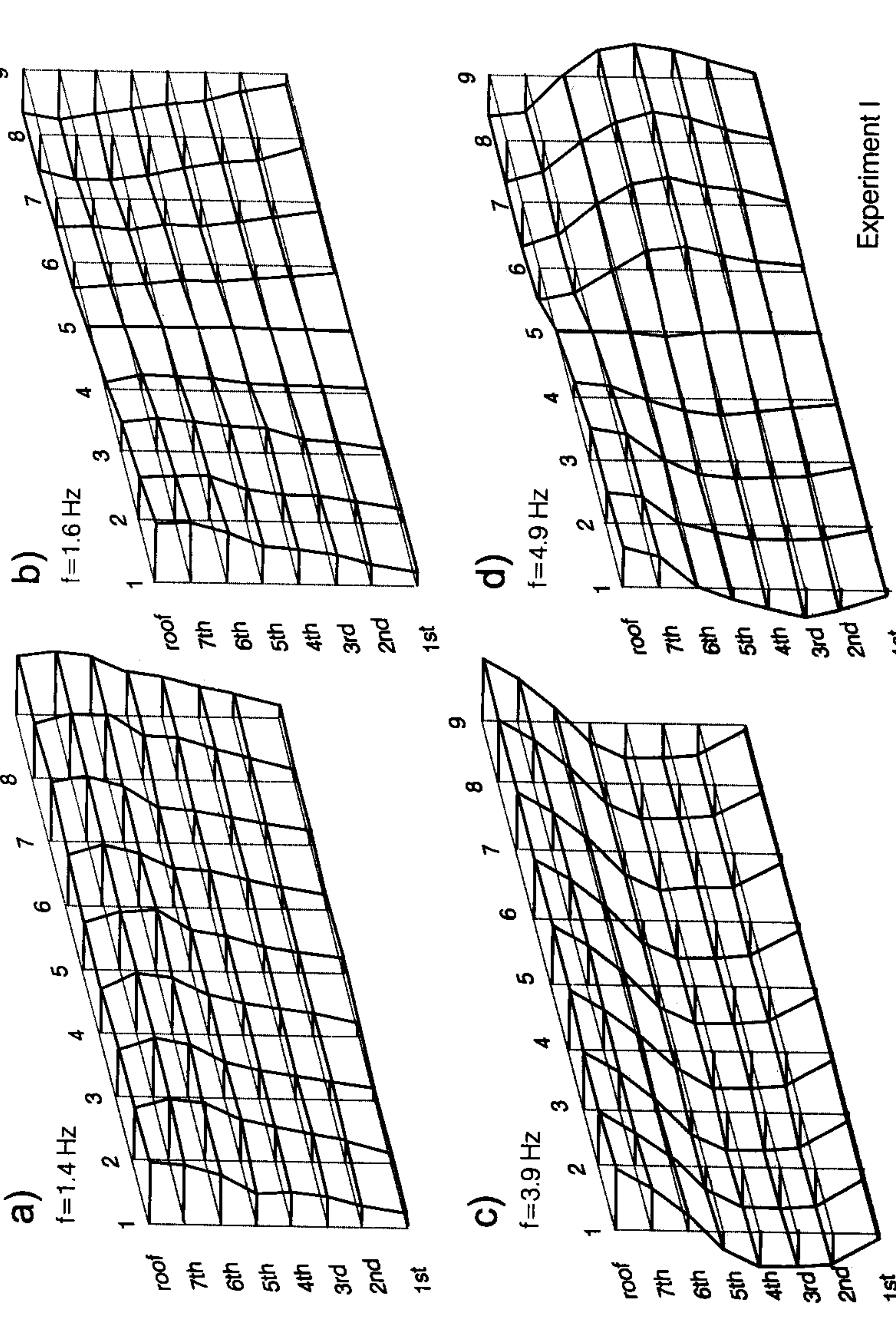
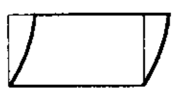
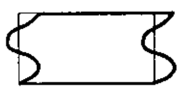
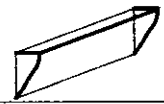
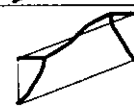
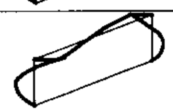


Fig. 12 Experiment I - transverse (NS) apparent modes of vibration: (a) first transverse, f = 1.4 Hz (b) first torsional, f = 1.6 Hz (c) second transverse, f = 3.9 Hz and (d) second torsional, f = 4.9 Hz

Table 1: Frequencies and Mode Shapes of the Identified Modes at Interior Longitudinal Frame C

Mode shapes EW	f - Hz		Δf - %
	Expl. I Feb. 94	Expl. II Apr. 94	
	1.0	1.1	10
	3.5	3.7	6
	5.7	5.7	0
	8.1	8.5	5

Mode shapes NS	f - Hz		Δf - %
	Expl. I Feb. 94	Expl. II Apr. 94	
	1.4	1.4	0
	1.6	1.6	0
	3.9	4.2	10
	4.9	4.9	0

2.4 System Frequencies and Mode-Shapes

Only system frequencies corresponding to the first four longitudinal and to the first four transverse modes of vibration could be identified from the transfer-functions of recorded horizontal motions (the signal-to-noise ratio was small for frequencies higher than those). No vertical modes of vibration could be identified from the transfer-functions of recorded vertical motions. Table 1 shows the frequencies of the identified modes (measured at interior longitudinal frame C) and sketches of the corresponding mode-shapes. Values for the frequencies are listed for each experiment. The percentage changes are given in the last column.

Figures 11 and 12 show the normalized mode-shapes respectively for longitudinal (E-W) and for transverse (N-S) vibrations, determined from the Experiment I data. In Figure 12, the first and third identified frequencies correspond to translational and the second and fourth, to torsional modes of vibration. The latter could also be identified from the records of longitudinal vibrations. Figure 12 however shows only the transverse components of the displacements of the torsional modes. The mode-shapes determined from the Experiment II data are similar to those in Figures 11 and 12 (Ivanović et al., 1999b).

2.5 Two-Dimensional Displacements along the Floor Slabs – Migration of Centers of Torsion

Figure 13 shows the modal displacements in the plane of each floor, determined from both the longitudinal and transverse vibrations. Parts (a) and (b) show respectively results from Experiments I and II. We recall that these measurements were taken along longitudinal frame C, and that most severely damaged were columns 7 and 8 of longitudinal frame A (south of frame C) and at the fifth floor (columns 3, 4 and 5 were also cracked). We also recall that braces were added to the damaged building before Experiment II (Figure 7).

It can be seen that the reinforced concrete floor slabs, 21.6 cm thick and stiff in their own plane, translate and rotate about vertical axes. While the transverse component of motion is dominant, the response in the longitudinal direction is also significant, especially for the top floors. It can be noticed that during Experiment I (Figure 13(a)), the transverse component of motion changes phase, but the longitudinal does not. Also, the amplitudes of the longitudinal displacements are not proportional to the transverse displacements, as it would be expected for a "clean" rotation (maximum displacements at the end columns for both directions of motion, and almost zero displacements at the centers of rotation). The longitudinal response of the middle columns (C4, C5 and C6) is clearly seen at each floor. This indicates coupling of the torsional response for this mode with the longitudinal response. The phase of the longitudinal response of the upper floors (roof, 7th and 6th) is opposite from the one at the lower floors (3rd and 4th), and it was difficult to define this for the 2nd and 5th floors.

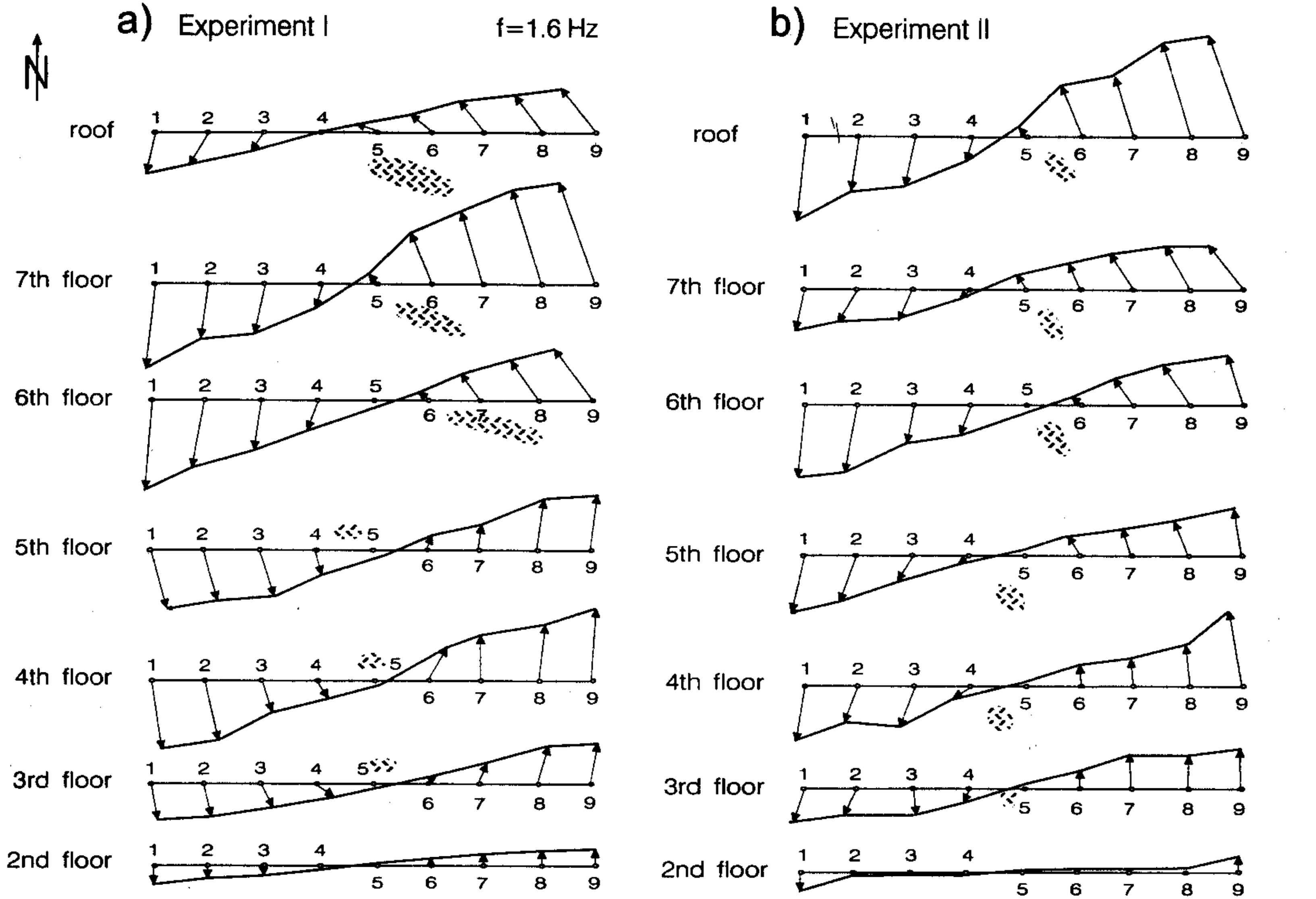


Fig. 13 Vector displacement amplitudes in the planes of the floor slabs (at 2nd through 7th floors and roof) at the frequency of the first torsional mode ($f = 1.6$ Hz): (a) Experiment I (b) Experiment II (the oval gray zones show approximate locations of the centers of rotation; notice in part (a) the jump in the position of the centers of rotation between 5th and 6th floors)

The shaded oval zones in Figure 13(a) illustrate the locations of the centers of rotation for the floor slabs, determined by drawing a normal to the displacement vectors. Due to measurement errors and some deformation of the floor slabs, the "center of rotation" for a floor slab is not a point but a zone. The "centers of rotation" are located south of frame C at the upper floors (above the 5th), and north of frame C at the lower floors (5th and below). At the lower floors, they are located close to the middle (column 5), and then they "jump" to the east part of the frame at the 6th floor. Between 6th floor and the roof, they move again towards the center of the frame. The jump from south to north is between the 5th and 6th floors, exactly where the most severe damage occurred (Figure 4).

The results of Experiment II (Figure 13(b)) show that, in contrast to Experiment I, the "centers of rotation" are all south of frame C, and are all near the center of the frame (near column line 5). This may be explained by the added braces (Figure 7) which have eliminated torsional eccentricities caused by the damaged columns at 5th floor and mainly along (south) frame A.

2.6 Two-Dimensional Displacements along Transverse Building Cross-Sections

No characteristic frequencies and mode-shapes were identified for vertical vibrations (strong vertical motion at $f = 0.5$ Hz was observed during Experiment I, but it was probably due to vibrations caused by the electrical equipment in the building (Ivanović et al., 1999b)).

As the building vibrates, most of the deformations occur in the columns. Consequently, the floor slabs move also in the vertical direction. The transverse and longitudinal modes could also be seen in the vertical response, especially at the upper floors. Figure 14 shows two-dimensional, in-plane motions of transverse frames 2, 5 and 8 at the frequency of the first transverse mode (part (a), $f = 1.4$ Hz), and at the frequency of the first torsional mode (part (b), $f = 1.6$ Hz). The vertical displacements have been exaggerated by a factor of two to emphasize the deformation of the columns. A noticeable vertical displacement is seen only at column A5 of the 5th floor, and only at the frequency of the first transverse mode (Figure 12(a)). This column experienced large shear cracks (Figure 4) during the Northridge earthquake, and the large vertical displacement in Figure 14 indicates decreased axial capacity of this column. No large vertical displacement is noticeable of column A5 at the 7th floor, presumably because of participation of the neighboring frames and slabs. The vertical displacements of transverse frames 2 and 8 were small, as it would be expected, because the columns in these frames suffered less (frame 8) or no (frame 2) damage (Figure 4). No unusual vertical displacements of the transverse frames 2, 5 and 8 could be seen at the first torsional frequency ($f = 1.6$ Hz, Figure 14, part (b)) (Ivanović et al., 1999b).

2.7 Summary of the Ambient Vibration Studies

Both ambient vibration surveys took place after the building was damaged from the 17 January 1994 Northridge earthquake and its aftershocks and before it was repaired. The structural damage was extensive in the exterior longitudinal frames (A and D, Figure 4). The building was declared unsafe and was not in use at the time of the experiments. Experiment I was conducted approximately two and a half weeks after the earthquake. Experiment II was conducted three months after the earthquake, and one month after the 20 March aftershock $(M = 5.2)$. Between the experiments, the building was restrained temporarily by wooden braces (Figure 7).

Changes in the modal frequencies were expected because of differences in the state of the structure and of the underlying soil (addition of wooden braces, and possible additional damage from the $M = 5.2$ aftershock with epicenter at distance of only about 1.2 km). Table 1 shows that three out of the four identified frequencies for longitudinal vibrations increased (the first one by 10 percent, and the second and fourth 6 and 5 percents), due to the addition of wooden braces, placed at the longitudinal frames (Figure 7). The frequency of the third longitudinal mode was not affected. Table 1 (right) shows that the frequencies of the first transverse and first torsional modes did not change due to the addition of braces, but the frequency of the second purely transverse mode got increased by 10 percent.

The building has symmetric geometry and approximately uniform distribution of mass and stiffness. It has thick and heavy slabs, which are stiff in their own planes. Spandrel beams connect the outside columns, and most of the lateral loads are carried by the exterior frames. According to these characteristics, its structural system cannot be described as a "weak beam-strong column" system. In spite of its apparent symmetry, the structure experienced strong torsional response (Trifunac et al., 1999a).

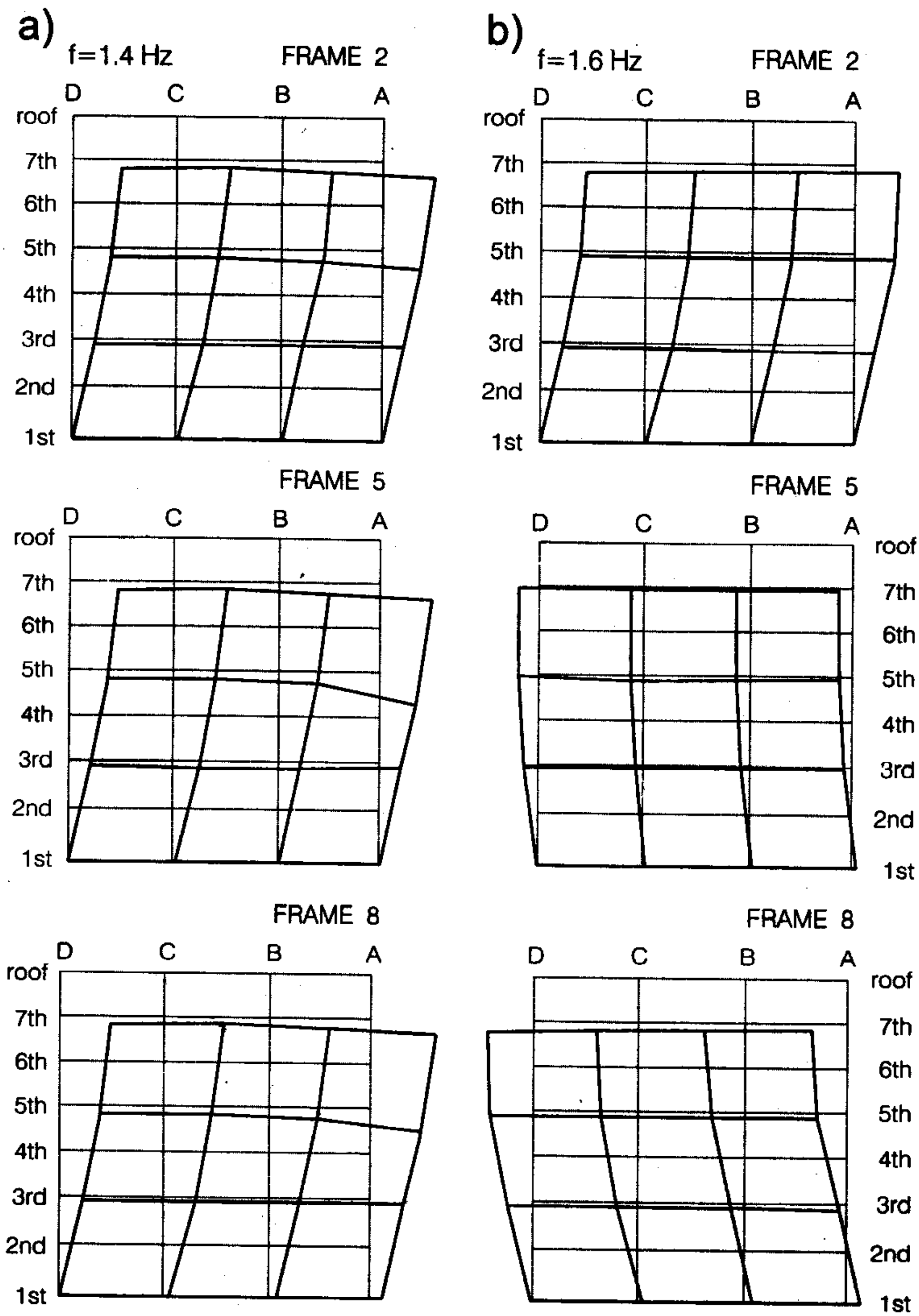


Fig. 14 Experiment II - in-plane displacements of transverse frames 2, 5 and 8: (a) at the frequency of the first transverse mode (f = 1.4 Hz) and (b) at the frequency of the first torsional mode (f = 1.6 Hz)

The transfer-function of the recorded vertical motions had peaks at the frequencies of the longitudinal and transverse modes. The vertical responses at these frequencies, analyzed alone or combined with the corresponding horizontal responses, may be useful for identification of the damage of the columns. Changes in the amplitudes of the vertical responses (large vertical amplitude of a column implied loss of axial capacity) were noticed near one of the damaged columns where ambient noise was measured (column A5 at the 5th floor). The identification of the damaged structural members would have been more complete, had the vertical motions been recorded at more points throughout the structure.

The purpose of the tests was to find out whether highly localized damage in structural members could be detected by ambient vibration surveys. The results so far show that this cannot be done with the common procedures. Analyses of complex building behavior (with interaction of many structural components) require much higher spatial resolution of recording. To detect severe and localized column damage (e.g., about 0.5 m long shear and compression failure), ambient noise should be recorded at comparable separation distances. Performing such detailed experiments may not be feasible in most practical situations. Although loss of axial capacity could be seen in the vertical response of the damaged columns, moderate or weak damage of this kind will not be noticed in most ambient response surveys. More detailed surveys of ambient vibrations should be developed for damage detection, when the loss of strength is moderate and when the damage is less obvious.

3. Summary of Other Studies of This Building

Ambient vibration surveys are a useful tool to be used along with other types of data and methods of analysis. This building is an interesting case study because three types of independent data are available: earthquake response data over the period of 25 years, earthquake damage data for two earthquakes, and ambient vibration data recorded in the damaged building as well as in the ground around the building (Mulhern and Maley, 1973; Trifunac et al., 1999a). In the following, we briefly summarize our previous and other current work related to this building, and in the next section we present our general conclusions and recommendations for future earthquake and ambient noise recording and analyses.

3.1 Nonlinear Soil-Structure System Identification

The building was first instrumented by the U.S. Geological Survey, and then by the California Division of Mines and Geology (CDMG). We gathered or processed records of the following earthquakes in this building, listed in chronological order (*R* is the epicentral distance) (Trifunac et al., 1999a):

Earthquake	Date	M_L	R [km]
San Fernando	02/09/1971	6.6	22
Whittier Narrows	10/01/1987	5.9	41
Whittier-Narrows Aftershock	10/04/1987	5.3	38
Pasadena	10/03/1988	4.9	32
Montebello	06/12/1989	4.1	34
Malibu	01/19/1989	5.0	36
Sierra Madre	06/28/1991	5.8	44
Landers	06/28/1992	7.5	186
Big Bear	06/28/1992	6.5	149
Northridge	01/17/1994	6.4	1.5
Northridge Aftershock	03/20/1994	5.2	1.2
Northridge Aftershock	12/06/1994	4.5	10.8

As far as we know, the first strong motion recording in this building was of the 1968 Borrego Mountain earthquake, but the processed data is not available at present. The 1971 San Fernando earthquake was recorded by three self-contained AR-240 (analogue) triaxial accelerographs (Hudson, 1970b), and for other events – by a 13 channel CR-1 recording system (analogue) and one SMA-1 (analogue) triaxial accelerograph (Trifunac and Hudson, 1970). The largest recorded response was of the 1994 event (Trifunac et al., 1999a, 1999b).

In Trifunac et al. (2000a, 2000b) we present Fourier and time-frequency analyses of the earthquake response data. Unfortunately, there were neither free-field recordings nor two or more vertical base recordings to separate the foundation rocking and translation due to soil-structure interaction from the recorded (total) response. The rocking part of response was separated, only roughly, by low-pass filtering, thanks to the fact that the fixed-base frequency of this building appears to be much higher than the foundation rocking frequency (Trifunac et al., 2001a, 2001b). The ambient vibration analysis was particularly useful in the interpretation of non-linear soil-structure interaction using the earthquake response data.

Fourier Analysis: The transfer-functions between the floor responses and the base response were calculated (Trifunac et al., 2001a). For the events for which the building response was large (San Fernando, Northridge, Landers and Whittier-Narrows), these transfer-functions had a broad peak at frequencies much smaller than the fundamental fixed-base frequency, calculated from the structural characteristics or estimated from the ambient vibration tests (> 1.4 Hz for the NS direction and > 1.0 Hz for the EW direction). This indicated that the "loss of stiffness" could not be explained by non-linearity of the structure alone and its degradation due to damage. We concluded that the non-linearity of the soil response contributed mainly to the non-linear behavior of the overall system.

Time-Frequency Analysis: Non-linear effects in the response of soil-structure system depend on the level of the excitation and also on the initial state of the system (e.g., the state of the soil, such as degree of consolidation, water content etc.). The building damage changes the system, but the soil can "heal" itself and recover the original stiffness by settlement with time and dynamic compaction from shaking during smaller events. Time-frequency analysis of all the earthquake records by two independent methods (short-time Fourier transform and zero-crossing analyses) showed that the system frequency changed, from earthquake to earthquake and during a particular earthquake (Trifunac et al., 2001b). Figures 15 and 16 show the instantaneous system frequency f_p on the abscissa and the peaks of the "rocking" accelerations $\ddot{\theta}_x(t)$ and $\ddot{\theta}_y(t)$ (band-pass filtered by Ormsby filters between 0.1–0.2 Hz to 0.8–1.0 Hz). These figures show progressive reduction of f_p with increasing amplitude of response, but the reduction is not permanent. It is seen that during both ambient vibration tests, f_p of the transverse (NS) response is near 1.4 Hz and close to the value for the smallest earthquake motions (Montebello, 1989 earthquake, Figure 16). This suggests that the soil-foundation-structure stiffness was "regenerated" by the weak shaking of the Northridge aftershocks. For the longitudinal (EW) response, ambient vibration surveys indicate only partial recovery, from 0.4–0.6 Hz during Northridge to $f_p = 1 - 1.1$ Hz (note that during the aftershock on March 20, 1994, $f_p \sim 1.3 - 1.4$ Hz, Figure 15). The shift in f_p from 1.0 Hz (4–5 February, 1994) to 1.1 Hz (19–20 April 1994) was interpreted to have resulted in part from increase in the "building stiffness" associated with temporary wooden braces, primarily along frames A and D (Figure 7). Consistent with this interpretation is the fact that, for Experiment II, the peaks of the transfer-functions were smaller by 30% than for Experiment I, suggesting stiffer overall system at the time of Experiment II. Figures 15 and 16 also suggest that the soil-pile foundation system during strong shaking behaves as a non-linear system with gap elements, which open and close during strong motion and may be closed again by aftershock excitations.

3.2 Wave Propagation in the Building

In Ivanović et al. (1999a), we present an analysis of wave propagation in the building from the recorded acceleration earthquake response. By cross-correlation and cross-spectrum analyses, we obtained estimates of the velocity of waves propagating vertically along the columns (~100 m/s) and horizontally along the floor slabs (~2000 m/s). A comparison of results for different earthquakes showed a reduction of velocity in the areas of severely damaged columns by the Northridge earthquake (Figure 4).

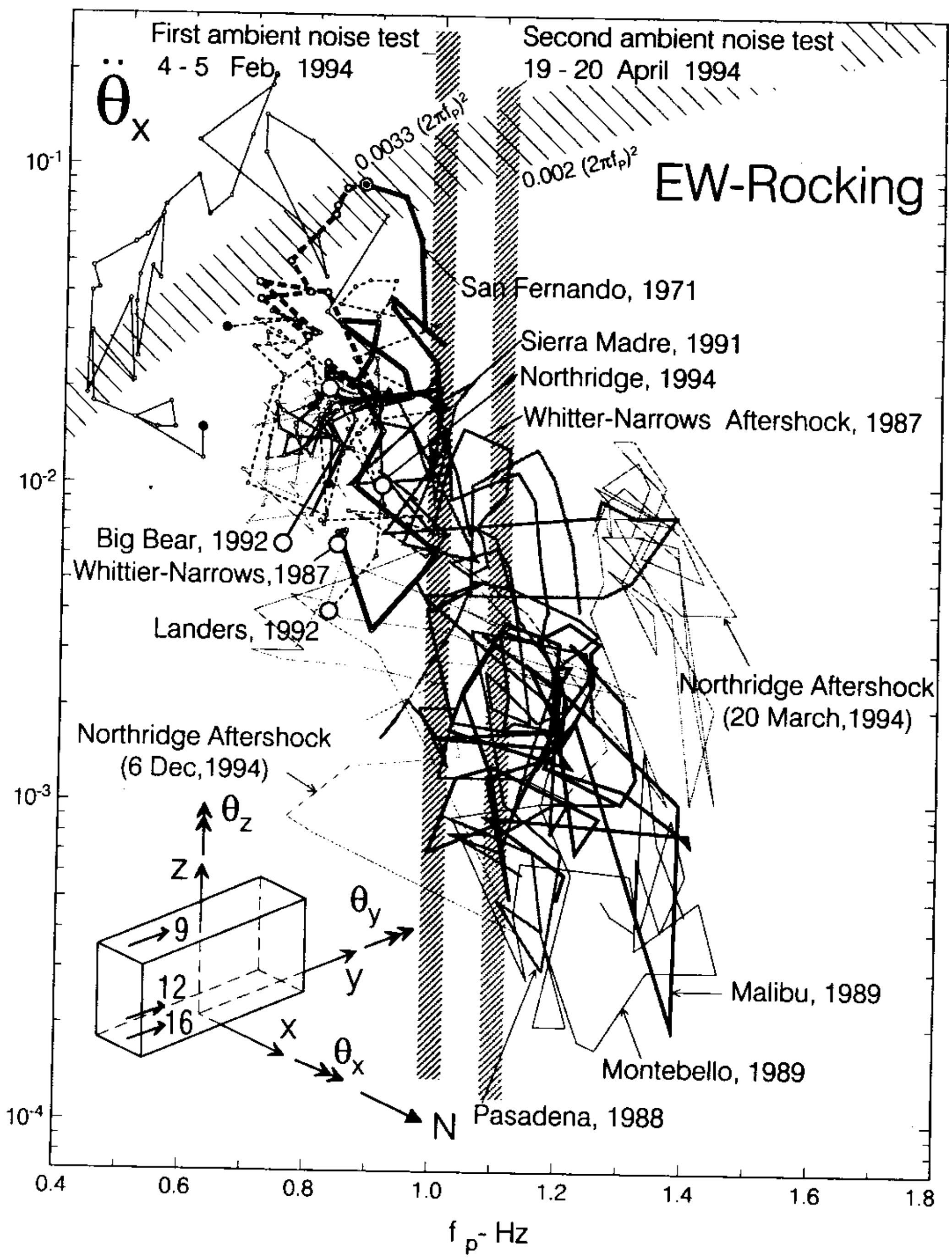


Fig. 15 Peak amplitudes of $\ddot{\theta}_x$ (EW rocking acceleration) versus f_p (apparent frequency of the soil-foundation-structure system) during twelve earthquakes (see Fig. 2). (the gray vertical lines show the apparent EW frequencies of the system response as determined from Experiments I and II)

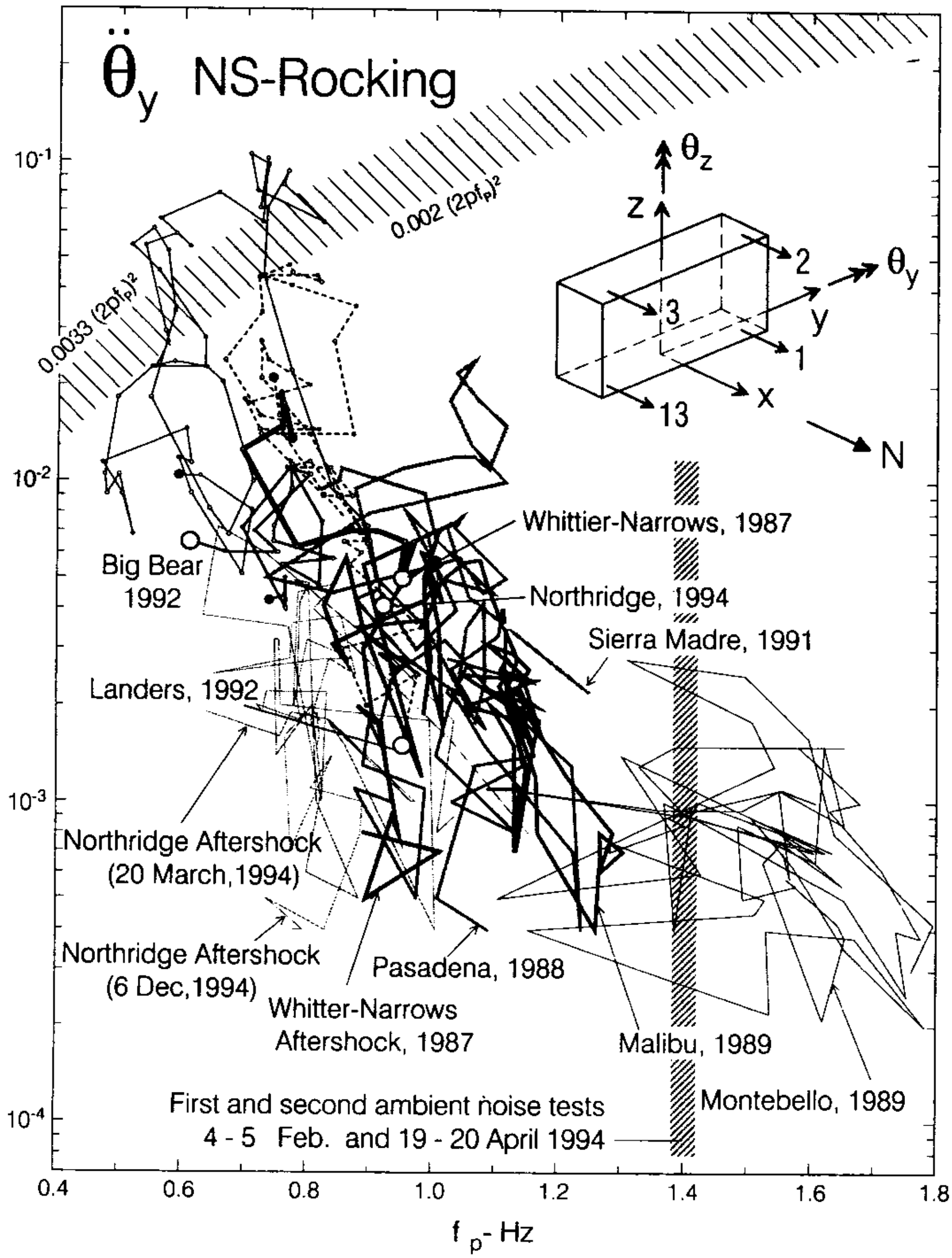


Fig. 16 Peak amplitudes of $\ddot{\theta}_y$ (NS rocking acceleration) versus f_p (apparent frequency of the soil-foundation-structure system) during twelve earthquakes (see Fig. 2) (the gray vertical line shows the apparent EW frequency of the system response as determined from Experiments I and II)

3.3 Microtremor Motions of the Ground Surrounding the Building

At the time of Experiment II, we also recorded motions at a grid of points in the parking lot surrounding the building, and at the reference sites located on the ground floor inside the building. The purpose was to detect some characteristic ground deformation associated with soil-structure interaction for at least the fundamental transverse $(f = 1.4 \text{ Hz})$ and longitudinal $(f = 1.1 \text{ Hz})$ modes of vibration (as in our study of Millikan Library (Foutch et al., 1975)). We could not see any peaks in the Fourier spectra of the motion around the building that could be associated with the building rocking or translation, because (as the analysis of the recorded data showed) the foundation of this building was "flexible". We did find evidence of wave propagation and of deformation of the building foundation during the passage of microtremor waves. The overall pattern of the computed time delays implied that the microtremor waves arrived primarily from the west, and then got scattered from and diffracted around the building foundation. That computed time delays corresponded to horizontal phase velocity of about 300 m/s, is consistent with the interpretation that microtremors are high frequency Rayleigh waves propagating through shallow soil layers. The contours of time delays for vertical motion also implied wave arrival from west and southwest, with apparent phase velocities between 250 and 300 m/s. The results of that analysis are presented in Trifunac et al. (1999a).

4. Conclusions and Recommendations Drawn from the Studies of the VN7SH

The studies of the response of this building to ambient noise and to earthquake shaking show the value of full-scale experiments in describing dynamic characteristics of real, three-dimensional structures. The results of these studies can be used to guide future implementations of recording systems in buildings (number of channels and their location in the buildings), so that future building recordings provide more valuable and complete information on the structural performance during earthquakes.

The existing strong motion instrumentation in the VN7SH building was not sufficient to estimate the rocking response associated with soil-structure interaction, which significantly affected the response of the building. Additional recorders, placed at the opposite diagonal corners of each floor, would have provided valuable information to measure the rocking response (longitudinal and transverse) and the foundation warping during the passage of seismic waves. Free-field recordings near the building would have, as well, provided valuable information for analysis of the building-foundation response.

The following simple and useful "standard" practice is recommended for instrumentation of future buildings. First, a three-dimensional ambient vibration test of the building should be performed, similar, but not necessarily as detailed as the test of the VN7SH building. Based on the three-dimensional deformations and mode shapes determined from these tests, a knowledgeable expert or a committee, with expertise in full-scale testing of structures, should select the optimum number, location and orientation of acceleration sensors. When necessary (after a moderate or a large earthquake) additional ambient vibration tests can be performed, creating updated "finger-prints" of the structural system. Such "finger-prints" will document the state of the structure preceding an earthquake and the subsequent changes. The quality of the recorded data will enable numerous new studies for better understanding of the earthquake response of actual structures, and can be used for validation of analytical and numerical model procedures for forward evaluation of structural response.

SUMMARY AND CONCLUSIONS

This paper presented a literature review on the subject of ambient vibration testing, and a review of a recent study by the authors, to illustrate the state-of-the-art in application of the ambient vibration method. The literature review showed that reports on testing of full-scale structures by the ambient vibration method began to appear regularly around 1970, with about 3/4 of all contributions devoted to the experiments in buildings, dams, chimneys and silos, and about 1/4 to bridges. Since 1985, there are only about 3 papers per year world-wide dealing with this subject. The reviewed study of the seven storey hotel building showed that detailed ambient vibration tests of healthy and damaged structures recordings can provide valuable information about the state of the building, that can help interpret past and future earthquake response recordings of the building. Detailed ambient vibration testing is recommended before strong motion instrumentation, in order to optimize the information to be obtained from the available number of sensors.

The ambient vibration tests measure the response of structures to microtremor and microseism waves, which are 4 to 6 orders of magnitude smaller than the destructive strong earthquake motions (Trifunac and Todorovska, 2000). Because the soil-structure system frequencies in the example illustrated changed by factors that can be as large as 4, within the range of strong motion amplitudes covering two orders of magnitude (see Figures 15 and 16 and Trifunac et al., 2001a, 2001b), the results of ambient vibration tests can provide estimates of the soil-structure system frequencies for the smallest level of excitation. This is very useful for design purposes, since it will help select "appropriate design system frequencies" (Trifunac, 1999) for different scenarios of dynamic response analysis or for probabilistic estimates of response (Todorovska, 1995).

Ambient vibration measurements can and should be conducted at a large and dense set of points, and should thus provide detailed data on the spatial properties of the soil-structure system. In contrast, measurements at one or two points, via transfer-function analysis of a system, can be indeterminate, lacking resolution, or be non-unique. This is because one must assume a model to be able to interpret the data (Trifunac and Todorovska, 1999a). With a high density of measuring points, the ambient vibration tests provide strong spatial constraints that would result in more realistic and representative models. Too many structural health monitoring and structural control studies are arbitrary and therefore useless because of poor selection of model, model parameters and boundary conditions.

Finally, the ambient vibration tests are "complete" full-scale experiments. Even the carefully planned laboratory experiments will represent only those aspects of the problem that the experiment designer had chosen to study and had incorporated into the model. The best and most complete laboratory tests can verify and quantify only those aspects of the problem that the investigator knows. Except when fortunate accidents occur, we do not know how to model what we are not aware of and what we do not understand. The full-scale ambient vibration tests present a completely different situation that cannot be controlled easily. The as-built environment contains all the properties of reality. We only have to find ingenious ways to discover, record and interpret this reality. In this respect, we hope that the above review and description of one of our full-scale studies will show that dense measurements, coupled with elementary analysis, will produce useful and occasionally new results.

We chose the example of the VN7SH building not to focus on innovative methods of analysis but on the aspects of detailed spatial data gathering. Developments of new methods of analysis and system representation are valuable, but better understanding of the intricate nature of the real world (as-built environment) can result only from detailed observations with an abundant number of data points. In the past, the profession has emphasized the need for a high dynamic range and ever increasing resolution of dynamic measurements. It is now time that we recognize the need to apply and extend the same concept also to the space coordinates.

Detection of damage can be based on visual inspection, on localized measurements, or on both. In the case of the VN7SH building, there was no need to perform measurements to detect the damage – it was obvious to the naked eye. We chose to use this building as a test case to determine the capabilities of various experimental damage detection procedures. We found that typical ambient vibration surveys and common analyses cannot detect the location or the extent of damage in buildings similar to the VN7SH building.

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