Experimental evidence for flexibility of a building foundation supported by concrete friction piles

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Abstract

This article explores the possibility to measure deformations of building foundations from measurements of ambient noise and strong motion recordings. The case under study is a seven-storey hotel building in Van Nuys, California. It has been instrumented by strong motion accelerographs, and has recorded several earthquakes, including the 1971 San Fernando (ML = 6.6, R = 22 km), 1987 Whittier–Narrows (ML = 5.9, R = 41 km), 1992 Landers (ML = 7.5, R = 186 km), 1992 Big Bear (ML = 6.5, R = 149 km), and 1994 Northridge (ML = 6.4, R = 1.5 km) earthquake and its aftershocks (20 March: ML = 5.2, R = 1.2 km; 6 December, 1994: ML = 4.3, R = 11 km). It suffered minor structural damage in 1971 earthquake and extensive damage in 1994. Two detailed ambient vibration tests were performed following the Northridge earthquake, one before and the other one after the 20 March aftershock. These included measurements at a grid of points on the ground floor and in the parking lot surrounding the building, presented and analyzed in this article. The analysis shows that the foundation system, consisting of grade beams on friction piles, does not act as a ‘‘rigid body’’ but deforms during the passage of microtremor and therefore earthquake waves. For this geometricaly and by design essentially symmetric building, the center of stiffness of the foundation system appears to have large eccentricity (this is seen both from the microtremor measurements and from the earthquake recordings). This eccentricity may have contributed to strong coupling of transverse and torsional responses, and to larger than expected torsional response, contributing to damage during the 1994 Northridge, earthquake. © 1999 Elsevier Science Ltd. All rights reserved.

Keywords: Soil–structure interaction; Flexible foundation; Northridge earthquake; Ambient vibration test; Structural response; Instrumented buildings

1. Introduction

Earthquake-resistant design of structures must be based on analyses of realistic models of the structure, foundation and soil system, considering wave propagation and all the aspects of nonlinear response. Such analyses require solution of a complicated and difficult to solve system of governing equations and boundary conditions. Hence, it has been necessary to make various simplifications. In doing so, it is important to evaluate the accuracy of the approximations and to define the range of the model parameters for which the approximations are valid. This is best accomplished by careful experimental verification using full scale tests of actual structures.

A common assumption in many models which consider soil–structure interaction effects is that the foundation is rigid. This reduces the number of degrees-of-freedom of the model, and gives good approximations for long wavelengths relative to the foundation dimensions [7]. For short wavelengths, this assumption can result in non-conservative estimates of the relative deformations in the structure [23, 25] and, in general, is expected to result in excessive estimates of scattering of the incident wave energy and in excessive radiation damping [15,18,19]. The extent to which this assumption is valid depends on the stiffness of the foundation system relative to that of the soil, and also on the overall rigidity of the structure [5,8]. For a nine-storey reinforced concrete building, extensively tested during the 1970s, the foundation could be represented by a ‘‘rigid’’ slab for NS vibrations (because of stiffening effects of the end shear walls) but not for EW vibrations [2,9–13,27]. The other extreme is to neglect the stiffness of the foundation system and to assume that the wave energy is transmitted from soil into the building according to the principles of wave propagation [14–17,20]. This approximate approach...
Fig. 1. Geometrical relationship of the building site to the earthquakes causing strong motion. Whittier, Landers and Big Bear earthquake are outside the limits of this figure, at epicentral distances 41, 186 and 149 km, respectively.

Fig. 2. (a) Typical floor plan. (b) Foundation plan. (c) Typical transverse section. (d) Log of typical soil boring.
underestimates the incident wave energy scattered by the foundation and overestimates the energy transmitted into the building. The reality is somewhere between these two approximations, and can be studied in detail only by means of numerical methods.

In this article, an instrumented seven-storey hotel building in Van Nuys, California, is studied. Records of several earthquakes were available for the study, including the 1971 San Fernando ($M_L = 6.6$, $R = 22$ km), 1987 Whittier–Narrows ($M_L = 5.9$, $R = 41$ km), 1992 Landers ($M_L = 7.5$, $R = 186$ km), 1992 Big Bear ($M_L = 6.5$, $R = 149$ km), and 1994 Northridge ($M_L = 6.4$, $R = 1.5$ km) earthquake and two of its aftershocks (20 March: $M_L = 5.2$, $R = 1.2$ km; and 6 December, 1994: $M_L = 4.3$, $R = 11$ km). The building is supported by a friction pile foundation. The Northridge earthquake caused severe damage, and the building was declared unsafe. The damage was most severe at the fifth floor, where many columns were damaged, just below the spandrel beam. The specific aspects of the response, which caused this type of failure, have not been deciphered so far. One plausible group of causes can be sought in the large relative deformations of the foundation system (pile caps connected by grade beams; [23]), but the limited number of accelerographs, which recorded the main event, is not sufficient to verify this hypotheses.

In this article, an analysis of ambient noise measurements in the parking lot and on the ground floor of this building is presented. The objective is to describe the deformations of the foundation system during the passage of ambient noise waves (mostly Rayleigh waves caused by surface traffic), and to speculate on how the foundation may have moved during the Northridge earthquake.

2. Description of the building

The building analyzed in this article is a seven-storey reinforced concrete structure, in the city of Van Nuys (Los Angeles metropolitan area), near the intersection of Roscoe Ave. and the San Diego Freeway (I-405; Fig. 1). It will be referred to as VN7SH for short. It was designed in 1965 [1] and served as a hotel until 1994. Its plan dimensions are about 62 by 160 feet (Fig. 2a). 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 the 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. The contribution to the overall stiffness and mass from the non-structural brick filler walls and some of the exterior cement plaster could cause some asymmetry.
for lateral motion in the longitudinal direction, which is expected to be minor.

The first floor is a slab on grade over about 2 ft. of compacted fill. Except for two small areas at the ground floor, covered by one-storey canopies, the plan configurations of the floors, including the roof, are the same. The floor system is a 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. A penthouse with mechanical equipment covers approximately 10 percent of the roof area.

The interior partitions, in general, are gypsum wallboard on metal studs. Cement plaster, 1 inch thick, is used for exterior facing at each end of the building and at the stair and elevator bays on the long side of the building. Double 16 gauge metal studs support the cement plaster. Some additional cement plaster walls are located on the south side of the building at the first floor. The north side of the building, along column line D (Fig. 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 site lies on recent alluvium. A typical boring log (Fig. 2d) shows the underlying soil to be primarily fine sandy silts and silty fine sands. The average shear-wave velocity in the top 30 m is \( \sim 300 \text{ m/s} \). The foundation system (Fig. 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 pile caps are connected by a grid of the 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. The structure is constructed of regular weight reinforced concrete [1]. The structure (~ 63 000 square feet of floor area) was designed in 1965 and constructed in 1966 at a cost of approximately US$1.3 million.

The February 9, 1971 San Fernando earthquake caused minor structural damage. Epoxy was used to repair the spalled concrete of the second floor beam column joints on the north side and east end of the building (the cost of this repair was less than US$2,000). The non-structural damage, however, was extensive and about 80 percent of all repair cost was used to fix the drywall partitions, bathroom tiles and plumbing fixtures. The damage was most
severe on the second and third floors and minimal at the sixth and seventh floors. Forty five bath-tubs and 12 water closets had to be replaced in more than half of the bathrooms [1]. The cost for the repairs was about US$143 000.

The building was severely damaged by the 17 January, 1994, Northridge earthquake, and was not in use on February 4, 1994, when we conducted the first ambient vibrations experiment. 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 fifth floor. Those cracks significantly decreased the axial, moment and shear capacity of the columns. The shear cracks which appeared in the north (D) frame on third and fourth floors, and the damage of columns D2, D3 and D4 on the first 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 foundations. 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.

3. Earthquake recordings

The first known recorded strong motion in the building is of the February 9, 1971, San Fernando earthquake (Fig. 1). The location of the sensors, three self-contained tri-axial AR-240 accelerographs, is shown in Fig. 3a. During this earthquake, the first strong motion waves started to arrive from N22°E, having originated at depth ~9 to 13 km below epicenter (Trifunac, 1974). With rupture propagating up towards south at about 2 km/s, the last direct waves were arriving from N 62°E, 9–10 s later. The 1987 Whittier–Narrows, 1992 Landers and 1992 Big Bear earthquakes occurred at epicentral distances of 41, 186 and 149 km, respectively, and caused strong motion arrivals from E 27°S, East, and E 1.5°S. During the 1994 Northridge earthquake, the first motions started to arrive from the West, with the last arrivals coming from N 42°W, about 7–10 s later [26]; Fig. 1. These latter earthquakes were recorded by a CR-1 system; the sensor locations are shown in Fig. 3b.

Table 1 summarizes selected parameters of the aforementioned earthquakes and accelerograms. The San Fernando accelerogram was digitized manually, at a sampling rate of minimum 50 points per second [4,24]. The accelerograms of the Whittier Narrows, Landers, Big Bear and Northridge earthquakes were processed by the California Division of Mines and Geology. Fig. 4 shows the computed displacements for channels 1 (solid line) and 13 (dashed line) (see Fig. 3b) during the Whittier–Narrows, Landers, Big Bear and Northridge earthquakes and during two aftershocks of Northridge (Table 1). The other earthquakes which triggered the instruments (Table 2) generally resulted in smaller displacement amplitudes and are not presented.

Analyses of the displacement time histories for channels 1, 2, 3 and 13 show that during the larger peaks of the relative response, the torsion within the building contributes 20 to 40 percent of the peak relative response, at the locations of channel 2, for example (Whittier–Narrows ~ 23 percent, Fig. 5; Landers ~ 33 percent, Fig. 6; Big Bear ~ 42 percent, Fig. 7; and Northridge ~ 22 percent, Fig. 8). Comparison of displacement time histories of channels 1 and 13 shows that at the site of channel 1, the peak displacements were up to 10 to 20 percent larger than at the site of channel 13 (~ 20 percent during Landers earthquake, 30–35 s after trigger; ~ 10 percent during Northridge earthquake, 5 to 10 s after trigger, see Fig. 4a and b). It is not likely that these differences were caused by the nature of incident waves. The motions arriving from the Landers earthquake had mainly long period surface waves, propagating from east to west (Fig. 1), while the waves generated by Northridge earthquake were mainly direct nearfield arrivals, propagating predominantly from west to east, with high phase velocities associated with mostly vertical incidence, during the first 3 to 4 s of strongest motion (between 4 and 8 s in Fig. 4b). The amplitudes of the observed differences depend somewhat on the choice of the band-pass filters (shown for each event in Fig. 4a and 4b), but

Table 1
Selected earthquake and accelerogram parameters describing the data used in this work

<table>
<thead>
<tr>
<th>Earthquake</th>
<th>Magnitude $M_L$</th>
<th>Date</th>
<th>Epicentral distance (km)</th>
<th>Azimuth of Arriving strong motion waves</th>
<th>Peak horizontal acceleration (g)</th>
<th>Peak vertical acceleration (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>San Fernando</td>
<td>6.6</td>
<td>9 Feb., 71</td>
<td>22</td>
<td>$22^\circ$–$62^\circ$</td>
<td>0.25</td>
<td>0.17</td>
</tr>
<tr>
<td>Whittier</td>
<td>5.9</td>
<td>1 Oct., 87</td>
<td>41</td>
<td>$117^\circ$</td>
<td>0.16</td>
<td>—</td>
</tr>
<tr>
<td>Landers</td>
<td>$M_L = 7.5$</td>
<td>28 Jun., 92</td>
<td>186</td>
<td>$90^\circ$</td>
<td>0.041</td>
<td>0.007</td>
</tr>
<tr>
<td>Big Bear</td>
<td>6.5</td>
<td>28 Jun., 92</td>
<td>149</td>
<td>$91.5^\circ$</td>
<td>0.01</td>
<td>0.007</td>
</tr>
<tr>
<td>Northridge</td>
<td>6.4</td>
<td>17 Jan., 94</td>
<td>1.5</td>
<td>$240^\circ$–$350^\circ$</td>
<td>0.44</td>
<td>0.27</td>
</tr>
<tr>
<td>Northridge aft.</td>
<td>5.2</td>
<td>20 Mar., 94</td>
<td>1.2</td>
<td>$320^\circ$</td>
<td>0.27</td>
<td>0.10</td>
</tr>
<tr>
<td>Northridge aft.</td>
<td>4.3</td>
<td>6 Dec., 94</td>
<td>10.9</td>
<td>$40^\circ$</td>
<td>0.06</td>
<td>0.03</td>
</tr>
</tbody>
</table>

*Not considered in this analysis; provided for general background only.*
Fig. 4. Displacement time histories for channels 1 (solid lines) and 13 (dashed lines) located at ground floor, at west and east ends of the building respectively for strong motion shaking during (a) Whittier–Narrows, 1987; Landers, 1992 and Big Bear, 1992 earthquakes and (b) Northridge, 1994; and two aftershocks of Northridge earthquake.
occur systematically only during large motions. Further, the differences should be emphasized by EW wave arrivals, because the building is elongated in EW direction. If the observed differences result from separation and relative displacement between the soft soil surrounding the foundation and the piles, it is expected that these differences would be large during Landers and Northridge earthquakes, which both caused large motions at this site, and both arrived predominantly along EW direction. During two aftershocks of Northridge earthquake the waves arrived from north-west and north-east (Fig. 1), and so were less efficient in exciting the torsional response, but their motions were also small (see Fig. 4b).

4. Ambient vibration experiments

4.1. General overview and objectives

Two ambient vibration experiments were conducted in the building, one on Feb. 4–5 (about two and a half weeks after the Northridge main event) and the other one on April 19–20, 1994 (about three months after the main event and one month after one of the largest aftershocks, of March 20, $M = 5.2$; see Fig. 1). Between the two experiments, the building was temporarily restrained, as it was severely damaged by the main event.

The objective of the first experiment was to measure the dynamic characteristics of the damaged building and to see whether the changes in stiffness because of the extensive structural damage could be identified by small amplitude tests. The second experiment was much more detailed. Besides detecting changes in stiffness as a result of new damage from the 20 March aftershock, it also had as an objective to measure the motion of the ground around the building. This was planned to be done by a series of measurements at a dense grid of points in the parking lot of the building. Similar measurements were made during a three-dimensional forced vibration survey of a 9-storey reinforced concrete building [2,9]. The analysis of the amplitudes and phases of the recorded motion confirmed that soil deformed as predicted by theoretical models, and provided an experimental verification of various simplifying assumptions, which usually accompany soil–structure interaction models (e.g., the rigid foundation assumption, and the effects of embedment; [11]).

The aim of the parking lot measurements was to detect ground deformations associated with at least the fundamental

![Fig. 5. Contributions from ground translation, rocking $\theta_y$, and torsion $\theta_z$ to computed displacement at the site of recording channel 2.](image)
transverse and longitudinal modes of vibration. This would have been useful for characterization of soil–structure interaction involving a complex pile foundation. However, no peaks associated with rocking or translation at the apparent frequencies of the building-soil system could be found in the Fourier amplitude spectrum, above the noise level. Nevertheless, the results came out to be even more useful, revealing evidence of flexibility of the foundation and of wave propagation through the first floor slab and the surrounding soil.

For the analysis of this article, the parking lot measurements of the second experiment are of interest, and are presented and analyzed. From the measurements in the building, only the results on the apparent modal frequencies for both experiments are summarized.

4.2. Instrumentation and methods of analysis — second experiment

Four Ranger SS-1 seismometers and two Earth Sciences Rangers were used [6, 21]. The response was measured along frame C (Fig. 2a) at all columns and at each floor,
Three of the SS-1 Ranger seismometers were used to record motion at various locations (the location and orientation were changed as required). The motion of the ground floor was measured at each column and in all three directions (N, E and vertical).

Three reference points were used for calculations of the transfer functions (marked by ‘‘R’’ in Fig. 9). Two of the Earth Sciences Ranger seismometers were placed on the ground floor, at reference locations A5 and D5. Their orientation was always up. The reference instrument for horizontal motions was at location B2 on the ground floor. It was oriented either along the longitudinal (east) or along the transverse (north) direction, depending on the measurement.

The measurements in the parking lot were carried out at 46 locations within 15 to 20 m from the structure, and in three directions (north, east and up; Fig. 9). This was done during daytime, when high direct sun could have contributed to the noise in the soft asphalt surface. To abate this, the instruments were covered with towels. The experiment was carried out continuously from 12 noon of April 19
(Tuesday), until 9 p.m., April 20 (Wednesday) 1994. Those were quiet sunny days (temperature was in the range of 12°C to 25°C). The building was not in use, and except for electricity, other facilities were not available (no elevators, air-conditioning, or running water...). Each of the measurements lasted about 3 minutes, and the sampling frequency was 400 points per second. The PC computer used to record was located on the ground floor. The instruments were placed either directly onto the concrete slab, ceramic tiles, or onto the asphalt, for the outside measurements. Two calibration tests were performed for both horizontal and vertical transducer orientation, one at the beginning and the other one at the end of the experiment. The first test took place on the seventh floor, at the NW corner, near column D1, and the second one on the ground floor, between the frames B and C and 1 and 2.

To describe the overall nature of microtremors in the area surrounding the building, cross-correlation analyses were performed as follows. Measurements were performed by “new” Ranger seismometers (No. 3, 4 and 5; e.g., during run No. 102, see Fig. 9), while the reference instruments (No. 0, 1 and 2) were located inside the building at locations B2 (No. 2), D5 (No. 0) and A5 (No. 1, as shown in Fig. 9). Two “old” Ranger seismometers recorded vertical motion during all measurements, but only transducer No. 1 at A5 was used as reference for the analysis of vertical motions in the parking lot.

The data processing procedure used is described in [6] and will not be repeated here. Here we just mention that the transfer functions of the response were defined with respect to the reference points on the ground floor, mentioned earlier, and that ratios of smoothing $R_s = 2$ and $5$ were used for the horizontal and vertical responses respectively [6].

### 4.3. Results on modal frequencies

The results of the measurements in the building are summarized here only in terms of frequencies and mode shapes for horizontal motion. It was found that in the transverse (N–S) direction the soil–structure system vibrates with frequencies $f \approx 1.4, 1.6, 4.2$ and 4.9 Hz (Fig. 10, top). In the longitudinal direction, the apparent frequencies were at $f \approx 1.1, 3.7, 5.7$ and 8.5 Hz (Fig. 10, bottom). Detailed description of the mode shapes and of other aspects of the response is outside the scope of this article. Fig. 10 serves only to suggest the overall characteristics of the transfer functions for horizontal motions and to provide a general background for the analysis of the foundation response.

Tables 3 and 4 summarize the results on the apparent modal frequencies for both experiments, for the longitudinal (EW) and transverse (NS) directions. It is seen from Table 3...
that three out of the four identified frequencies in the longitudinal direction were larger during the second experiment, while one (f = 5.7 Hz), remained the same. The increase in frequency most probably resulted from the wooden braces restraining the building, placed at the longitudinal frames between the two experiments. The frequency of the first longitudinal mode increased by 10 percent, and of the second and fourth longitudinal modes by 6 and 5 percent. Apparently the restrainers did not affect the third mode. From Table 4, it is seen that the frequency of the first transverse mode and of the first torsional mode are the same (apparently, the braces located along the longitudinal frames, did not increase stiffness for those two modes), but of the third transverse mode had frequency larger by 10 percent for the second experiment.

4.4. Results on motion of the ground floor and of the surrounding soil

4.4.1. General characteristics

Fig. 11 shows average Fourier amplitude spectra of NS, EW and vertical components of motions in the parking lot (averaging was done to emphasize the predominant wave motion and to reduce the local noise). The average spectra were obtained from three runs at a group of three sites, located north, east, south and west of the building (i.e. total of 12 locations, highlighted in Fig. 9 by cross-hatched schematic representation of the recording instruments). It is seen that there are many large amplitude peaks in the spectra. In most cases, these do not coincide with identified apparent modal frequencies of the building (shown by solid, open and shaded bars in Fig. 11), and were created by strong periodic sources in San Fernando Valley (industrial sites with large moving machinery). The overall large amplitudes for frequencies centered near 4 Hz were caused by the NS traffic on I-405 (San Diego Freeway, ~150 m west of the building) and by EW traffic on Roscoe Boulevard (~50 m north of the building).

Fig. 12 shows a typical segment of vertical ground velocity recorded south of the building (~40 feet south of column A1, see Fig. 9). The amplitudes and spectral character of the measured ambient noise change during the day depending mostly on the density and average speed of vehicular NS traffic on San Diego Freeway and on EW traffic on Roscoe Boulevard (~50 m north of the building). We computed the peak amplitudes of the autocorrelation functions for τ = 0 (zero lag) corresponding to the recordings at the reference stations (No. 1 for vertical motions and No. 2 for NS and EW motions) for recording sessions 79–143 in the parking lot. An example of those is shown in Fig. 13 for the NS motion ("total" data in this figures implies that the record was not filtered). It is seen that the amplitudes of the noise peaked at ~9 AM. and then decreased towards a minimum at 3–5 PM.

4.4.2. Cross-correlation functions

The cross-correlation function, \( R_{\text{ref}} (\tau) \), was computed

<table>
<thead>
<tr>
<th>Mode shapes</th>
<th>f - Hz</th>
<th>Expl. I</th>
<th>Expl. II</th>
<th>Δf - %</th>
</tr>
</thead>
<tbody>
<tr>
<td>EW</td>
<td>1.0</td>
<td>1.1</td>
<td>10</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3.5</td>
<td>3.7</td>
<td>6</td>
<td></td>
</tr>
<tr>
<td></td>
<td>5.7</td>
<td>5.7</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>8.1</td>
<td>8.5</td>
<td>5</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Mode shapes</th>
<th>f - Hz</th>
<th>Expl. I</th>
<th>Expl. II</th>
<th>Δf - %</th>
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<tbody>
<tr>
<td>NS</td>
<td>1.4</td>
<td>1.4</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.6</td>
<td>1.6</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3.9</td>
<td>4.2</td>
<td>10</td>
<td></td>
</tr>
<tr>
<td></td>
<td>4.9</td>
<td>4.9</td>
<td>0</td>
<td></td>
</tr>
</tbody>
</table>
for each location, defined by
\[ R_{i, \text{ref}}(\tau) = \frac{1}{T} \int_{T}^{T/2} f_i(t) f_{\text{ref}}(t + \tau) dt \]  
where \( f_i(t) \) is the motion at the \( i \)th location and \( f_{\text{ref}}(t) \) is the simultaneous record at the reference point (B2 for EW and NS motions, and A5 for vertical motions, see Fig. 9) and \( T \sim 3 \) min. Then, the spatial distribution of the peak amplitude, \( R_{i, \text{ref}}(\tau)_{\text{max}} \), and corresponding time lag, \( \tau \), were plotted and analyzed. Parts (a) of Figs 14–16 show the measuring locations, transducer orientations and recording session numbers. Parts (b) of the same figures show contour plots of the peak amplitudes of \( R_{i, \text{ref}}(\tau) \) (solid line) and of the relative delay \( \tau \) (dotted lines). The amplitudes are on an arbitrary scale, but consistent for the NS, EW and vertical directions, and \( \tau \) is in seconds. In Fig. 14b, 30 feet (\( \sim 10 \) m) west of the building, \( \tau \sim 0.03 \) s. This corresponds to apparent horizontal phase velocity of about 300 m/s, consistent with the interpretation that microtremors are high frequency Rayleigh waves propagating through shallow soil layers. The overall pattern of the time lag, \( \tau \), implies wave arrival from the west, and scattering and diffraction around the building foundation. The corresponding contours for vertical motion (Fig. 15b) imply wave arrival from west and south-west, with apparent phase velocities between 250 and 300 m/s. The corresponding contours for the EW motions are shown in Fig. 16b.

The results in Figs. 14–16 have been evaluated using unfiltered recordings. As it can be seen from Fig. 11, this motion has most of its energy between 3 and 6 Hz. The recorded signal is small between 1 and 1.5 Hz, where the motions of the soil driven by soil–structure interaction are expected to be seen at frequencies near the horizontal and vertical apparent frequencies of the building (\( \sim 1.0 \) Hz for EW and \( \sim 1.4 \) Hz for NS and vertical motions). To analyze the motions near these frequencies, the signals were first band-pass filtered, using a “cosine bell” function centered at 1.0 Hz for EW motions and at 1.4 Hz for NS and vertical motions, and 0.6 Hz wide. The results are shown in Figs. 17–19. It is seen that while the NS and vertical motions are consistent with the overall propagation of energy from west to east, the contours of \( \tau \) in Fig. 19 are complex and difficult to interpret. The contours of the amplitudes of \( R_{i, \text{ref}}(\tau)_{\text{max}} \) imply strong warping of the building foundation and of the parking lot, while slowly decaying motions with same relative phase away from the building would be expected from soil–structure interaction effects based on rigid foundation modeling [2,9].

To eliminate the consequences of amplitude variations of the ambient noise with the time of the day (see Fig. 13), the cross-correlation functions in Figs. 14–19 were normalized by \( R_{1,1}(0) \) or \( R_{2,2}(0) \) for the unfiltered record and for band-pass filtered data, for all measurement runs (runs 79–143). The variations in amplitude and phase caused by fluctuations in the direction of approach of the ambient noise cannot easily be accounted for by simple normalization, and the associated effects were not corrected for in the presented results.

Seismometers No. 2, 3, 4 and 5 are “new” Ranger seismometers and have essentially same instrument characteristics, while No. 0 and 1 are “old” Ranger seismometers and have different characteristics (see Ref. [6]). Consequently \( R_{i, \text{ref}}(\tau) \sim 1 \) and \( \tau \sim 0 \) for \( R_{3,2}(\tau) \) and \( R_{5,2}(\tau) \) evaluated for same transducer orientation and essentially same recording point, while \( (R_{i, \text{ref}}(\tau))_{\text{max}} \neq 1 \) for \( R_{3,1}(\tau) \), \( R_{4,1}(\tau) \) and \( R_{5,1}(\tau) \). These differences were all factored into the relative normalization of all the amplitudes shown in Figs. 14–19.

This 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”
Fig. 14. (a) Position of instruments during the recording of the NS response in the parking lot of the building, top view. Arrows show orientations of the transducers. Numbers show the "run numbers," while the subscripts identify transducer numbers (3, 4 and 5). (b) Contours of $(R_{ref}(\tau))_{max}$ for NS motion (arbitrary normalized amplitudes, shown by heavy lines) and $(\tau)$ (in seconds) of $(R_{ref}(\tau))_{max}$ relative to the reference station at B2.
Fig. 15. (a) Position of instruments during the recording of the vertical response in the parking lot of the building, top view. All transducers were oriented ‘‘UP’’. Numbers show the ‘‘run numbers’’, while the subscripts identify transducer number (3, 4 and 5). (b) Contours of \((R_{\text{ref}}(\tau))_{\text{max}}\) for vertical motion (arbitrary normalized amplitudes, shown by heavy lines) and \((\tau)\) (in seconds) of \((R_{\text{ref}}(\tau))_{\text{max}}\) relative to the reference station at A5.
Fig. 16. (a) Position of instruments during the recording of the EW response in the parking lot of the building, top view, second experiment. Arrows show orientations of the transducers. Numbers show the ‘‘run numbers’’, while the subscripts identify transducer number (3, 4 and 5). (b) Contours of \((R_{rel}(\tau))_{\text{max}}\) for EW motion (arbitrary normalized amplitudes, shown by heavy lines) and \((\tau)\) (in seconds) of \((R_{rel}(\tau))_{\text{max}}\) relative to the reference station at B2.
Fig. 17. Contours of \( (R_{i,\text{ref}}(\tau))_{\text{max}} \) for band-pass filtered NS motion (arbitrary normalized amplitudes, shown by heavy lines) and \( (\tau) \) (in seconds) of \( (R_{i,\text{ref}}(\tau))_{\text{max}} \) relative to the reference station at B2.

Fig. 18. Contours of \( (R_{i,\text{ref}}(\tau))_{\text{max}} \) for band-pass filtered vertical motion (arbitrary normalized amplitudes, shown by heavy lines) and \( (\tau) \) (in seconds) of \( (R_{i,\text{ref}}(\tau))_{\text{max}} \) relative to the reference station at A5.
between the vertical walls of the building and soil, and piles and the surrounding soil to have been ‘‘recemented’’. It may be that what we see in the aforementioned measurements is 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 so irregular. Of course, this is further complicated by apparent arrival of wave energy from different directions, though mainly from moving sources on a major freeway just 150 m west of the site (NS vehicular traffic on I-405).  

5. Discussion and conclusions

One of the more interesting results of this analysis is seen in Fig. 14b, displaying normalized amplitudes of the cross-correlation function of NS velocities for the complete (unfiltered) recorded motions. It shows that during passage of microtremor waves, mainly from west to east, the foundation essentially rotates about a point close to the south-eastern corner of the building (near A9). The EW components of this motion, shown in Fig. 16b, are consistent with this interpretation if one allows for some in-plane deformation of the foundation system, in the north and west ends of the building. If present during strong motion, this would imply very large eccentricities of torsional stiffness of the overall foundation, and consequently strong coupling of the NS and torsional components of response.

We speculate that during very strong motion, the soil is pushed sideways by large relative response of the foundation and piles, in this case more along the west end of the building. The width of this separation probably closes partially during ‘‘dynamic compaction’’ effected by many small aftershocks. Therefore, the pattern of recorded ambient noise amplitudes, shown in Fig. 14b, and our interpretation could also depend on the status of this separation during the time of our experiment.

The described mechanism acts as a powerful passive energy absorption system, but analyses of its nonlinear, time dependent behavior and complex soil–structure interaction analyses would be very difficult. Evaluation of its effects on the dynamics of the system would require analyses in which the geometric characteristics experience large changes during the duration of the excitation. Analyses of such problems are possible, but it is helpful first to learn more about the expected nature of the changes with time from full scale observations during actual earthquakes.

The aforementioned observations could result from non-uniform soil properties below the foundation or from partial shear failure of several piles (probably as early as in 1971 during the San Fernando earthquake), resulting in ‘‘softer’’ soil–pile system below the western end of the building. Such variations in stiffness must be included in the response...
analyses which should explicitly address the strong coupling between translation and torsion. Several repeated full scale tests of the as-built structure would have detected the range of actual variations of these centers of stiffness.

The measuring grid shown in Fig. 9 was not sufficiently dense to determine whether the side walls of the building and the soil moved as a continuum or independently. Most horizontal displacement contours are consistent with the assumption that these two are in contact. The contours of vertical motions, however, suggest that some separation may be present, for example, near the north-western corner of the building (Fig. 15b).

All the contour plots of horizontal and vertical amplitudes of deformation of the ground floor show that the foundation of this building did not act as a rigid body, but it deformed with the passage of incident waves. The grade beams allowed differential vertical (Fig. 15b; Fig. 19) and horizontal motions (Figs. 14b, 16b, 17 and 19), which followed the deformation of the “body” of soil with piles. In this example this “body” is stiffer than the surrounding soil by a factor perhaps as large as two, because the velocity of NS displacements is roughly 40 percent higher over the ground floor than outside the building (see the dashed lines representing the relative phases of motion in Figs. 14b and 17). Consequently, in addition to the inertial forces, the differential motions at the base of the first story columns would contribute additional moments and shears to the superstructure. The actual amplitudes of these additional effects can be calculated by numerical modeling, but conservative estimates of their upper bounds can be obtained by assuming that the soil–pile-foundation system has same stiffness as the surrounding soil, and that there is no soil–structure interaction [23,25].

Albeit speculative and qualitative, the considerations taken into account earlier show that the state of the art of strong motion instrumentation of buildings is not adequate to address most of the aspects of the problems we considered. Additional recording channels on the ground floor would have provided invaluable information. Analyses of the damage in this building is outside the scope of this article, but it could be shown that additional instruments were called for at the upper floors as well. Therefore, with limited resources, we should explore what is better: to instrument in more detail selected geometrically simple buildings, or to continue with the present programs which instrument many buildings, but with limited instrumentation in each building. At present, while this decision is made, the emphasis should be placed on processing and dissemination of all recorded accelerations in structures, so that small (linear) and large nonlinear motions can be analyzed and compared. For identification of the soil–structure system, all recorded motions are valuable, even the very small ones, particularly when those contribute to the database of simple and symmetric buildings, for which most of our analysis tools should be applicable.

During the last 20 years, the emphasis has been placed on laboratory experiments, while the full-scale tests of structures have been neglected. The best and truly informative experiments are the full-scale tests in actual buildings. In the laboratory, we mimic imperfectly our hypotheses and expectations. Only the full scale observations of the actual nature can unveil our misconceptions and occasionally provide a basis for better understanding and for creation of new theories and ideas.

References


[6] Ivanović S, Trifunac MD. Ambient vibration survey of full scale structures using personal computers (with examples in Kaprielian Hall), Dept. of Civil Engng, Report No. CE 95-05, Univ. of Southern California, Los Angeles, California, 1995.


waves, Dept. of Civil Engng, Report No. CE 90-01, Univ. of Southern California, Los Angeles, California.


