UNIVERSITY OF SOUTHERN CALIFORNIA
Department of Civil Engineering

INSTRUMENTED 7-STOREY REINFORCED CONCRETE
BUILDING IN VAN NUYS, CALIFORNIA:
AMBIENT VIBRATION SURVEYS FOLLOWING THE DAMAGE
FROM THE 1994 NORTHRIDGE EARTHQUAKE

by

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ABSTRACT

This report describes two ambient vibration tests of an instrumented seven-storey reinforced concrete (RC) building in Van Nuys, California. This building is interesting to study because it was severely damaged by the Northridge earthquake of 17 January, 1994 ($M_L=6.4$, $R=1.5$ km) and its aftershocks, and because its response to the Northridge earthquake as well as to other earthquakes (since 1971) has been recorded by the permanent strong motion instrumentation. The ambient noise was recorded by an array of sensitive portable instruments and a PC (personal computer) based data acquisition system. The first ambient noise test was conducted on February 4 and 5, 1994, about 20 days after the building was damaged by the Northridge earthquake and its early aftershocks. The second test was conducted on April 19 and 20, 1994, about one month after one of the strongest aftershocks, of March 20, 1994 ($M_L=5.3$, $R=1.5$ km), which may have further damaged the building. The purpose of the tests was to find whether the existence and location of damage could be detected from the ambient noise tests using established procedures, and to develop new procedures for damage detection in structures using the detailed ambient noise data from these experiments. Detailed surveys of the damage were also conducted at the time of both tests, and have been published elsewhere.

Ambient noise was measured along the longitudinal, transverse and vertical directions. Three-dimensional mode shapes of vibration were constructed, and the apparent frequencies for the first several modes were determined. Measurements were made at every column of every floor. In spite of this detail, the attempts to detect the highly localized damage by simple spectral analyses were not successful. It was concluded that, to identify localized column and beam damage, very high spatial resolution of recording points is required, because of interactions between the numerous structural components and because of the complex building behavior. It was also concluded that loss of axial capacity of severely damaged columns can be detected from the vertical response of the columns, but moderate or weak damage of this kind would not be noticed in most ambient response surveys.

A comparison of the results for weak and strong motions suggests that soil-structure interaction plays an important role in the seismic response of this building. Analyses of its seismic response to twelve earthquakes (published elsewhere) suggest that, during strong motion response, the foundation and piles push the surrounding soil sideways and a gap is created between the soil and foundation, which appears to be closing after shaking from aftershocks (probably due to “dynamic compaction” of the soil). The apparent frequencies of the soil-foundation-structure system appear to be influenced significantly by the variations in the effective soil-foundation stiffness. These variations can be monitored by a sequence of specialized ambient vibration tests.
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1. INTRODUCTION

1.1 General Introduction

The mathematical models used in dynamic analyses of structures are idealizations required to represent the response of real structures to various dynamic loads (e.g. strong earthquake shaking, strong winds, explosions etc.). They can be verified by conducting experiments on full-scale structures, e.g. ambient and forced vibration [Trifunac and Todorovska, 1999]. Both of these can be used to identify the structural characteristics, e.g. the frequencies of vibration, damping ratios and mode shapes. While these experiments are to a great degree repeatable, measurements of the response to strong earthquake shaking are not (large earthquakes occurring anywhere close to the structure are rare events compared to its service time).

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 guide researchers in developing time and amplitude dependent structural models and analysis algorithms, to be used in structural health monitoring and in structural control studies. Therefore, further development of experimental methods for in situ measurement of full-scale partially damaged structures is of great interest [Ivanović et al., 1999; Todorovska et al., 1999; Trifunac et al., 2000a,b]. An advantage of the ambient vibration over the forced vibration surveys is that they usually require light equipment and smaller number of operators. The sources of excitation are wind, microtremors, microseisms, and various local random and periodic sources (e.g. traffic or heavy machinery).

The forced vibration tests may require large forces to produce useful (larger) response amplitudes of full-scale structures. The force (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. However, the paths of waves propagating through the structure are different from those in case of ground shaking or wind excitation, and cautious interpretation of the results is required to take such differences into account [Luco et al., 1975; 1986; 1987; 1988].

In the U.S., ambient and forced vibration tests of structures have been conducted for about 60 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]. 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,b] used wind and microtremor induced vibrations to test a twenty-two and a thirty-nine storey steel frame
building. 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 storey steel frame building, a thirty-nine storey steel frame building, a nine-storey steel frame building and a nine-storey 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 [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 nonlinearity (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 Yuzugullu, 1966].

1.2 Objectives and Organization of this Report

This report describes two detailed ambient vibration surveys of an instrumented seven-storey RC building in Van Nuys, California. This building is interesting to study because it was severely damaged by the Northridge earthquake of 17 January, 1994 ($M_L=6.4$, $R=1.5$ km) and its aftershocks, and because its response to the Northridge earthquake, as well as to other earthquakes (since 1971), was recorded by permanent strong motion instrumentation. The purpose of the surveys was to find whether the damage could be detected using established analysis procedures, and to search for new procedures for damage detection in structures using the detailed ambient noise data from these experiments. Detailed survey of the damage was also conducted at the time these two tests took place, and has been published elsewhere [Trifunac et al., 1999a].

Chapter 2 describes the building, Chapter 3 describes the two ambient vibration surveys, and Chapter 4 presents the results. Chapter 5 reviews other work by the authors related to this building and draws general conclusions.
2. THE BUILDING

2.1 Design Characteristics and Site Geology

The seven-storey reinforced concrete hotel building analyzed in this report is located in the city of Van Nuys in the North-West part of the Los Angeles metropolitan area. Figure 2.1 shows the location of the building relative to the fault planes of the 1971 San Fernando and 1994 Northridge earthquakes [Trifunac, 1974; Wald and Heaton, 1996] which both damaged the building. It was designed in 1965, constructed in 1966 [Blume et al., 1973; Freeman and Honda 1973], and served as a hotel at the time of the 1994 Northridge earthquake. Figure 2.2 shows a plan view of a typical floor (a), a plan view of the foundation layout (b), a side view of the building frame (c), and a typical soil-boring log data at the building site (d). The building is 62 × 150 feet in plan (1 foot = 30.48 cm). The typical framing consists of columns spaced at 20-foot centers in the transverse direction and 19 foot 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. With the exception of some light framing members supporting the stairway and elevator openings, the structure is essentially symmetric. Except for two small areas at the ground floor, covered by one-storey canopies, the plan configurations of each floor, including the roof, are the same. The floor system is reinforced concrete flat slab, 10 inches thick at the second floor, 8.5 inches thick at the third to seventh floors and 8 inches thick at the roof. The north side of the building, along column line D (Fig. 2.2a), has four bays of brick masonry walls located between the ground and the second floor at the East end of the structure. Nominal 1 inch expansion joints separate the walls from the underside of the second floor spandrel beams. Although none of the wall elements described are designed as a part of the lateral force-resisting system, they do contribute in varying degrees to the stiffness of the structure.

The building is situated on recent alluvium. The soil-boring log in Fig 2.2d shows that the underlying soil consists primarily of fine sandy silts and silty fine sands. The foundation system (Fig. 2.2b) consists of 38 inch-deep pile caps, supported by groups of two to four poured-in-place 24-inch-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 40 feet long and has design capacity of over 100 kips vertical load and up to 20 kips lateral load (1 kip = 4,448.2 N). The structure is constructed of regular weight reinforced concrete [Blume et al., 1973].
Fig. 2.1 Central San Fernando Valley, California, and the site of VN7SH. The horizontal projections of the faults of the 1971 San Fernando and 1994 Northridge earthquakes are also shown. The duration of strong motion from these two earthquakes was respectively 9 and 6 s. For seven other earthquakes, the directions and distances to are shown by arrows. The epicenters of two Northridge aftershocks are shown by solid stars.
Fig. 2.2  a) Typical floor framing plan.  b) Foundation plan.
Figure 2.2   c) Typical transverse section.    b) Typical soil boring log.
Fig. 2.3  Schematic representation of damage (top) frame D (North view); (bottom) frame A (south view). The strong motion sensor locations for channels 1-8 and 13 (oriented towards north), are also shown [Ivanović et al. 1999].
2.2 Earthquake Damage

The $M_L = 6.6$ San Fernando earthquake of February 9, 1971 (Fig. 2.1) [Trifunac, 1974] caused minor structural damage. 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 nonstructural damage, however, was extensive and about 80 percent of all repair cost was used on fixing the drywall partitions, bathroom tiles and on plumbing fixtures.

The $M_L = 6.4$ Northridge earthquake of 17 January, 1994 (Fig. 2.1) [Wald and Heaton, 1996] severely damaged the building, and it was declared as unsafe and red-tagged 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 (Fig. 2.3). 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 nonstructural damage was significant. Almost every guestroom suffered considerable damage. Severe cracks were noticed in the masonry brick walls, and in the exterior cement plaster.

Photographs and detailed description of the damage from the Northridge earthquake can be found in Trifunac et al. [1999a], and an 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. [1999]. 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. [2000a,b].
3. AMBIENT VIBRATION SURVEYS

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 structural damage was extensive in the exterior longitudinal North (D) and South (A) frames (Fig. 2.2a,c), and the changes of stiffness in the damaged areas implied changes in the structural system. The first experiment took place on 4–5 February, 1994, approximately two and a half weeks following the Northridge main event of 17 January. The second experiment was on 19–20 April, 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., 1999a].

3.1 Measuring Equipment

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.

3.2 Experiment I: Instrument Layout and Other Details

Figure 3.2.1 shows a schematic representation of the locations of structural damage as observed at the time of this experiment (4–5 February, 1994). The size of the “hinges” in this figure is proportional to the level of damage. A detailed pictorial documentation of this damage can be found in the report by Trifunac et al. [1999a].

Figure 3.2.2 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
Figure 3.2.1 Schematic representation of the structure and locations of damage observed at the time of Experiment I (Feb. 4, 1994).
Figure 3.2.2 Position of the instruments during the first experiment, at frame C.
columns, and on each floor. We will refer in this report to the location of a measuring point in the building by the one letter code for the longitudinal frame and the one digit code for the transverse frame to which the column closest to the transducer belongs. 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 Fig. 3.2.2). It was impossible to position instruments near column C9 on all the floors because the bathtubs were located next to this column, and all measurements related to location C9 were actually recorded about 0.7 m to the West. Unfortunately, the instrument at column C4 on the roof was not working while E-W motion was recorded. This omission was discovered during the analysis of the recorded data, too late to repeat the measurement.

Three reference points were used, all at the ground floor, marked by “R” in Fig. 3.2.3. 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 PC based data acquisition system was located on the 5th floor. The transducers were placed either directly on the concrete, or on the ceramic tiles (carpets and other floor covers were removed during measurement). The motion was recorded for about 3 min, at sampling rate 400 points per second.

The experiment was carried out continuously from the morning of 4 February until the morning of 5 February, 1994. During this time, strong wind (about 50 km/hour) 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 Fig. 2.1). 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.
Figure 3.2.3 Position of the instruments during the first experiment, at the ground floor.
3.3 Experiment II: Instrument Layout and Other Details

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, $M = 5.2$; Fig. 2.1). The building was restrained between the two experiments. Wooden braces were installed to increase the structural capacity near the areas of structural damage. 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 exactly 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 3.3.1 shows the location of structural damage and the braces as observed on April 19, 1994. As in Fig. 3.2.1, the size of the “hinges” is proportional to the level of damage.

Figure 3.3.2 shows the location of the measuring points along a vertical cross-section of the building (along longitudinal frame C). As in Experiment I, the order of the measurements was from top to bottom and from East to West (A1, A2, A3; B1, B2, B3; … Y1, Y2, Y3), and the measurements at column 9 were made at a point about 0.7 m West from the ideal location. 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 (see Figs 3.3.3−3.3.5).

The location of the three reference points on the ground floor is marked by “R” in Fig. 3.3.6. 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 meters around the structure. Fig. 3.3.7 shows the locations of these points. A detailed description of this part of the experiment and analysis of the recorded amplitudes and phases can be found in Trifunac et al. [1999b] and will not be repeated here.
Figure 3.3.1 Schematic representation of structure and location of damage of VN7SH, as seen on April 19, 1994.
Figure 3.3.2 Positions of the instruments during the second experiment at frame C.
Figure 3.3.3 Positions of the instruments during the second experiment, at third floor.
Figure 3.3.4 Positions of the instruments during the second experiment, at fifth floor.
Figure 3.3.5 Positions of the instruments during the second experiment, at the seventh floor.
Figure 3.3.6 Positions of the instruments during the second experiment at the ground floor.
Figure 3.3.7  Positions of the instruments during the recording of the response of the Parking lot of VN7SH, top view, second experiment.
The first calibration test (at the beginning of the experiment) was performed on the 7th floor, at the North-West corner of the structure, near column D1. The second test (at the end of the experiment) was performed on the ground floor, between longitudinal frames B and C and transverse frames 1 and 2.

The PC based data acquisition system was located on the ground floor. The transducers were also placed either directly on the concrete, ceramic tiles, or onto the asphalt for the outside measurements. Motion was recorded for about 3 min, at sampling rate 400 pts/sec.

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…).
4. RESULTS FROM TWO AMBIENT VIBRATION SURVEYS

This chapter presents the frequencies of vibration and mode shapes for the E-W, N-S and vertical motions determined from the ambient noise data recorded during Experiments I and II. The motions were recorded for about 3 min, at 400 points per second sampling rate.

4.1 Data Processing

The data processing procedure described in Ivanović and Trifunac [1995] was followed. Briefly, 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]). The transfer-function amplitudes themselves 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 $\pi$.

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. Cross-correlation functions were also used to check selected amplitudes of the response. This method was particularly useful for analyses of the recordings at points C1 and C9. In addition to the previously identified natural frequencies, the Fourier amplitude spectra of these recordings displayed additional peaks, which we believe were due to electrical motors operating on the western end of the roof. These peaks were close to the peaks associated with the natural frequencies of the structure, and caused difficulties in the computation and interpretation of the transfer-functions.

4.2 Results from Experiment I

4.2.1 System Frequencies and Mode-Shapes for Horizontal Motions

Figures 4.2.1 and 4.2.2 show typical transfer-functions. The motions are in the longitudinal (E-W) and transverse (N-S) directions, at columns C5 and C9, on the 5th floor (see Fig. 2.2a).

The identified first four system frequencies for the longitudinal (E-W) vibrations are: $f = 1.0, 3.5, 5.7$ and $8.1$ Hz. The corresponding mode-shapes are shown in Figs 4.2.3–4.2.6, and the normalized modal amplitudes are tabulated in Tables 4.2.1–4.2.4. The
Figure 4.2.1 Transfer function for longitudinal (West) recording at 5th floor, column C5.
Figure 4.2.2 Transfer function for transverse (North) recording at 5th floor, column C9.
Figure 4.2.3  First longitudinal mode, $f=1.0$ Hz, first experiment.
Figure 4.2.4  Second longitudinal mode, $f=3.5$ Hz, first experiment.
Figure 4.2.5 Third longitudinal mode, $f=5.7$ Hz, first experiment.
Figure 4.2.6 Fourth longitudinal mode, $f=8.1$ Hz, first experiment.
Table 4.2.1  Normalized amplitudes of EW mode of vibration: $f = 1.0$ Hz, Experiment I.

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Table 4.2.2  Normalized amplitudes of EW mode of vibration: $f = 3.5$ Hz, Experiment I.

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<td>0.80</td>
<td>–</td>
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<td>0.80</td>
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<td>0.80</td>
</tr>
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<td>0.53</td>
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<td>0.53</td>
<td>0.53</td>
<td>0.53</td>
<td>0.53</td>
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</tr>
<tr>
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<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
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<td>0.00</td>
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</tr>
<tr>
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<td>-0.73</td>
<td>-0.73</td>
<td>-0.73</td>
<td>-0.73</td>
<td>-0.73</td>
<td>-0.73</td>
<td>-0.73</td>
<td>-0.73</td>
</tr>
<tr>
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<td>-0.93</td>
<td>-0.93</td>
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<td>-0.93</td>
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<tr>
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</table>

Table 4.2.3  Normalized amplitudes of EW mode of vibration: $f = 5.7$ Hz, Experiment I.

<table>
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</thead>
<tbody>
<tr>
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<td>1.00</td>
<td>1.00</td>
<td>0.88</td>
<td>–</td>
<td>1.00</td>
<td>1.00</td>
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<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>7th floor</td>
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<td>0.12</td>
<td>0.12</td>
<td>0.12</td>
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<td>6th floor</td>
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<td>-0.88</td>
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</tr>
<tr>
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<td>-0.75</td>
<td>-0.75</td>
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<td>-0.75</td>
<td>-0.75</td>
<td>-0.75</td>
<td>-0.75</td>
<td>-0.75</td>
</tr>
<tr>
<td>4th floor</td>
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<td>-0.12</td>
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<tr>
<td>3rd floor</td>
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<td>1.00</td>
<td>1.00</td>
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Table 4.2.4 Normalized amplitudes of EW mode of vibration: $f = 8.1$ Hz, Experiment I.

<table>
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<th>C6</th>
<th>C7</th>
<th>C8</th>
<th>C9</th>
</tr>
</thead>
<tbody>
<tr>
<td>roof</td>
<td>0.75</td>
<td>0.75</td>
<td>0.75</td>
<td>–</td>
<td>0.75</td>
<td>0.75</td>
<td>0.75</td>
<td>0.75</td>
<td>0.75</td>
</tr>
<tr>
<td>7th floor</td>
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<td>0.38</td>
<td>0.25</td>
<td>0.25</td>
<td>0.25</td>
<td>0.25</td>
<td>0.25</td>
<td>0.25</td>
<td>0.25</td>
</tr>
<tr>
<td>6th floor</td>
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<td>-1.00</td>
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<td>-0.88</td>
<td>-0.88</td>
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<td>-0.88</td>
<td>-0.88</td>
</tr>
<tr>
<td>5th floor</td>
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<td>-0.25</td>
<td>-0.25</td>
<td>-0.25</td>
<td>-0.25</td>
<td>-0.25</td>
<td>-0.25</td>
<td>-0.25</td>
</tr>
<tr>
<td>4th floor</td>
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<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
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</tr>
<tr>
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<td>-0.25</td>
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</tr>
<tr>
<td>2nd floor</td>
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<td>-1.00</td>
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</tr>
<tr>
<td>1st floor</td>
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<td>-0.25</td>
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<td>-0.25</td>
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</tr>
</tbody>
</table>

1 On this floor, there were difficulties in defining the phase.

Normalization is such that the maximum amplitude is unity. The signals corresponding to the first three mode-shapes were very clear in all E-W recordings. For the fourth mode ($f = 8.1$ Hz), difficulties were encountered in defining the phase of the motion, because of small signal-to-noise ratio.

The identified first four system frequencies for the transverse (N-S) vibrations are: $f = 1.4$, 1.6, 3.9 and 4.9 Hz. The corresponding mode-shapes are plotted in Figs. 4.2.7–4.2.10, and the normalized modal amplitudes are tabulated in Tables 4.2.5 through 4.2.8.

Table 4.2.5 Normalized amplitudes of NS mode of vibration: $f = 1.4$ Hz, Experiment I.

<table>
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<th>C2</th>
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<th>C7</th>
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<th>C9</th>
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<tbody>
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<td>0.80</td>
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</tr>
<tr>
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<td>1.00</td>
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<td>0.88</td>
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</tr>
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</tr>
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<td>0.80</td>
<td>0.88</td>
<td>0.88</td>
</tr>
<tr>
<td>4th floor</td>
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<td>0.50</td>
<td>0.50</td>
<td>0.62</td>
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<td>0.38</td>
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<td>0.38</td>
<td>0.38</td>
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<td>0.60</td>
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<td>0.28</td>
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<td>0.25</td>
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<tr>
<td>1st floor</td>
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<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
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<td>0.02</td>
<td>0.02</td>
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</table>

31
Table 4.2.6  Normalized amplitudes of NS and EW response: \( f = 1.6 \) Hz, Experiment I.

<table>
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<td>NS</td>
<td>EW</td>
<td>NS</td>
<td>EW</td>
<td>NS</td>
<td>EW</td>
<td>NS</td>
<td>EW</td>
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<td>0.30</td>
<td>0.046</td>
<td>0.23</td>
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<td>0.00</td>
<td>0.077</td>
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<tr>
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<td>0.070</td>
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<td>0.061</td>
<td>0.46</td>
<td>0.070</td>
<td>0.20</td>
<td>0.077</td>
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<tr>
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<td>0.70</td>
<td>0.046</td>
<td>0.57</td>
<td>0.061</td>
<td>0.46</td>
<td>0.090</td>
<td>0.25</td>
<td>0.077</td>
</tr>
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<td>-0.092</td>
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<td>0.046</td>
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<td>0.12</td>
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<td>-0.50</td>
</tr>
<tr>
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<td>0.12</td>
<td>-0.046</td>
<td>0.09</td>
<td>-0.061</td>
<td>0.12</td>
<td>-0.046</td>
<td>0.12</td>
<td>-0.07</td>
</tr>
<tr>
<td>2nd floor</td>
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<td>-0.015</td>
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<td>0.09</td>
<td>-0.015</td>
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<td>0.000</td>
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<tr>
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<td>0.01</td>
<td>-0.015</td>
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1. There were difficulties in determining the phase
2. There were difficulties in determining the amplitude
Table 4.2.7  Normalized amplitudes of NS mode of vibration: $f = 3.9$ Hz, Experiment I.

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<th>C7</th>
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<tbody>
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<td>0.78</td>
<td>0.67</td>
<td>0.78</td>
<td>0.67</td>
<td>0.78</td>
<td>0.78</td>
<td>0.89</td>
</tr>
<tr>
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<td>0.44</td>
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<td>0.55</td>
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<td>0.44</td>
<td>0.44</td>
</tr>
<tr>
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<td>0.33</td>
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</tr>
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<td>4th floor</td>
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</tr>
<tr>
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<td>-0.78</td>
<td>-0.89</td>
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<td>-0.78</td>
<td>-0.78</td>
<td>-0.78</td>
<td>-0.78</td>
</tr>
<tr>
<td>2nd floor</td>
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</tr>
<tr>
<td>1st floor</td>
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<td>-0.11</td>
<td>-0.11</td>
<td>-0.11</td>
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<td>-0.11</td>
<td>-0.11</td>
<td>-0.11</td>
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</tr>
</tbody>
</table>

The computed transfer functions for the ground floor are very similar to the transfer-functions for the calibration tests. All are almost constant, with amplitudes close to unity. The modal amplitudes were normalized so that the largest amplitude for the modal response is unity. Therefore, the amplitude at the ground floor of the normalized modal responses varies for different modes. It is “small” for the lower frequency modes, which are more excited and associated with larger structural response relative to the ground, and it is “large” for the less excited higher frequency modes.

Because of the high stiffness of the slabs (connecting the frames) in their own planes, the measured response of longitudinal frame “C” can describe the response of all the longitudinal frames, including damaged frames A and D, and for both horizontal components of motion.

4.2.2 Torsional Modes

Frequency $f = 1.6$ Hz corresponds to the first torsional mode of vibration, and can be identified in both the longitudinal (E-W) and transverse (N-S) recordings. Fig. 4.2.8a shows the corresponding mode-shape (measured at frame C), as determined from the transverse recordings. It was difficult to determine the phase lags in the transverse recordings at locations C4 and C6, due to the small signal to noise ratio. The centers of rotation (points near column 5, where the transverse response changes phase) were determined by calculating the phase angles from the transfer-functions, and from the cross-correlation functions. The locations of these centers may not be accurate for the lower floors (bellow 6th).
Table 4.2.8  Normalized amplitudes of NS and EW response: $f = 4.9$ Hz, Experiment I.

<table>
<thead>
<tr>
<th>Column</th>
<th>C1</th>
<th>C2</th>
<th>C3</th>
<th>C4</th>
<th>C5</th>
<th>C6</th>
<th>C7</th>
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</thead>
<tbody>
<tr>
<td>Orientation</td>
<td>EW</td>
<td>NS</td>
<td>EW</td>
<td>NS</td>
<td>EW</td>
<td>NS</td>
<td>EW</td>
<td>NS</td>
<td>EW</td>
</tr>
<tr>
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<td>0.130$^i$</td>
<td>0.66</td>
<td>0.330</td>
<td>0.53</td>
<td>0.330</td>
<td>0.40</td>
<td>0.060$^i$</td>
<td>0.20</td>
<td>-0.060</td>
</tr>
<tr>
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<td>0.33</td>
<td>0.130$^i$</td>
<td>0.26</td>
<td>0.130$^i$</td>
<td>0.20</td>
<td>0.060$^i$</td>
<td>0.13</td>
<td>0.000$^{1,2}$</td>
</tr>
<tr>
<td>6th floor</td>
<td>0.130$^i$</td>
<td>0.13</td>
<td>0.130$^i$</td>
<td>0.13</td>
<td>0.060$^i$</td>
<td>0.13</td>
<td>0.000</td>
<td>0.13</td>
<td>0.000$^{1,2}$</td>
</tr>
<tr>
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<td>0.200</td>
<td>-0.39</td>
<td>0.130$^i$</td>
<td>-0.33</td>
<td>0.060$^i$</td>
<td>-0.13</td>
<td>0.000</td>
</tr>
<tr>
<td>4th floor</td>
<td>0.270$^i$</td>
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<td>-0.060$^{1,2}$</td>
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1 There were difficulties in defining phase.
2 There were difficulties in defining amplitude.
Figure 4.2.7 First transverse mode, \( f=1.4 \) Hz, first experiment.
Figure 4.2.8a  First torsion mode, $f=1.6$ Hz, first experiment.
Figure 4.2.8b  EW and NS displacement, $f=1.6$ Hz, first experiment.
Figure 4.2.8c  Longitudinal displacement, $f=1.6$ Hz, first experiment.
Figure 4.2.8d  EW and NS  response plotted with relative longitudinal displacement, $f=1.6$ Hz, first experiment.
Figure 4.2.9  Second transverse mode, $f=3.9$ Hz, first experiment.
Figure 4.2.10a  Second torsion mode, $f=4.9$ Hz, first experiment.
Figure 4.2.10b  EW and NS response, $f=4.9$ Hz, first experiment.
Fig. 4.2.8b shows the mode-shape of the same mode, evaluated from both longitudinal and transverse recordings, and plotted in the plane of each floor. It is seen that, at each floor, the phase of the transverse response (measured along longitudinal frame C) changes, while of the longitudinal response remains the same. For this mode, 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 columns in the middle (C4, C5, C6) is clearly seen in all the recordings. Therefore, the torsional response for this mode seems to be combined with the longitudinal response. At the roof, 7th and 6th floors, this longitudinal response is out of phase with respect to the response of the 3rd and 4th floors. It was difficult to define the phases of longitudinal response at the 2nd and 5th floors.

The shaded oval zones in Fig. 4.2.8b illustrate the distribution of the centers of rotation for each of the floor slabs (determined by drawing a normal to the displacement vectors). Due to measurement errors and some deformation of the floor slabs, these “centers of rotation” fluctuate and are inside or close to the gray zones. It is seen that these centers of rotation are South of frame C, and move from between columns 5 and 6 at the roof, toward column 8 on the 6th floor. At and below the 5th floor, these centers of rotation are North of column line C and close to column line 5. The jump in the position of these centers of rotation, between the 5th and 6th floors coincides with the location of the damaged columns (Fig. 2.3 and 3.2.1).

Fig. 4.2.8c shows a plot only of the longitudinal displacement for the same mode. On the left, this displacement is plotted along column C5 and is denoted by $\Delta$. It is seen that, from column C1 towards column C9, $\Delta$ increases at the roof, 7th and 6th floors, decreases at the 5th and 4th floors, and changes sign between the 5th and the 6th floors. On the right, the longitudinal response amplitude is shown along the entire frame (C).

The damage in frames A and D, at the 2nd and 5th floors (Fig. 3.2.1), made the structure “softer” at those levels. The reduced stiffness in the vicinity of the plastic hinges, therefore, may have contributed to or even caused the coupling of the longitudinal displacement with the first torsional mode. This displacement is also noticeable in the longitudinal response, presumably because of the relatively large participation in the overall stiffness of the longitudinal frame A. The transverse response appears to be less affected by the damage of external, longitudinal frames A and D.

Fig. 4.2.8d is similar to Fig. 4.2.8b. The only difference is that the vectors show the response after reducing the longitudinal displacements by $\Delta$, the longitudinal displacement of column C5 (see Fig. 4.2.8c). It is seen that the middle columns,
especially at the higher floors, are experiencing significant torsion. The drift amplitudes
due to the torsional mode are larger at the exterior frames, because the slabs are stiff in
their own planes. Therefore, the torsion may well have been a major contributor to the
severe shear cracks in the middle columns of the 5th floor.

In search of possible scenarios that produced the observed damage, the above
measurements may be interpreted to mean that, once shear cracks occurred in the
columns at the 5th floor, the energy associated with the rotation was first dissipated by the
displacements in the newly formed “hinges”. Then either the increased eccentricity of the
response [Trifunac et a., 1999b], or the ground motion which excited the torsional and the
first transverse mode simultaneously, or both may have resulted in finally the external
frame A being more damaged than the external frame D.

Frequency $f = 4.9$ corresponds to the second torsional mode. Figure 4.2.10a shows the
corresponding mode-shape evaluated from the transverse recordings. As for the first
torsional mode, Fig. 4.2.10b shows the vectors of the combined transverse and
longitudinal response at this frequency.

### 4.2.3 Vertical Motions

No vertical modes of vibration could be detected from the vertical transfer-functions.
Figure 4.2.11 shows the in-plane (vertical and longitudinal) response of longitudinal
frame C at the frequency of the first longitudinal mode ($f = 1.0$ Hz), and Table 4.2.9
shows the corresponding normalized amplitudes. Figure 4.2.12 shows the vertical
response for the same frame at the frequency of the first transverse mode ($f = 1.4$ Hz), and
Table 4.2.10 lists the corresponding normalized amplitudes. The vertical and longitudinal
displacements are drawn on same scale. The observed building motions at those
frequencies are difficult to explain. The peaks in the transfer functions of vertical
responses can result from nonstructural vibrations of the slabs (if the instruments were
not placed close enough to the columns), could be associated with damage we did not
notice, or perhaps could be caused by some complicated coupling of horizontal and
vertical motions. At $f = 0.5$ Hz, an unexpected peak in all of the vertical recordings was
observed. Figure 4.2.13 shows the vertical displacements of frame C at this frequency,
and Table 4.2.11 shows the corresponding normalized amplitudes. An analysis of the
phases of the response along column C2 showed that the motion of the 5th, 6th and 7th
floors and of the roof was out of phase with respect to the motion of the ground floor.
This type of response may be due to some concentrated sources of vibration. At the roof,
the air conditioning fan was on during the entire duration of this experiment. This
unexplained peak could have resulted from some eccentricity in this equipment.
Figure 4.2.11  Longitudinal and vertical response, $f=1.0$ Hz, first experiment.
Figure 4.2.12  Vertical response, $f=1.4$ Hz, first experiment.
Figure 4.2.13  Vertical response, $f=0.5$ Hz, first experiment.
Table 4.2.9 Normalized amplitudes of vertical motion: $f = 1.0 \text{ Hz}$, Experiment I.

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<td>0.23</td>
<td>0.28</td>
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<tr>
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<td>0.01</td>
<td>0.11</td>
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<td>0.08</td>
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Table 4.2.10 Normalized amplitudes of vertical motion: $f = 1.4 \text{ Hz}$, Experiment I.

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Table 4.2.11 Normalized amplitudes of vertical motion: $f = 0.5 \text{ Hz}$, Experiment I.

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<td>0.02</td>
<td>0.02</td>
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4.3 Results from Experiment II

4.3.1 System Frequencies and Mode-Shapes for Horizontal Motions

The system frequencies identified from the transverse (N-S) recordings of this experiment were \( f = 1.4, 1.6, 4.2 \) and 4.9 Hz, the corresponding mode shapes are shown in Figs. 4.3.1–4.3.4, and the normalized amplitudes are given in the Tables 4.3.1–4.3.4. The peaks of the transfer functions for these modes are all above noise level.

For the longitudinal motions, the identified frequencies were \( f = 1.1, 3.7, 5.7 \) and 8.5 Hz. The corresponding mode shapes are shown in Fig. 4.3.5–4.3.9, and the normalized amplitudes are listed in Tables 4.3.5–4.3.8. For the mode for \( f = 8.5 \) Hz, the signal-to-noise ratio was small, and it was difficult to analyze the phases.

4.3.2 Torsional Modes

As for Experiment I, the second and third frequencies identified in the transverse recordings \( f = 1.6 \) Hz and \( f = 4.9 \) Hz, correspond to the first two torsional modes and can also be seen in the longitudinal recordings. Figs. 4.3.9 and 4.3.10 show these two modes constructed from both the transverse and longitudinal recordings, and Tables 4.3.9 and 4.3.10 show the amplitudes, normalized by the largest transverse response. It can be seen that the reinforced concrete floor slabs, 8.5 inches thick and stiff in their own plane, are translating and rotating 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. In contrast to the corresponding result for Experiment I (Fig. 4.2.8b), the centers of rotation for the first torsional mode in Fig. 4.3.9 are all south of frame C, and are all near column line 5. One explanation is that the added braces (see Fig. 3.3.1) have eliminated torsional eccentricities caused by the damaged columns, at fifth floor and mainly along the south frame A.

4.3.3 Vertical Motions

The translational horizontal modes may also be seen in the vertical response. As the building vibrates, most of the deformations occur in the columns. Due to bending of the columns, the slabs of the upper floors deform in the vertical direction also. Tables 4.3.11 and 4.3.12 show the normalized amplitudes of the vertical response of longitudinal frame C for the frequencies corresponding to the first longitudinal \( f = 1.1 \) Hz and to the first transverse \( f = 1.4 \) Hz modes. In Table 4.3.13 vertical response amplitudes along transverse frames 2, 5 and 8, on the third, fifth and seventh floors are given for the first three frequencies of the horizontal response \( f = 1.1, 1.4, 1.6 \) Hz. The amplitudes of the
Figure 4.3.1 First transverse mode, $f=1.4$ Hz, second experiment.
Figure 4.3.2 First torsion mode, $f=1.6$ Hz, second experiment.
Figure 4.3.3  Second transverse mode, $f=4.2$ Hz, second experiment.
Figure 4.3.4  Second torsion mode, \( f \approx 4.9 \) Hz, second experiment.
Table 4.3.1 Normalized amplitudes of NS mode of vibration: \( f = 1.4 \) Hz, Experiment II.

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<td>1.00</td>
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Table 4.3.2 Normalized amplitudes of torsional mode of vibration: \( f = 1.6 \) Hz, Exper. II.

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Table 4.3.3 Normalized Amplitudes of NS mode of vibration: \( f = 4.2 \) Hz, Experiment II.

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<td>0.66</td>
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1 On this floor, there were difficulties in determining the phase.
Table 4.3.4  Normalized amplitudes of torsional mode of vibration: $f = 4.9$ Hz, Exp. II.

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<td>0.10</td>
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<td>-0.52</td>
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<td>0.02</td>
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<td>-0.37</td>
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<td>0.37</td>
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Table 4.3.5  Normalized amplitudes of EW mode of vibration: $f = 1.1$ Hz, Experiment II.

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<td>0.99</td>
<td>1.00</td>
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<td>4th floor</td>
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<td>0.02</td>
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Table 4.3.6  Normalized amplitudes of EW mode of vibration: $f = 3.7$ Hz, Experiment II.

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<td>0.42</td>
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<td>0.47</td>
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<td>0.01</td>
<td>0.01</td>
<td>0.01</td>
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<td>0.01</td>
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<tr>
<td>4th floor</td>
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<td>-0.68</td>
<td>-0.69</td>
<td>-0.70</td>
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<td>-0.34</td>
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<td>-0.27</td>
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<td>-0.30</td>
<td>-0.30</td>
</tr>
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1 On this floor, there were difficulties in determining the phase.
Table 4.3.7  Normalized amplitudes of EW mode of vibration: $f = 5.7$ Hz, Experiment II.

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<td>1.00</td>
<td>0.98</td>
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<td>1.00</td>
</tr>
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<td>0.02</td>
<td>0.02</td>
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<td>0.01</td>
<td>0.01</td>
</tr>
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<td>0.64</td>
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</tr>
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<td>0.42</td>
</tr>
<tr>
<td>1st floor</td>
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Table 4.3.8  Normalized amplitudes of EW mode of vibration: $f = 8.5$ Hz, Experiment II.

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<td>0.43</td>
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<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.01</td>
<td>0.02</td>
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<td>-0.21</td>
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<td>-0.27</td>
<td>-0.24</td>
<td>-0.27</td>
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<td>-0.76</td>
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<td>-0.67$^1$</td>
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$^1$ On this floor, there were difficulties in determining the phase.
$^2$ On this floor, there were difficulties in determining the amplitude.
Figure 4.3.5  First longitudinal mode, $f=1.1$ Hz, second experiment.
Figure 4.3.6 Second longitudinal mode, $f=3.7$ Hz, second experiment.
Figure 4.3.7 Third longitudinal mode, $f=5.7$ Hz, second experiment.
Figure 4.3.8 Fourth longitudinal mode, $f=8.5$ Hz, second experiment.
Figure 4.3.9 EW and NS response plotted with total longitudinal displacement, $f=1.6$ Hz, second experiment.
Figure 4.3.10 EW and NS responses $f=4.9$ Hz, second experiment.
Table 4.3.9  Normalized amplitudes of EW and NS response: $f = 1.6$ Hz, Experiment II.

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<td>NS</td>
<td>EW</td>
<td>NS</td>
<td>EW</td>
<td>NS</td>
<td>EW</td>
<td>NS</td>
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<td>0.27</td>
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<td>0.100</td>
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<td>0.30</td>
<td>0.005</td>
<td>0.30</td>
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<tr>
<td>4th floor</td>
<td>0.020</td>
<td>0.60</td>
<td>0.080</td>
<td>0.70</td>
<td>0.020</td>
<td>0.31</td>
<td>0.000</td>
<td>0.19</td>
<td>0.000</td>
</tr>
<tr>
<td>3rd floor</td>
<td>0.005</td>
<td>0.48</td>
<td>0.005</td>
<td>0.47</td>
<td>0.010</td>
<td>0.27</td>
<td>0.000</td>
<td>0.20</td>
<td>0.000</td>
</tr>
<tr>
<td>2nd floor</td>
<td>0.005</td>
<td>0.40</td>
<td>0.000</td>
<td>0.23</td>
<td>-0.005</td>
<td>0.19</td>
<td>0.000</td>
<td>0.10</td>
<td>0.000</td>
</tr>
<tr>
<td>1st floor</td>
<td>0.000</td>
<td>0.04</td>
<td>0.000</td>
<td>0.04</td>
<td>0.000</td>
<td>0.04</td>
<td>0.000</td>
<td>0.04</td>
<td>0.000</td>
</tr>
</tbody>
</table>

1 There were difficulties in determining the phase.
Table 4.3.10  Normalized amplitudes of EW and NS response: $f = 4.9$ Hz, Experiment II.

<table>
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<th>C1</th>
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<th>C4</th>
<th>C5</th>
<th>C6</th>
<th>C7</th>
<th>C8</th>
<th>C9</th>
</tr>
</thead>
<tbody>
<tr>
<td>Orientation</td>
<td></td>
<td>EW</td>
<td>NS</td>
<td>EW</td>
<td>NS</td>
<td>EW</td>
<td>NS</td>
<td>EW</td>
<td>NS</td>
</tr>
<tr>
<td>roof</td>
<td>0.125$^1$</td>
<td>1.00</td>
<td>0.010$^1$</td>
<td>0.66</td>
<td>0.005$^1$</td>
<td>0.38</td>
<td>0.100$^1$</td>
<td>0.10</td>
<td>0.000</td>
</tr>
<tr>
<td>7th floor</td>
<td>0.050$^1$</td>
<td>0.56</td>
<td>0.015$^1$</td>
<td>0.40</td>
<td>0.005$^1$</td>
<td>0.27</td>
<td>0.050$^1$</td>
<td>0.12</td>
<td>0.000</td>
</tr>
<tr>
<td>6th floor</td>
<td>0.025$^1$</td>
<td>0.32</td>
<td>0.010$^1$</td>
<td>0.18</td>
<td>0.005$^1$</td>
<td>0.09</td>
<td>0.025$^1$</td>
<td>0.02$^2$</td>
<td>0.000</td>
</tr>
<tr>
<td>5th floor</td>
<td>0.000</td>
<td>-0.12$^2$</td>
<td>0.000$^1$</td>
<td>-0.08</td>
<td>0.000</td>
<td>0.00</td>
<td>0.010$^1$</td>
<td>0.00</td>
<td>0.000</td>
</tr>
<tr>
<td>4th floor</td>
<td>0.000</td>
<td>-0.21</td>
<td>0.010$^1$</td>
<td>-0.17</td>
<td>0.005$^1$</td>
<td>-0.08</td>
<td>0.010$^1$</td>
<td>0.00</td>
<td>0.000</td>
</tr>
<tr>
<td>3rd floor</td>
<td>0.025$^1$</td>
<td>-0.62</td>
<td>0.025$^1$</td>
<td>-0.47</td>
<td>0.015$^1$</td>
<td>-0.27</td>
<td>0.025$^1$</td>
<td>-0.01</td>
<td>0.000</td>
</tr>
<tr>
<td>2nd floor</td>
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<td>0.020$^1$</td>
<td>-0.38</td>
<td>0.010$^1$</td>
<td>-0.21</td>
<td>0.010$^1$</td>
<td>-0.10</td>
<td>0.000</td>
</tr>
<tr>
<td>1st floor</td>
<td>0.000</td>
<td>-0.09</td>
<td>-0.000</td>
<td>0.09</td>
<td>0.000</td>
<td>0.00</td>
<td>0.000</td>
<td>0.00</td>
<td>0.000</td>
</tr>
</tbody>
</table>

$^1$ There were difficulties in defining the phase.  
$^2$ There were difficulties in defining the amplitude.
Table 4.3.11  Normalized amplitudes of vertical motion: $f = 1.1$ Hz, Experiment II.

<table>
<thead>
<tr>
<th>Column</th>
<th>C1</th>
<th>C2</th>
<th>C3</th>
<th>C4</th>
<th>C5</th>
<th>C6</th>
<th>C7</th>
<th>C8</th>
<th>C9</th>
</tr>
</thead>
<tbody>
<tr>
<td>roof</td>
<td>0.29</td>
<td>0.14</td>
<td>0.14</td>
<td>0.48</td>
<td>0.80</td>
<td>0.38</td>
<td>1.00</td>
<td>0.10</td>
<td>0.36</td>
</tr>
<tr>
<td>7th floor</td>
<td>0.72</td>
<td>0.30</td>
<td>0.24</td>
<td>0.34</td>
<td>0.15</td>
<td>0.50</td>
<td>0.89</td>
<td>0.20</td>
<td>0.69</td>
</tr>
<tr>
<td>6th floor</td>
<td>0.73</td>
<td>0.18</td>
<td>0.06</td>
<td>0.56</td>
<td>0.14</td>
<td>0.29</td>
<td>0.49</td>
<td>0.19</td>
<td>0.18</td>
</tr>
<tr>
<td>5th floor</td>
<td>0.30</td>
<td>0.30</td>
<td>0.12</td>
<td>0.39</td>
<td>0.18</td>
<td>0.11</td>
<td>0.20</td>
<td>0.25</td>
<td>0.23</td>
</tr>
<tr>
<td>4th floor</td>
<td>0.14</td>
<td>0.14</td>
<td>0.68</td>
<td>0.16</td>
<td>0.11</td>
<td>0.58</td>
<td>0.40</td>
<td>0.24</td>
<td>0.20</td>
</tr>
<tr>
<td>3rd floor</td>
<td>0.14</td>
<td>0.14</td>
<td>0.10</td>
<td>0.10</td>
<td>0.14</td>
<td>0.10</td>
<td>0.14</td>
<td>0.11</td>
<td>0.10</td>
</tr>
<tr>
<td>2nd floor</td>
<td>0.10</td>
<td>0.10</td>
<td>0.10</td>
<td>0.10</td>
<td>0.10</td>
<td>0.10</td>
<td>0.10</td>
<td>0.10</td>
<td>0.10</td>
</tr>
<tr>
<td>1st floor</td>
<td>0.10</td>
<td>0.10</td>
<td>0.10</td>
<td>0.10</td>
<td>0.10</td>
<td>0.11</td>
<td>0.10</td>
<td>0.08</td>
<td>0.10</td>
</tr>
</tbody>
</table>

Table 4.3.12  Normalized amplitudes of vertical motion: $f = 1.4$ Hz, Experiment II.

<table>
<thead>
<tr>
<th>Column</th>
<th>C1</th>
<th>C2</th>
<th>C3</th>
<th>C4</th>
<th>C5</th>
<th>C6</th>
<th>C7</th>
<th>C8</th>
<th>C9</th>
</tr>
</thead>
<tbody>
<tr>
<td>roof</td>
<td>0.53</td>
<td>0.64</td>
<td>0.42</td>
<td>0.72</td>
<td>0.72</td>
<td>0.72</td>
<td>0.72</td>
<td>0.72</td>
<td>0.72</td>
</tr>
<tr>
<td>7th floor</td>
<td>1.00</td>
<td>0.52</td>
<td>0.48</td>
<td>0.72</td>
<td>0.71</td>
<td>0.52</td>
<td>1.00</td>
<td>0.72</td>
<td>0.72</td>
</tr>
<tr>
<td>6th floor</td>
<td>0.50</td>
<td>0.48</td>
<td>0.42</td>
<td>0.48</td>
<td>0.48</td>
<td>0.48</td>
<td>0.48</td>
<td>0.48</td>
<td>0.68</td>
</tr>
<tr>
<td>5th floor</td>
<td>0.00</td>
<td>0.61</td>
<td>1.00</td>
<td>0.33</td>
<td>0.74</td>
<td>0.62</td>
<td>0.48</td>
<td>0.67</td>
<td>0.42</td>
</tr>
<tr>
<td>4th floor</td>
<td>0.33</td>
<td>0.00</td>
<td>0.33</td>
<td>0.50</td>
<td>0.33</td>
<td>1.00</td>
<td>1.00</td>
<td>0.33</td>
<td>0.48</td>
</tr>
<tr>
<td>3rd floor</td>
<td>0.33</td>
<td>0.00</td>
<td>0.33</td>
<td>0.50</td>
<td>0.33</td>
<td>1.00</td>
<td>1.00</td>
<td>0.33</td>
<td>0.48</td>
</tr>
<tr>
<td>2nd floor</td>
<td>0.30</td>
<td>0.33</td>
<td>0.33</td>
<td>0.30</td>
<td>0.37</td>
<td>0.33</td>
<td>0.33</td>
<td>0.33</td>
<td>0.33</td>
</tr>
<tr>
<td>1st floor</td>
<td>0.33</td>
<td>0.33</td>
<td>0.33</td>
<td>0.33</td>
<td>0.33</td>
<td>0.30</td>
<td>0.33</td>
<td>0.33</td>
<td>0.33</td>
</tr>
</tbody>
</table>
transfer-functions for these frequencies are comparable. Normalization has been performed for each frequency separately.

Table 4.3.13  Normalized amplitudes of the vertical response: $f = 1.1$ Hz, $f = 1.4$, $f = 1.6$ Hz, Experiment II.

<table>
<thead>
<tr>
<th>Columns</th>
<th>3rd Floor</th>
<th>5th Floor</th>
<th>7th Floor</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f=1.1$ Hz</td>
<td>A2, A5, A8</td>
<td>0.20 0.14 0.16</td>
<td>0.10 0.10 0.10</td>
</tr>
<tr>
<td></td>
<td>B2, B5, B8</td>
<td>0.24 0.11 0.16</td>
<td>0.18 0.54 0.60</td>
</tr>
<tr>
<td></td>
<td>D2, D5, D8</td>
<td>0.10 0.10 0.10</td>
<td>0.10 0.10 0.10</td>
</tr>
<tr>
<td>$f=1.4$ Hz</td>
<td>A2, A5, A8</td>
<td>0.16 0.10 0.10</td>
<td>0.56 1.00 0.86</td>
</tr>
<tr>
<td></td>
<td>B2, B5, B8</td>
<td>0.19 0.14 0.10</td>
<td>0.42 0.25 0.22</td>
</tr>
<tr>
<td></td>
<td>D2, D5, D8</td>
<td>0.16 0.10 0.10</td>
<td>0.24 0.34 0.34</td>
</tr>
<tr>
<td>$f=1.6$ Hz</td>
<td>A2, A5, A8</td>
<td>0.10 0.10 0.10</td>
<td>0.10 0.10 0.20</td>
</tr>
<tr>
<td></td>
<td>B2, B5, B8</td>
<td>0.10 0.10 0.10</td>
<td>0.10 0.15 0.10</td>
</tr>
<tr>
<td></td>
<td>D2, D5, D8</td>
<td>0.10 0.10 0.10</td>
<td>0.10 0.10 0.10</td>
</tr>
</tbody>
</table>

Figures 4.3.11–4.3.13 show the transverse and vertical motions at $f = 1.4$ Hz, the frequency of the first transverse mode, along transverse frames 2, 5 and 8. The vertical displacements have been exaggerated by a factor of two to emphasize the deformation of the columns. A large vertical displacement can be seen from Fig. 4.3.12 for column A5 at the 5th floor. This column was severely damaged during the Northridge earthquake. Large shear cracks (Fig. 2.3) decreased its axial capacity to resist relative vertical displacements. While, at 5th floor, the vertical deflection of the slab is noticeable, that is not the case at the 7th floor, presumably because of the 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 (Fig. 2.3).
Figure 4.3.11 Horizontal and vertical response of frame 2, $f=1.4$ Hz, second experiment.
Figure 4.3.12  Horizontal and vertical response of frame 5, \( f=1.4 \text{ Hz} \), second experiment.
Figure 4.3.13  Horizontal and vertical response of frame 8, $f=1.4$ Hz, second experiment.
Figures 4.3.14−4.3.16 show the transverse and vertical motions at \( f = 1.6 \) Hz, the frequency of the first torsional mode, along transverse frames 2, 5 and 8. It is seen that, for this mode, the vertical displacements are small. As in the previous set of figures, the vertical displacements have been exaggerated two times relative to the horizontal ones, and it is assumed that all floor slabs have horizontal response same as frame C.

Figures 4.3.17−4.3.20 show the horizontal and vertical displacements at \( f = 1.1 \) Hz, the frequency of the first longitudinal mode, along longitudinal frames A, B, C and D. The motions of frame C (Fig. 4.3.19) are described more completely (at all columns). Large vertical displacement is seen along column C3 on the 5th and 4th floors, and along column C7 on the roof, 7th and 6th floors. We cannot explain these large vertical displacements, as there were no visible signs of damage at these locations. The motions of longitudinal frames A, B and D (Fig. 4.3.17, 4.3.18 and 4.3.20) are described in less detail: only at columns 2, 5 and 8, and at the 1st, 3rd, 5th and 7th floors. The plotted vertical motions were actually measured at these locations, and the longitudinal displacements are those measured along frame C (it was assumed that the differences between the motion of frame C and frames A, B and D would be small because the slabs are stiff in the longitudinal direction). Although the exterior frames, A and D, were damaged, we did not expect them to experience large longitudinal displacements, because of participation of the interior frames, B and C, through the stiff slab. However, the measured vertical displacements, associated with this (first longitudinal) mode, were also not large even at the columns which lost most of their axial capacity (A5 at the 5th floor). We recall that it was different for the vertical displacements associated with the first transverse mode. It had large response at the damaged columns. This difference may be due to interaction of damaged column A5 with the two surrounding columns, A4 and A6. During earthquake shaking, column A4 was damaged less than column A5, while column A6 was not damaged (see Fig. 2.3).

Figure 4.3.21 shows the transverse and vertical displacements of longitudinal frame C at \( f = 1.4 \) Hz, the frequency of the first transverse mode. The vertical displacements have been exaggerated by factor of two. The plotted mode shape does not suggest damage in the columns of frame A, presumably because of participation of the transverse frames, through the stiff slabs.
Figure 4.3.14  Horizontal and vertical response of frame 2, $f=1.6$ Hz, second experiment.
Figure 4.3.15  Horizontal and vertical response of frame 5, \( f=1.6 \) Hz, second experiment.
Figure 4.3.16  Horizontal and vertical response of frame 8, $f=1.6$ Hz, second experiment.
Figure 4.3.17  Horizontal and vertical response of frame A, $f=1.1$ Hz, second experiment.
Figure 4.3.18  Horizontal and vertical response of frame B, $f=1.1$ Hz, second experiment.
Figure 4.3.19 Horizontal and vertical response of frame C, $f=1.1$ Hz, second experiment.
Figure 4.3.20  Horizontal and vertical response of frame D, $f=1.1$ Hz, second experiment.
Figure 4.3.21  Horizontal and vertical response of frame C, $f=1.4$ Hz, second experiment.
4.4 Summary of the Ambient Vibrations Survey Results

The VN7SH building has symmetric geometry and roughly uniform distribution of mass and stiffness. It has thick heavy slabs, stiff in their own planes, and spandrel beams connecting the outside columns, causing the exterior frames to carry most of the lateral loads. According to these characteristics, its structural system cannot be described as a “weak beam-strong column” system. In spite of its symmetry, the structure experienced strong torsional response [Trifunac et al., 1999b].

The dynamic characteristics of the building (frequencies and mode shapes) derived from the two ambient vibrations surveys, are summarized in Table 4.4.1 for longitudinal (E-W) and transverse (N-S) motions (measured at the interior longitudinal frame “C”). The first columns of these tables show sketches of the mode-shapes, the second and third columns show the modal frequencies for the first four modes, and the fourth column shows the change in the modal frequencies. Changes are expected because of differences in the state of the structure and of the underlying soil. The first survey was conducted two weeks after the 1994 Northridge earthquake (which, along with its early aftershocks, caused severe structural damage), and the second experiment was conducted three months following the main event. During the time between the two surveys, the structure was temporarily restrained by wooden braces (Fig. 3.3.1), and about one month before the second survey, one of the largest aftershocks occurred very close to the building site ($M=5.2$, $R=1.2$ km). It may have caused additional damage.

The changes in modal frequencies are as follows. From Table 4.4.1 (left) it is seen that three out of the four identified frequencies for longitudinal vibrations increased, and one ($f=5.7$ Hz) reminded the same. This increase in frequencies resulted from the addition of the wooden braces, placed at the longitudinal frames (Fig. 3.3.1). The deformation of the first longitudinal mode is such that the added braces increased its frequency by 10 percent. The frequencies of the second and of the fourth longitudinal modes increased by 6 and 5 percents respectively. The frequency of the third mode was not affected. The data in Table 4.4.1 (right) show that the frequencies of the first transverse mode and of the first torsional mode did not change. The braces located along the longitudinal frames did not increase stiffness for those two modes. However, the frequency of the third mode increased by 10 percents.

No mode-shapes were identified for vertical vibrations (the strong vertical motion at $f=0.5$ Hz, observed during the first survey, is probably due to vibrations caused by the electrical equipment). The Fourier spectra of the recorded vertical motions did have peaks, corresponding to the modes associated primarily with horizontal motions. The recorded vertical responses at these frequencies, analyzed alone or combined with the
corresponding horizontal response, were useful for identification of the damage of the columns. Clear changes in the amplitudes of the vertical responses (larger vertical amplitudes implied more severe loss of axial capacity) were noticed near one of the damaged columns where ambient noise was measured. 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 peaks of the transfer-functions evaluated from Experiment II had 30% smaller amplitudes than those evaluated for Experiment I, even though the measuring and reference points coincided, and same instrumentation and data processing procedures were used. All the calibration tests gave consistent results for the frequency dependent sensitivity ratios for all sensors.
5. SUMMARY OF OTHER STUDIES OF THE VN7SH BUILDING

The following summarizes our previous and other current work on the VN7SH building, as it relates to the work presented in this report. At the end, we draw conclusions and present recommendations for future recording and analyses of earthquake response and of ambient noise response of structures.

5.1 Earthquake Damage and Strong Motion Data

These were described in detail in a report by Trifunac et al. [1999a]. In the following, we summarize the principal findings.

5.1.1 1971 San Fernando Earthquake

The structural damage following this earthquake was minor. Epoxy was used to repair the spalled concrete of the second floor beam-column joints on the North and East ends of the building. The nonstructural damage was extensive; about 80% of all the repair cost was spent on repair of the drywall partitions, bathroom tiles and on plumbing fixtures. The damage was most severe at the 2nd and 3rd floors, and was minimal at the 6th and 7th floors.

5.1.2 1994 Northridge Earthquake and its Early Aftershocks

The damage following this earthquake was extensive. The building was classified as unsafe (red-tagged) by the Los Angeles Department of Building Safety and it was closed. Severe structural damage occurred in the exterior longitudinal frames (South-A and North-D frames) designed to take most of the lateral loads in the E-W direction. In Frame A (south side), wide shear cracks appeared in columns A3, A4, A5, A7 and A8, just below the contact with the spandrel beam of the fifth floor. At contacts A5F5 (longitudinal frame A, column 5, 5th floor), the cracks at the surface were 5 to 10 cm wide. The shear cracks in the exterior frame D were moderate (0.2−1 cm wide on the surface). Those cracks had clearly visible “x” shape. The nonstructural damage was also extensive. Every guestroom suffered some type of nonstructural damage. The furniture was overturned on the upper floors (above 3rd). Due to the large relative motions and deformation of the interior walls, wallpaper was distorted or torn off. The relative displacements also caused extensive damage to the brittle ceramic tile covers in the bathrooms.
5.1.3 Earthquake Data

Recordings of the following earthquakes were available and were analyzed (listed in chronological order; R is the epicentral distance):

Table 5.1.1 Earthquakes for which accelerograms in the VN7SH building are known to have been recorded.

<table>
<thead>
<tr>
<th>No.</th>
<th>Earthquake</th>
<th>Date</th>
<th>M</th>
<th>R [km]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>San Fernando</td>
<td>02/09/1971</td>
<td>6.6</td>
<td>22</td>
</tr>
<tr>
<td>2</td>
<td>Whittier Narrows</td>
<td>10/01/1987</td>
<td>5.9</td>
<td>41</td>
</tr>
<tr>
<td>3</td>
<td>Whitter-Narrows aft.</td>
<td>10/04/1987</td>
<td>5.3</td>
<td>38</td>
</tr>
<tr>
<td>4</td>
<td>Pasadena</td>
<td>10/03/1988</td>
<td>4.9</td>
<td>32</td>
</tr>
<tr>
<td>5</td>
<td>Montebello</td>
<td>06/12/1989</td>
<td>4.1</td>
<td>34</td>
</tr>
<tr>
<td>6</td>
<td>Malibu</td>
<td>01/19/1989</td>
<td>5.0</td>
<td>36</td>
</tr>
<tr>
<td>7</td>
<td>Sierra Madre</td>
<td>06/28/1991</td>
<td>5.8</td>
<td>44</td>
</tr>
<tr>
<td>8</td>
<td>Landers</td>
<td>06/28/1992</td>
<td>7.5</td>
<td>186</td>
</tr>
<tr>
<td>9</td>
<td>Big Bear</td>
<td>06/28/1992</td>
<td>6.5</td>
<td>149</td>
</tr>
<tr>
<td>10</td>
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<td>01/17/1994</td>
<td>6.5</td>
<td>1.5</td>
</tr>
<tr>
<td>11</td>
<td>Northridge aft.</td>
<td>03/20/1994</td>
<td>5.2</td>
<td>1.2</td>
</tr>
<tr>
<td>12</td>
<td>Northridge aft.</td>
<td>10/06/1994</td>
<td>4.5</td>
<td>10.8</td>
</tr>
</tbody>
</table>

During the 1971 San Fernando earthquake, the building had only three self-contained AR-240 (analogue) triaxial accelerographs. Prior to 1987, a 13 channel CR-1 recording system (analogue) was installed and one SMA-1 triaxial accelerograph on the first floor. The largest recorded motion was during the 1994 Northridge earthquake. The current instrumentation is operated by the California Division of Mines and Geology (CDMG).

Processed data of the Northridge main event were made available by CDMG in 1995. Processed data of the Whittier-Narrows, Landers and Big Bear earthquakes were released at a later date. The records of the other events (see Table 5.1.1) were digitized at USC from contact copies provided by CDMG. For the purpose of our analysis, the data was reprocessed by the 1997 version of the LeBatch software package of Lee and Trifunac [1990].
5.2 Analysis of Earthquake Response Data

The purpose of these analyses was to detect from the data time and amplitude dependent changes in the response of the building. The simplicity of the building geometry made the interpretation of the recorded structural response feasible and relatively simple.

5.2.1 Wave Propagation Analyses

Todorovska et al. [2000] estimated the wave velocities in the VN7SH building from the recorded acceleration response (13 CR-1 channels and 3 SMA-1 channels) by computation of cross-correlation of the motion recorded by different channels and via Fourier transforms. They estimated the apparent surface wave velocity in the soil from the recordings at the ground floor, assuming no soil-structure interaction. The values they obtained are ~100 m/s for the waves propagating vertically along the columns and ~2,000 m/s for the waves propagating horizontally along the floor slabs. They compared the values for different earthquakes and studied the changes in wave speed in the areas that were severely damaged during the Northridge earthquake.

Ivanović et al. [1999] analyzed the building response data using cross-correlation analysis with sliding time window. Their results indicate significant delays in wave travel times through those parts of the structure that were known to have experienced major damage from Northridge earthquake. This type of analysis is a promising tool for structural health monitoring and damage detection.

5.2.2 Spectral Characteristics of the Building-Soil Response

Trifunac et al. [2000a] analyzed ambient noise data recorded in the damaged building following the Northridge earthquake. They computed the ratio of Fourier spectra of the building response at the upper floors relative to the first (ground) floor (these would have been the building transfer-functions if its response were linear) and found that it had a broad peak below 1 Hz. This frequency is lower than the fundamental fixed-base frequency estimated either from the structural characteristics or from ambient vibration measurements \((f > 1.4 \text{ Hz for the NS direction and } f > 1.1 \text{ Hz for the EW direction})\). They showed that the apparent “loss of stiffness” cannot be explained either by nonlinearity of the structure alone nor by its degradation due to damage. This indicated the possibility that the nonlinearity of the soil contributed mainly to the nonlinear behavior of the overall system.

The soil-structure interaction results in rocking and torsion of the building foundation. The motions in each of the two planes of symmetry of the building can be represented as a sum of the rocking motion of the building as a rigid body and the relative deflection due
to lateral deformation of the columns. Ideally, for a rigid foundation, the rocking in the 
NS direction during strong earthquake shaking would be calculated from two recordings 
of vertical motion at the opposite ends of the building. Unfortunately, this could not be 
done for the VN7SH building because of insufficient instrumentation (the building had 
only one vertical sensor at ground level). In their study of the amplitude and time 
dependent changes in the period of the VN7SH building using all the available processed 
strong motion recordings, Trifunac et al. [2000a] “extracted” approximately the 
foundation “rocking” and the building relative response by low-pass and high-pass 
filtering of the recorded total system response. The next section describes their findings.

5.2.3 Nonlinear Building-Soil Response

The nonlinear effects in a time response of a system depend on the level of the excitation 
and on its initial state. In the case of a building-soil system, the initial state would 
depend on the state of the structure as well as on the state of the soil (degree of 
consolidation, water content etc.). A change in the system stiffness may be due to 
changes only in the structure, only in the soil, or in both. While the changes due to 
damage within the structure are permanent, the soil may recover its original stiffness 
following strong shaking, but this may take time. Short term temporal and amplitude 
variations in the system period can be studied by moving-window [Udwadia and 
Trifunac, 1974] or zero-crossing analyses of individual earthquake recordings, and long 
term variations can be analyzed by comparison of results for different earthquakes.

Trifunac et al. [2000b] performed zero-crossing analyses on the “rocking” angles $\theta_y(t)$ 
and $\theta_x(t)$ band-pass filtered by Ormsby filters between 0.1 and 1.0 Hz (with roll-offs at 
0.2 and 0.8 Hz). This analysis consisted of reading manually the half periods for all of 
the approximately symmetric response peaks in the time history, and computing and 
plotting (versus time) the corresponding “period” of vibration ($T_p = 1/f_p$). The results 
showed progressive decrease in the system frequency, proportional to the intensity of 
motion, but the reduction in soil “stiffness” was not permanent (Fig. 5.2.1 and 5.2.2). 
The frequency of the soil-structure system apparently can return to its original value, if 
there is no major permanent damage in the building. Based on the limited data that we 
analyzed, “healing” of the soil seems to have occurred, resulting probably from dynamic 
consolidation caused by the aftershocks and by smaller earthquakes. The regenerated soil 
stiffness is capable of experiencing repeated large deformations, suggesting that quick 
“healing” of the soil, after an earthquake and caused by the aftershocks, can increase the 
cumulative energy absorption capacity, thus increasing the safety of the system. 
Trifunac et al. [2000b] concluded that the nonlinear soil-structure interaction affected 
significantly the system response of the VN7SH building, and that the soil nonlinearity
Fig. 5.2.1 Peaks of $\ddot{\theta}_x$ (EW rocking acceleration) versus $f_p$ (apparent frequency of soil-foundation-structure system) for all twelve earthquakes listed in Table 5.1.1 (see Trifunac et al., 2000a).
Fig. 5.2.2   Peaks of $\ddot{\theta}_y$ (NS rocking acceleration versus $f_p$ (apparent frequency of soil-foundation-structure system) for all twelve earthquake listed in table 5.1.1 (see Trifunac et al., 2000a).
accounted for much of the reduction of the system stiffness. They considered such an effect to be beneficial for the building, because of dissipation of energy in the soil via nonlinear deformations, and consequent reduction of the energy exciting the structure. Finally, they speculated that, if founded on stiffer soil, this building would have suffered more severe damage from the Northridge earthquake.

Besides its significance for explaining correctly the changes in the system stiffness, considering soil-structure interaction is also important for correct estimation of the inter-storey drift from earthquake recordings. For the Northridge earthquake, the drift angles of relative building deformation exceeded 0.004 and severe damage occurred. This drift was up to four times smaller than the apparent “drift” (computed based on the assumption that there is no soil-structure interaction). The peak amplitudes of $\dot{\theta}_x$ and $\dot{\theta}_y$, corresponding to drift angles 0.002 to 0.0033 are shown in Figs 5.2.1 and 5.2.2 by hatched zones.

### 5.3 Ambient Vibration Surveys

#### 5.3.1 Building Response to Ambient Noise

This report describes two detailed ambient vibration surveys of the VN7SH building. The first one was carried out on 4–5 February, 1994, about two weeks following the Northridge earthquake, and the second one was carried out two months later, on 19–20 April 1994, about a month after the M=5.3 aftershock of 20 March, 1994. Total of six transducers were used, four Ranger SS-1 seismometers and two “old” Earth Sciences Rangers. Ambient noise was recorded along one of the interior longitudinal frames (C), on each floor, at each of the nine columns, and for all three components of motion (L, T and V). The three Ranger SS-1 seismometers were used to record at all the measuring points. The values of the apparent frequencies for the first four modes of vibration obtained from the first experiment were: $f = 1.0, 3.5, 5.7$ and $8.1$ Hz for the longitudinal (E-W) direction, and $f = 1.4, 1.6, 3.9$ and $4.9$ Hz for the transverse (N-S) direction. The corresponding values obtained from the second experiment were: $f = 1.1, 3.7, 5.7$ and $8.5$ Hz for the longitudinal (E-W) direction, and $f = 1.4, 1.6, 3.9$ and $4.9$ Hz for the transverse (N-S) direction. Table 4.4.1 shows sketches of the mode-shapes and summarizes the first four identified frequencies for transverse and longitudinal vibrations for both experiments.
5.3.2 Ground Response to Ambient Noise

During the second ambient vibration experiment, detailed ambient noise measurements were conducted in the parking lot surrounding the building. Trifunac et al. [1999b] present the results in form of contours of amplitude and time delay of the ground motion relative to reference points on the ground floor of the building. The aim was to detect ground deformation associated with at least the fundamental transverse (1.4 Hz) and longitudinal (1.1 Hz) modes of vibration of the building [Foutch et al., 1975; Trifunac et al., 2000b]. The overall pattern of the time delay, \( \tau \), (computed via cross-correlation), implies that the waves arrived from the west and scattered and diffracted around the building foundation. The measured delays for the horizontal components of motions imply apparent horizontal phase velocity of about 300 m/s, consistent with the interpretation that microtremors are high frequency Rayleigh waves propagating through the shallow soil layers. The delays for vertical motions also imply wave arrival from West and South-West, with apparent phase velocities 250 to 300 m/s.

5.3.3 Small (Ambient Noise) versus Large (Strong Motion) Response Amplitudes

The second ambient vibration experiment was carried out about three months after the earthquake and many of its aftershocks, perhaps too soon after the earthquake for the “gaps” and “clearances” between the vertical walls of the building and the soil, and between the piles and the surrounding soil to close and to be “recemented”. It may be that what we observed in the results of the ambient vibration measurements represents the response of the “disturbed” foundation system with “minute cracks” and gaps in the foundation soil, causing the wave motion in the parking lot to be irregular [Trifunac et al., 1999b]. Of course, this is further complicated by an apparent arrival of ambient wave energy from different directions, most likely from the moving sources (north-south traveling vehicle traffic on the San Diego Freeway, just 150 m west of the site).

With reference to Fig. 5.2.1 and 5.2.2, it is seen that during both ambient vibration tests (February 4-5 and April 19-20, 1994) the fundamental apparent frequency of the NS response is near 1.4 Hz, and close to the apparent system frequency during the smallest earthquake motions (1989 Montebello earthquake, Fig. 5.2.2). This suggests that the soil-foundation-structure stiffness “recovered” in full, during Northridge aftershocks. For the EW response (Fig. 5.2.1), the ambient vibration surveys indicate only a partial recovery, from 0.4-0.6 Hz during the main event to 1-1.1 Hz (note that during the 20 March aftershock, \( f_p \sim 1.3-1.4 \) Hz, see Fig. 5.2.1). The shift in \( f_p \) from 1.0 Hz (4-5 February, 1994) to 1.1 Hz (19-20 April, 1994) could be interpreted to result in part from
an increase in the “building stiffness” associated with temporary wooden braces, primarily along frames A and D. It appears that the soil-pile-foundation system behaved like a nonlinear system with many gap elements which are first closed, then opened and displaced by large amplitudes of strong motion, and then partly or completely closed during the aftershock excitations.

5.4 Discussion and Recommendations

We hope that the work presented in this report, as well as the related work reviewed in this chapter, has demonstrated once again the value of full-scale experiments in describing the dynamic characteristics of real, three-dimensional structures. The results of this analysis can be used as a guideline in selecting recording systems in buildings (number of channels and their location in the building), so that future earthquake recordings of building response provide more valuable information on the structural performance during earthquakes.

For the VN7SH building, most of the sixteen channels in the building recorded horizontal motions. The lack of vertical recordings made it impossible to estimate the rocking response associated with soil structure interaction. Additional vertical recorders, placed at the opposite diagonal corners, would have provided invaluable information to measure the rocking (longitudinal and transverse) responses due to soil-structure interaction. Free-field recordings of ground motion around the building would have, as well, provided valuable information for analysis of the building’s foundation response.

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


