SEISMIC RESPONSE OF AN INSTRUMENTED REINFORCED CONCRETE BUILDING FOUNDED ON PILES

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SUMMARY

The subject of this work is the seismic response of an instrumented seven-story reinforced concrete hotel located in San Fernando Valley of the Los Angeles metropolitan area. Analysis is presented of the building response to (1) nine earthquakes and three aftershocks (between February 1971 and December 1994), and (2) ambient noise recorded during two ambient vibration tests following 1994 Northridge earthquake. This building, supported by concrete friction piles and founded on a loose soil, was damaged by two of these earthquakes (1971 San Fernando and 1994 Northridge earthquake). Short and long term changes in non-linear response of the building-foundation-soil system were studied by moving-window and zero-crossing analyses to detect changes in the predominant system frequency. The response of the system is dominated by soil-structure interaction, which is manifested mainly through rocking in longitudinal and transverse directions. The rocking response and the relative building response were extracted approximately from the total system response by low-pass and high-pass filtering. The results showed progressive decrease in the system frequency, proportional to the intensity of motion, but the reduction in soil "stiffness" was not permanent. The frequency of the soil-structure system did return to its original value, when there was no permanent damage of the building. Based on the limited data of the Northridge earthquake, which we analyzed, "healing" of the soil appears to have occurred, resulting from dynamic consolidation caused by the many aftershocks. Ambient vibration data were used to estimate the system frequencies for low levels of excitation.

The nonlinear soil behavior was interpreted to be a significant factor influencing the system response for essentially all the analyzed strong motion events. This has been very beneficial for this building, because the soil dissipates energy via nonlinear deformations, reducing the energy available to excite the structure. If founded on more rigid soil, this building would have suffered more severe damage during the Northridge earthquake.

INTRODUCTION

Earthquake resistant design of structure must be based on analyses of realistic models of the structure, foundation and soil system, considering all aspects of nonlinear response. This is best accomplished by careful experimental verification using full-scale tests of actual structures and from observation of the performance of full-scale structures during severe earthquake shacking [11].

In this paper, an instrumented seven-story hotel building, located in Van Nuys, California is studied. The building, founded on the loose soil, is supported by friction pile foundation [1]. Records of twelve earthquakes were available for the study [12]. The purpose of the analyses was to detect, from available earthquake records, changes in the system (building-foundation-soil) response. Nonlinear effects depend on the level of the excitation (function of time) and also on the initial state of the system (e.g., the state of the soil such as degree of consolidation, water content

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e.t.c.), and when damage occurs permanently change the system [3]. Short and long term changes in non--linear response were studied by moving--window and zero-crossing analyses of individual records (to detect changes in the predominant system frequency), for each earthquake and by comparison of results of the former for different earthquakes, in some cases months and in others years apart.

DESCRIPTION OF THE BUILDING

The analyzed building is a seven-story reinforced concrete structure, located in the city of Van Nuys (Los Angeles metropolitan area). Building was design in 1965 [1] and served as a hotel until it was damaged by 1994 Northridge earthquake. Its plan dimensions are about 18.3 by 45.7 meters (Figure 1). The typical framing consists of columns spaced at 5.7 meter centers in the longitudinal direction and 6.1 meter centers in the longitudinal direction. Spandrel beams surround the perimeter of the structure. The interior column slab frames and exterior column spandrel beam frames resist lateral forces in each direction. The added stiffness in the exterior frames 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 of the overall stiffness and mass from the nonstructural brick filler walls and some of the exterior cement plaster could cause some asymmetry for lateral motion in the longitudinal direction. The floor system is a reinforced concrete flat slab, 25.4 cm thick at the first floor, 21.6 cm thick at the third to seventh floors and 20.3 cm thick at the roof.

The site lies on the recent alluvium. A typical boring log shows the underlying soil to be primarily fine sandy silts and silty fine sands. The foundation system, Figure 1, consists of 96.5 cm deep pile caps, supported by groups of two to four poured in place 61.0 cm diameter reinforced concrete friction piles. These are centered under the main building columns. The grid of the beams connects all pile caps. Each pile is roughly 12.2 m long and has a design capacity of over 45,500 kg vertical load and up to 9,100 kg lateral load.

The structure is constructed of regular weight reinforced concrete. The structure (~5860 square meters of floor area) was designed in 1965 and constructed in 1966 at a coast of approximately US$1.3 million

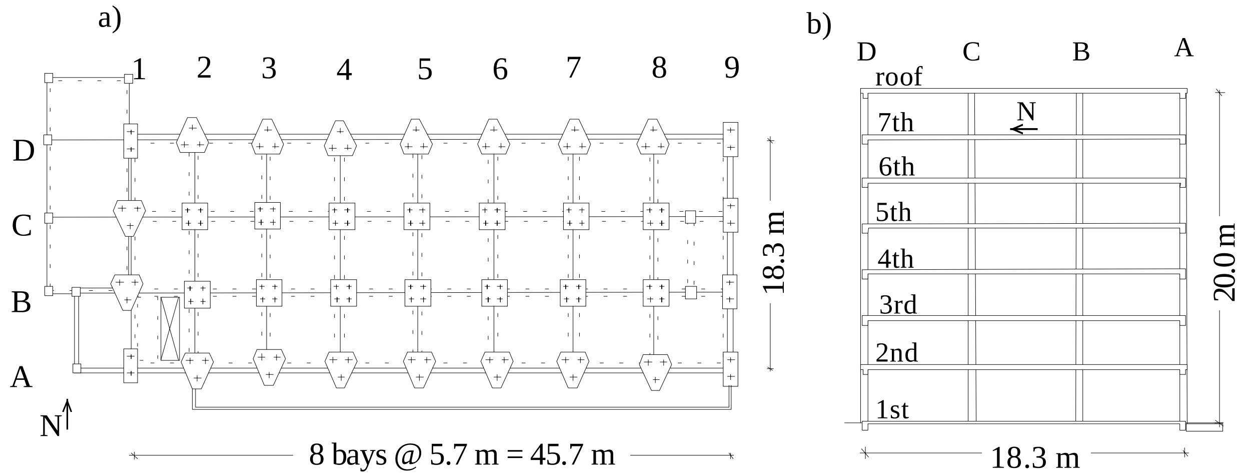

Figure 1: (a) Foundation plan, (b) Typical transverse section

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DESCRIPTION OF DAMAGE CAUSED BY SAN FERNANDO AND NORTHRIDGE EARTHQUAKES

The February 9, 1971 San Fernando earthquake [10] caused minor structural damage. Epoxy was used to repair the spelled concrete of the second floor beam column joints on the north side and the east end of the building (the cost of this repair was less then US$2,000). The non-structural damage was extensive and about 80 percent of the all repair cost was used to fix the drywall partitions, bathroom tiles and plumbing fixtures. The cost of the repairs was about US$ 143,000.

The building was severely damaged again by the 17 January 1994 Northridge earthquake [12]. The structural damage was extensive on 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 at the 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 on the north (D) frame, on the third and fourth floor, and the damage of columns D2, D3, and D4 on the first floor caused minor to moderate change in 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 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.

AMBIENT VIBRATION EXPERIMENTS

Two ambient vibration experiments [2,4] were conducted in the building, one on Feb. 4-5 (about two and a half weeks after the Northridge earthquake) and the second on the 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). Between the two experiments [12], 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 the stiffness caused by the extensive damage could be identified by small amplitude tests. The second experiment detected changes in stiffness as result of new damage from the 20 March aftershock.

Four Ranger SS-1 seismometers and two Earth Science Rangers were used. The response was measured along longitudinal frame C at all columns and at each floor, for all three components of motion. Results in terms of mode shapes and apparent frequencies for EW and NS responses are as follows. For the first experiment, frequencies for longitudinal (EW) response are f_1=1.0Hz, f_2=3.5Hz, f_3=5.7Hz, f_4=8.1Hz, and for the transverse, (NS) response, frequencies are f_1=1.4Hz, f_2=1.6Hz (first trorsional mode), f_3=3.9Hz, f_4=4.9Hz (second torsional mode). During the second experiment, frequencies for longitudinal, (EW) response were f_1=1.1Hz, f_2=3.7Hz, f_3=5.7Hz, f_4=8.5Hz, and for the transverse (NS) response frequencies were f_1=1.4Hz, f_2=1.6Hz (first torsional mode), f_3=4.2Hz, f_4=4.9Hz (second torsion mode).

It is seen that three out of four identified frequencies in the longitudinal direction were larger during the second experiment, while one (f=5.7Hz), reminded the same. The increase in frequency most probably resulted from the wooden braces restraining the building, placed into 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 restrainers didn't affect the third mode. It is seen that the frequency of the first transverse mode and of first torsion mode are the same, for both experiments. Apparently braces located along the longitudinal frames did not increase stiffness for those two modes. The third transverse mode had frequency larger by 10 percent during the second experiment.

EARTHQUAKE RECORDINGS

The first known strong motion records in the building date back to the 1971 San Fernando earthquake [10]. Then the building had only three self contained triaxial AR-240 accelerographs, one at the south-eastern corner at the ground level, one in the center of the fourth floor, and one in the south-western corner, on the roof [12].

The central recording system (CR-1; Fig. 2) was installed in the building prior to 1987 Whittier-Narrows earthquake. It is operated by California Division of Mines and Geology (CDMG). Between 1987 and 1994, the CR-1 system was triggered by many local and distant events. recorded accelerations only from the 1987 Whittier-Narrows (M_L=5.9), 1992 Landers (M_s=7.5), 1992 Big Bear (M_L=6.5) and 1994 Northridge (M_L=6.4) events, at epicentral distances 41, 186, 149 and 1.5 km respectively. The largest accelerations were those recorded during the 1994 Northridge earthquake [12].

NONLINEAR RESPONSE

Fig. 3a shows breakdown of the contributions to the recorded motion in the building on the flexible soil. The total horizontal motion, at j-th floor, U_{Tj} results from:

U_g, horizontal displacement of ground caused by passage of earthquake waves
U_o, local deformation of soil due to soil structure interaction,
$h_j\Phi_o$, horizontal motion caused by rocking of the foundation by angle Φ_o,
U_j, deformation of j-th floor relative to a moving coordinate system, attached to the foundation.

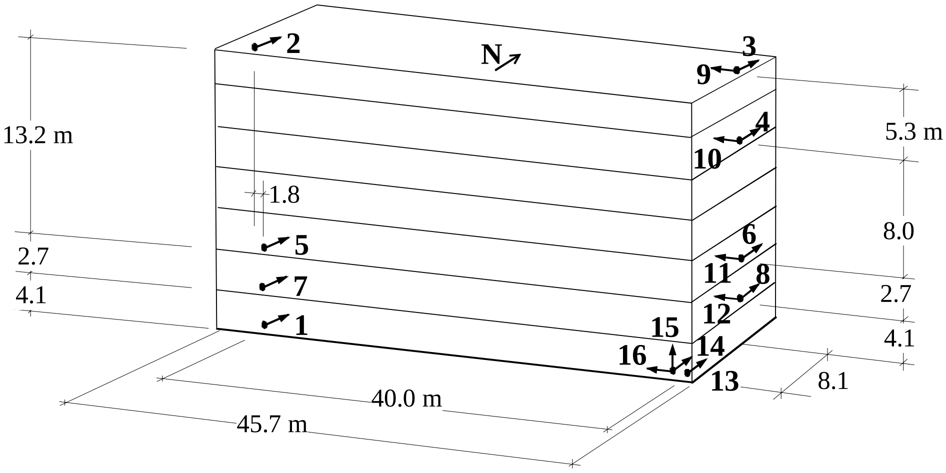

Figure 2: Location and orientation of 13 sensitivity vectors of CR-1 recording system. Channels 14, 15 and 16 belong to SMA-1 accelerograph

With horizontal transducers at 1st, 2nd, 3rd, 6th, and 8th (roof) floors, and with only one vertical transducer (near south east corner of the building), we cannot separate contributions to the total recorded motion, from the rocking Φ_o, and from the relative displacement response U_j. At least two vertical transducers near north and south ends of the first floor would have provided this information [5-8]. For the building analysis it is necessary to separate these contributions. First we assume that relative deformations of the structure U_j can be described approximately by the mode shapes of the fixed base building [13]. For NS motions the lowest (first mode) natural frequency is close to 1.4 Hz. During the earthquake, the transfer function of NS motion is dominated by a peak in the in the range from 0.5Hz to 0.9Hz, which we interpret to result mainly from the rocking motion Φ_o. To separate this rocking from the relative displacement, approximately, we low pass filter the data to obtain predominately the contributions from relative deformations u_j. For NS response we choose the transition zone (ramp) for those filters to go from 0.8 to 1.0 Hz.

It is found that response of this system, due to soil-structure interaction, is manifested through rocking in longitudinal and transverse directions and through the torsion, but in this paper, only rocking in the transverse NS, direction will be illustrated.

Figure 3b shows the transfer function for rocking in the transverse (NS) direction $\ddot{\theta}_y(t)$, computed from accelerations. To evaluate the Fourier transforms of "rocking"

$$\ddot{\theta}_y(t) = \left(\ddot{v}_3(t) + \ddot{v}_2(t) - \ddot{v}_1(t) - \ddot{v}_{13}(t)\right)/\,2H$$

is first computed and then the Fourier Transform of $\ddot{\theta}_y(t)$ is calculated. Figure 3b shows results only for the frequencies less then 1.6Hz. Spectral amplitudes for the frequencies beyond 1.6Hz are small and there are no identifiable peaks. The fixed base modes of vibration with displacements in NS direction (transverse mode) have frequencies near 1.4, 2.6 (first torsion), 3.9 and 4.9 (second torsion) Hz. Spectral amplitudes in Figure 3b show that the peaks move towards lower frequencies, as the amplitudes of the transfer function increase, indicating the presence of the softening type of non linearity somewhere in the system.

Total horizontal displacement of a point *j* on the building can be represented by superposition of U_g, U_o, $h_j\Phi_o$ and U_j. Representing this n-degree of freedom system by an equivalent single degree of freedom system (e.g. assuming that only the first mode of vibration dominates in the relative response of the structure), the apparent frequency of soil structure system, $\tilde{\omega} = 2\pi\tilde{f}$, can be approximated by [5,11]

$$\frac{1}{\tilde{\omega}^2} = \frac{1}{\omega_1^2} + \frac{1}{\omega_R^2} + \frac{1}{\omega_H^2}$$

where, ω_1 is the frequency of vibration of the equivalent single degree of freedom system, and ω_R and ω_H are the rocking and horizontal frequencies of vibration of the rigid structure on elastic soil.

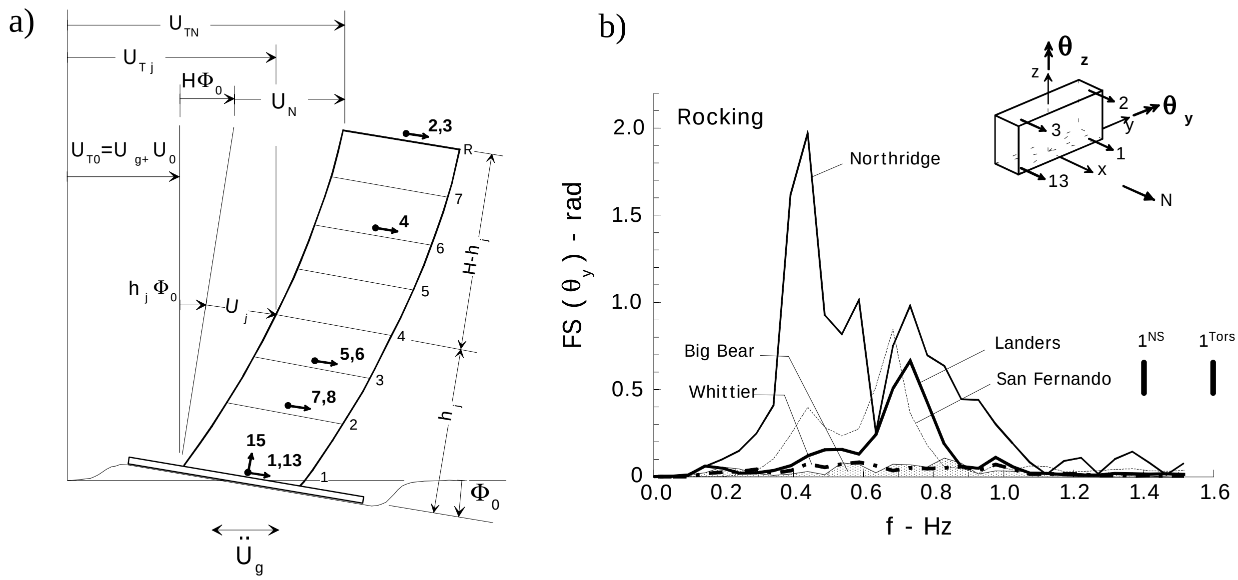

Figure 3: a) Deformation of the building due to relative deformation U_j, horizontal deformation Uo and rocking Φ_o of foundation soil. b) Fourier amplitude spectra of rocking $\ddot{\theta}_y(t)$ acceleration

Figure 4 summarizes the results of moving window and of the zero crossing analyses for NS vibrations during San Fernando, Whittier-Narrows, Landers, Big Bear and Northridge earthquakes. For moving window Fourier analysis, we worked with corrected data with equally spaced points at $\Delta t = 0.02$ sec (San Fernando and Northridge data). We used 8 sec long windows as follows: 1. ramp up, 1.0 sec (50 points), 2. Unit amplitude for 6.0 sec (300 pts), 3. ramp down, 1.0 sec (50 pts) followed by 112 zeros. For other earthquakes we worked with corrected accelerograms with $\Delta t = 0.01$ sec and 4 second long window (ramp up 50 pts, unit amplitude 300 pts, ramp down 50 pts). To perform zero crossing analyses we low-pass filtered $\theta_y(t)$ by a band pass filter, with pass band from 0.1-0.2 Hz to 0.8-1.0 using Ormsby filters. Then we read manually the half periods for all approximately symmetric peaks, and computed and plotted the corresponding "period" of vibration versus time for all peaks. This is shown by open circles in Figure 4. To suggest possible range of values and behavior of $\theta_y(t)$, we present combination of torsion and rocking, for San Fernando data, by dashed lines.

The moving window analyses in Fig. 4. starts "late" and finish "early" (2 or 4 seconds), depending on the window length used in the analyses (4 or 8 seconds), while the zero crossing analyses produce reliable data only during "large" response amplitudes. Together, these two analyses give rough, but consistent, description of the changes of $\bar{f}$ versus time. Starting with Wittier-Narrows earthquake, it appears that $\bar{f}$ decreases whit each additional earthquake, but this results from the fact, that strong motion amplitudes increase from Whittier-Narrows, to Landers to Northridge earthquakes. There was no reported damage in the building following Whittier-narrows and Landers earthquakes [12].

Two ambient vibration experiments, performed in 1967 and 1971, before and after San Fernando earthquake [4] gave only limited information about these measurements and so we do not know the detailed nature of the changes in the frequencies of this building with time, immediately following San Fernando 1971, earthquake. Related studies of Milikan Library [5,9] have indicated that in the months following earthquakes, apparent frequency of the system appears to begin to recover, but no regular periodic tests were conducted to find what this trend might be over long periods of time. For this building, comparison of post earthquake apparent frequencies, $\bar{f}$, suggests almost complete "recovery" which may be accelerated by the occurrence of aftershocks.

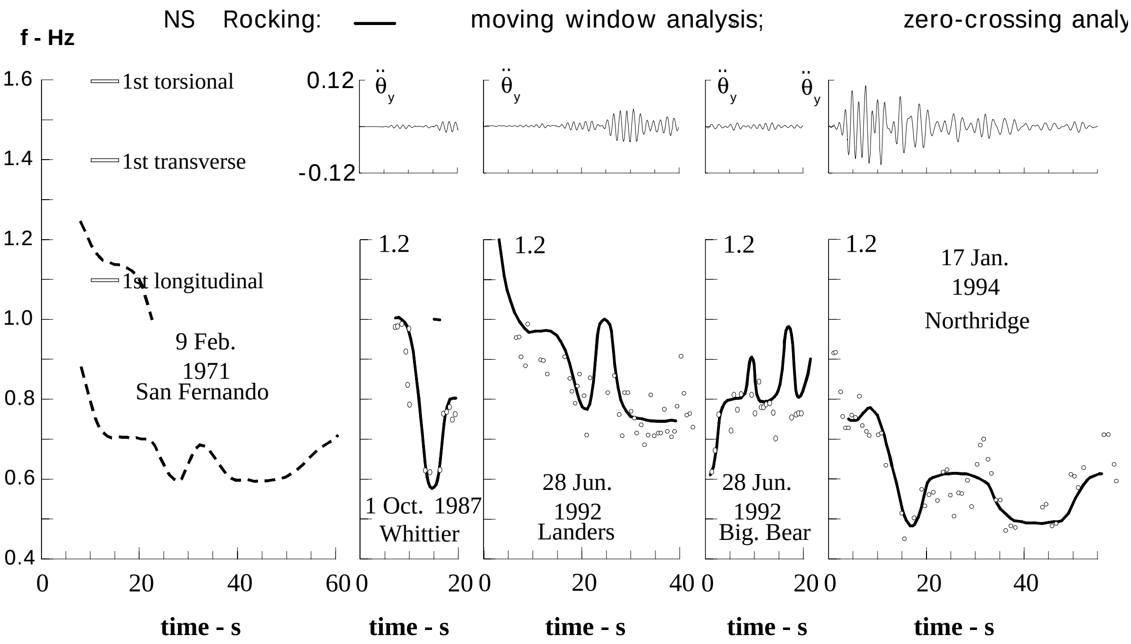

Figure 4: Moving window Fourier analyses (solid lines) and zero crossing analyses (open circles) of NS (transverse) rocking acceleration $\ddot{\theta}_y$

CONCLUSOINS

Figure 5 summarizes all observed changes in NS rocking frequencies, starting with San Fernando earthquake in 1971 and ending with December, 1994 aftershock of Northridge earthquake. The trends in Fig. 5 are interesting and challenging to interpret. Perusal of Fig. 4 would suggest that consecutive earthquakes progressively weaken the soil-structure system. However, inclusion of data on other intermediate earthquakes shows entirely different picture. What is "loosened" by severe strong motion rocking, appears to be "strengthened" by aftershocks and by intermediate smaller earthquakes. Aftershocks of both Whittier-Narrows, 1987 (event No. 3) and Northridge, 1994 earthquake (events No. 11 and 12) lead to higher apparent frequencies. It seems that this may be associated with "dynamic settlement and compaction". Analysis and physical modeling of these results is beyond the scope of this work. Here we focus on the detailed compilation and of interpretation of obvious trends only.

The above trends can be explained as follows. For small "initial" vibration as building begins to "push" soil sideways, its effective depth of "fixity" moves up and down (see d_{eq} in Fig. 6; d_{eq} is a function of time and of response amplitudes and history), which results in nonlinear response which can be represented by a stiffening spring. As the motions build up the soil yields and the equivalent stiffness K becomes analogous to a softening spring. With reversal of motion and loss of contact between the building and the soil d_{eq} increase and K returns towards hardening spring behavior.

This cycle repeats as long as the soil can be pushed sideways and the vibration amplitude increases (see Fig. 6a). Having reached the largest rotations, $\theta_y(t)$, during any given earthquake, or a burst of energy in strong motion of long strong ground motion, and having caused non-linear soil deformation near corners of the "rigid" foundation, the subsequent vibrations are next characterized by smaller displacements and longer periods of motion (because reduced L_{eq} has lengthened the period of motion, (Fig. 6b).

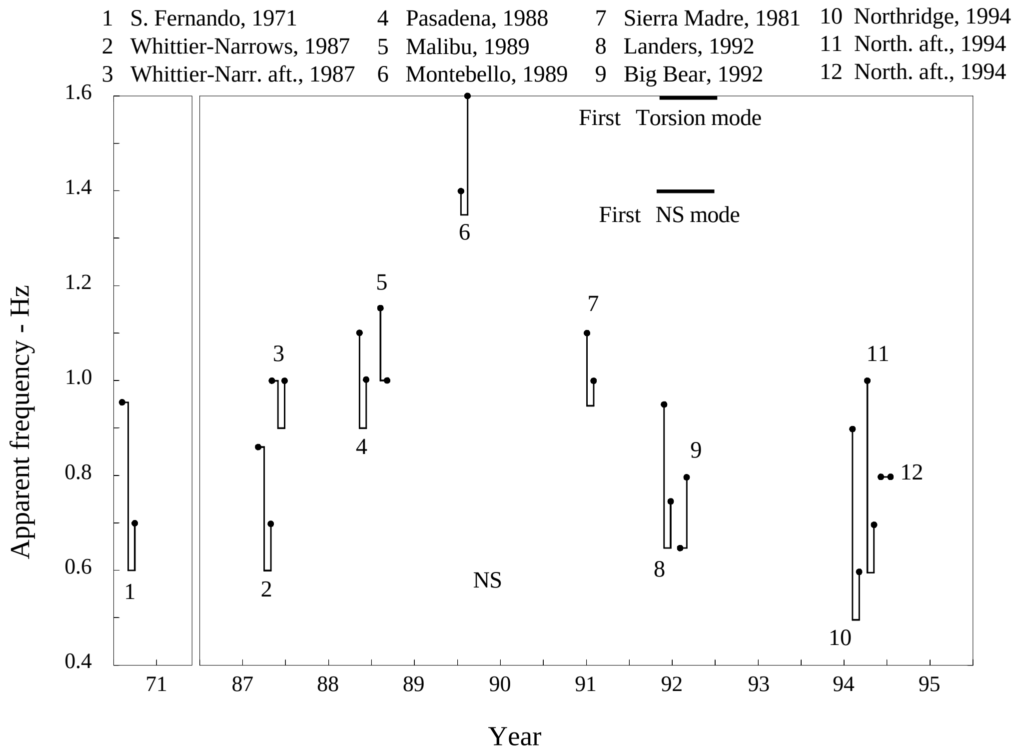

Figure 5: Schematic summary of the changes of predominant system frequency for NS direction, derived from moving window and zero-crossing analyses

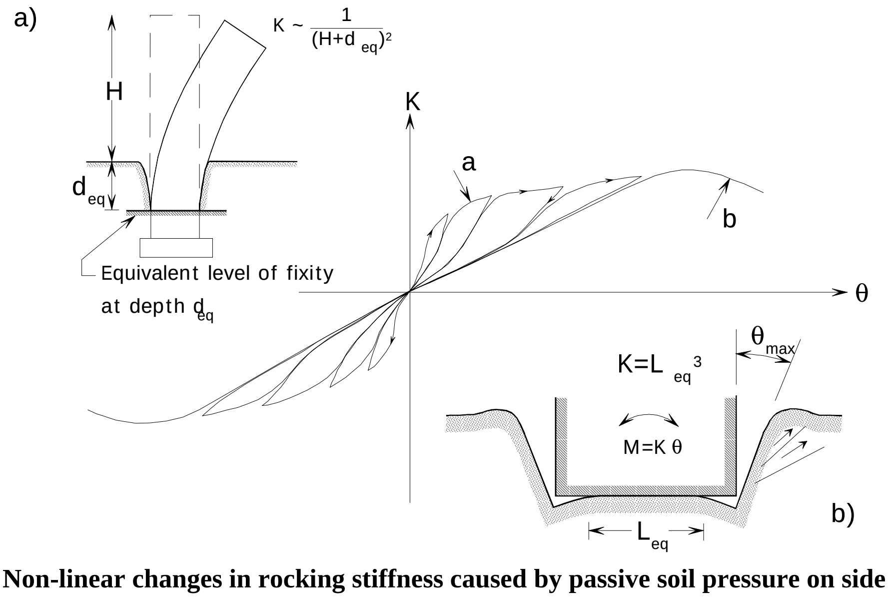

Figure 6: a) Non-linear changes in rocking stiffness caused by passive soil pressure on side walls of the building and variable depth of fixity d_{eq} . b) Schematic representation of "permanent soil deformation after large rocking response

The role of aftershocks and of ``small" earthquakes, following those with strong motion at this site, is potentially most interesting and unexpected. With ``small" amplitudes of motion and possibly higher frequency and shorter excitations these events may be acting as a spontaneous ``healing agents", which, through dynamic compaction and settlement of the material which was loosened and pushed aside by proceeding strong motion, soil is packed back around piles, grade beams, and sides of the building, rebuilding or even exceeding the original system stiffness. It seems that this cycle may be repeated many times, depending on the sequence of aftershocks and small earthquakes during ``quiet" intervals between ``strong motion" events. Following Northridge earthquake the building was repaired and strengthened, thus altering permanently it's structural stiffness characteristics. Its pile foundation system however was not altered. Therefore continued monitoring and recording of future earthquake motions there will contribute new and additional lessons to this interesting case.

The nonlinear soil behavior was interpreted to be a significant factor influencing the system response for essentially all the analyzed strong motion events. This has been beneficial for this building, because the soil dissipates energy via nonlinear deformations, reducing the energy available to excite the structure. Larger soil deformation thus may imply relative reduction in structural response. If founded on more rigid soil, this building would have suffered more severe damage during the Northridge earthquake.

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