On the Variation of Fundamental Frequency (Period) of an Undamaged Building – A Continuing Discussion

by

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ABSTRACT

Variation of fundamental period (frequency) of undamaged structures has been the subject matter of numerous studies. Recently, this topic is rekindled with the premise but repeat of the well known conclusion that fundamental period (frequency) varies with amplitude of shaking. Some researchers appropriately called this "wandering" of the natural frequencies of a structure. Although due to various sources of excitation and time-varying environmental conditions, variation of the fundamental period (frequency) of even an undamaged structure should not be a surprise to many, it is important to understand why such variation is important for practical purposes. In this paper, we investigate the fundamental frequencies of an undamaged building for which there are numerous studies of several sets of vibration data, including forced vibration testing, strong shaking due to a distant large earthquake, and low-amplitude shaking due to ambient excitations as well as several small nearby earthquakes. It is shown that the fundamental frequency "wanders" in a consistent way with the level of shaking, and that the significant difference between low-amplitude and strong shaking is attributed to soil-structure interaction during stronger shaking

KEYWORDS: fundamental frequency (period), accelerations, earthquake response, spectra, building,

1.0 INTRODUCTION

1.1 General

Reasonably accurate assessment of fundamental period (frequency) of a structure is an essential part of design and analysis processes. It is also known that variation of fundamental period (frequency) of undamaged structures has been subject matter of numerous studies – too long to cite herein. Recently, with advanced technologies and methods to acquire and analyze vibration data from structures excited by natural and man-made sources, study of the subject matter is rekindled with the premise that fundamental period (frequency) varies with the amplitude of shaking (*e.g.* Calvi et al , 2006, Dunand et al, 2006, Todorovska et al, 2006). Clinton and others (2006) appropriately called this "wandering" of the natural frequencies of a structure. While due to various sources of excitation and time-varying environmental conditions, variation of the fundamental period (frequency) of even an undamaged structure should not be a surprise to many, but nonetheless it is important to understand and dwell upon as to whether such variation is important for practical purposes. It is also important to mention that accurate assessment of fundamental frequency is important to establish a baseline linear elastic behavior of a structure in order to interpret its nonlinear elastic or nonlinear inelastic behavior that may be observed in future events. The objective of this paper is to investigate the fundamental frequencies of the undamaged Pacific Park Plaza Building in Emeryville, CA, for which there are numerous studies of numerous sets of vibration data, including forced

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vibration testing, strong shaking due to a far distance large earthquake, and low-amplitude shaking due to ambient excitations as well as several small nearby earthquakes. It is repeated herein that the particular building being studied has not been damaged but, as shown in this study that its fundamental period (frequency) is observed to "wander" in a consistent way with the level of shaking. The scope of the paper is based on findings using actual data and does not include mathematical modeling of the building (except in reference to existing analyses by others). In the case of this building, the significant change in the value of fundamental period (frequency) between low-amplitude and strong shaking is attributed to soil-structure interaction (SSI) during stronger shaking. However, detailed SSI investigation of the building is beyond the scope of this paper but has been reported elsewhere (Kagawa et al, 1993a, b, Aktan et al, 1992, Kambhatla et al, 1992, Çelebi, 1992, 1998). This paper introduces additional results from new data that reinforces this argument.

1.2 The Building, Design Spectra and Instrumentation

The Pacific Park Plaza (PPP) Building is an equally-spaced three-winged, cast in place, thirty-story, 312 ft. (95.1 m) tall, ductile reinforced concrete moment-resisting frame building. The three wings of the building are constructed monolithically and are equally spaced at angles of 120 degrees around a central core. Shear walls in the center core and wings extend to the second floor level only, but column lines are continuous from the foundation to the roof. The foundation is a 5-foot-thick concrete mat supported by 828 (14-inch-square) pre-stressed concrete friction piles, each 20-25 m in length, in a primarily soft-soil environment that has an average shear-wave velocity between 250 and 300 m/s and a depth of approximately 150 ft (~50 m) to harder soil. A three-dimensional schematic of the building and its seismic instrumentation is shown in Figure 1. The instrumentation integrates arrays for the structure, surface, and downhole, and comprises a 30-channel accelerometer deployment uniquely designed to capture (a) the translational motions of the wings of the building relative to its core, (b) the vertical motions of the mat foundation slab at the ground floor level, and (c) free-field motions at the surface and at a downhole depth of 200 ft (61 m). The South Free-field (SFF) station is often referred to as the Emeryville (EMV) ground site. This building is selected for this study because there is a variety of old and new data and because there is no evidence that it experienced any damage during the various levels of shaking described in this paper.

1.3 Site Conditions

Based on a relatively recent geologic log and shear-wave velocity profile (Gibbs et al, 1994), the soils at the site consist of artificial fill, soft silty clay (Holocene Bay Mud), and stiff to very stiff, undifferentiated deposits composed of numerous layers of clay, loam, sand, and gravel. The layer of Holocene Bay Mud, clearly evident on the shear-wave velocity profile shown in Figure 2, begins at about 16 ft. (5 m) depth and is approximately 10 ft. (3 m) thick. Stiff deposits with shear-wave velocity (V_s) of approximately 820 ft/s (0.25 km/s) extend from below the Holocene Bay Mud to a depth of approximately 80 ft. (24 m). Very stiff Pleistocene deposits with V_s approximately equal to 1300 ft/s (0.4 km/s) extend to a depth of about 155 ft (48 m). The computed site transfer function, corresponding to the shear-wave velocity profile in Figure 2, using Haskell's shear-wave propagation method (Haskell, 1953, 1960) and coded by Mueller (*pers. comm.* 2002) is also provided in Figure 2, and indicates a site frequency at approximately 0.7 Hz.

1.4 Design Spectra and Significant Shaking Experienced

To date, the most significant shaking recorded by the building arrays was during the 1989 Loma Prieta (LPE), CA earthquake (M_s=7.1). The data set from LPE is extensively used in several studies as well as in this investigation that specifically dwells upon the variation of fundamental period with level of shaking. As previously mentioned, the building was not damaged.

Responses of the building and the surface free-field recorded during the strong shaking caused by the LPE earthquake exhibit distinct amplification of motions (Figures 3a) at the site of the building as compared to the motions at Yerba Buena Island, both approximately 100 km (and at similar azimuths) from the epicenter of the LPE. The east-west components of acceleration recorded at the roof and the ground floor of the structure and at the associated free-field station (SFF in Figure 1) are shown in Figure 3a. The motion at Yerba Buena Island (YBI), the closest rock site, had a peak acceleration of 0.06 *g*, and is also shown for comparison. The response spectra (Figure 3b) clearly demonstrate that the motions at Emeryville (SFF) were amplified by as much as five times when compared with YBI. This is also inferred by the amplitude of the peak accelerations (0.26 *g* for SFF and 0.06 *g* for YBI). Furthermore, the differences in peak acceleration at SFF (0.26 *g*) and at the ground floor of the building (0.21 *g*) (Fig. 3a) suggest the possibility of significant soil-structure interaction. Figure 3c shows a comparison of actual response spectra with site-specific design response spectra (based on the probabilistic earthquakes related to levels of performance) used in the design of the building: (a) the maximum probable earthquake (50 % probability of being exceeded in 50 years with 5 % damping) anchored at zero period acceleration (ZPA) of 0.32g. [curve A in Figure 3c], and two maximum credible earthquakes both with 10 % damping but 10 % probability of being exceeded in (b) 100 years (ZPA of 0.63 g) [Curve B in Fig. 3c] and (c) 50 years [ZPA of 0.53 g][2] . The spectra of the EW components of recorded motions at the ground floor and SFF are also shown in Figure 3c. At 100 km from the epicenter, even though the recorded EW peak acceleration at SFF (0.26 g) is smaller than the ZPA of the postulated maximum probable earthquake (0.32 g), the spectral accelerations of the EW component of SFF is considerably higher than the maximum probable earthquake for periods >0.6 seconds – that is, practically the first three modes of the building. This implies that, when large earthquakes occur closer to the structure, the level of shaking and the response spectra of motions are likely to be higher (for some period bands) than the design response spectra, and, in many cases, the code design response spectrum (*e.g.* the 1979 Uniform Building Code).

[2] not shown in the figure

2.0 SUMMARY OF STUDIES RELATED TO THE BUILDING

2.1 Data Sets

Extensive data sets from this building include not only the Loma Prieta earthquake response data but also those from smaller earthquakes and from forced and ambient vibration tests (Stephen et al, 1985, Çelebi et al, 1993). Table 1 summarizes the events (including LPE) that have been recorded by the building array and are used in this study. Those related to LPE and test data are summarized in Table 2.

2.2 Pre-1991 Data sets including LPE and Studies

The building has been studied in detail or as part of a larger investigation by several researchers (Çelebi and Safak, 1992, Safak and Çelebi, 1992, Anderson et al, 1991, Bertero et al, 1992, Kagawa et al, 1993a, b, Aktan et al, 1992, Kambhatla et al, 1992, Çelebi, 1992, 1998). Using different methods, including spectral analyses, system identification techniques (Çelebi, 1998), and mathematical models, the majority of the investigators are in agreement that, for the 1989 Loma Prieta earthquake data, the predominant three response modes of the building and the associated frequencies (periods) are 0.38 Hz (2.63 s), 0.95 Hz (1.05 s), and 1.95 Hz (0.51 s). These three modes of the building are torsionally-translationally coupled (Çelebi, 1998) and are depicted in the cross-spectra (S_{xy}) of the orthogonal records obtained from the roof, ground floor and SFF (the south free-field site) and the normalized cross-spectra of the orthogonal records (Figure 4). The site frequency at 0.7 Hz (1.43 s) observed in the cross-spectrum of the roof (Figure 4a) appears as the dominant peak in the cross-spectra of the ground floor and the south free-field (SFF) (Figure 4b and 4c). This site frequency has been also confirmed by the wave propagation method using site borehole data by Gibbs and others (1994) as shown in Figure 2. Justification of the site frequency as determined from this set of records are reported in Çelebi (2003).

Dynamic characteristics of the building extracted from the data sets are summarized in Table 2 and show considerable differences in the fundamental frequency determined from strong shaking versus low-amplitude shaking and analyses. The differences are attributed to SSI effects during strong shaking (Çelebi, 1998, Kagawa et al, 1993a, b, Aktan et al, 1992, Kambhatla et al, 1992), and frequencies from recorded motions can be matched when SSI is incorporated into the mathematical models (Kagawa et al, 1993a, b). Furthermore, a study of the building for dynamic-pile-group interaction (Aktan et al, 1992, Kambhatla et al, 1992) indicates that there is significant interaction. The study shows that computed responses of the building using state-of-the-art techniques for dynamic-pile-group interaction compares well with the recorded responses. Clearly, the mathematical models developed at that time needed improvements (Stephen et al, 1985). This conclusion could only be reached because we have recorded on-scale motions.

In addition, system identification techniques, when applied to the records of this building, yielded very large damping ratios corresponding to the 0.38-Hz first-mode frequency. These are 11.6 percent (north-south) and 15.5 percent (east-west) [Table 2] (Çelebi, 1996, 1998). Such unusually high damping ratios have been attributed to radiation damping that commonly occurs for buildings with large mat foundations in relatively soft geotechnical environment (Çelebi, 1996).

Anderson and others (1991) compared the design criteria, code requirements, and the elastic and nonlinear dynamic response of this building due to the earthquake. They also found the fundamental frequency of the building to be ~ 0.37-0.39 Hz. However, contrary to others, but based only on comparison of ground level motions with those at the free-field, they concluded that soil-structure interaction was insignificant for this building during this earthquake.

2.3 Recent Data, Analyses and Discussion

Analyses of subsequent data sets listed in Table 1 show that for shaking much lower than caused by LPE, the fundamental frequency (period) is significantly lower (longer) than that determined using the LPE record. In Figure 5, for each of the 1998, 2000, 2003 and 2006 earthquakes (Table 1), plots of acceleration time history and corresponding amplitude spectra are shown for the 30th floor and ground floor of the building. Consistently, a structural fundamental frequency (period) of ~ 0.48 Hz (~2.08 s) is identified. This identified frequency is also confirmed by system identification method. For the sake of brevity, only a sample system identification plot is presented for the 2006 event (Figure 6) which clearly shows the fundamental frequency at 0.48 Hz. For all events and tests to date, Table 3 summarizes the level of shaking (acceleration in g's) and identified dynamic characteristics (frequencies and damping ratios). These results are also graphically depicted in Figure 7. Both Table 3 and Figure 7 complements and reinforces the argument that the fundamental frequency varies significantly with the level of shaking even if the building may not be damaged. In the case of Pacific Park Plaza Building, the variation is attributable to SSI.

As noted in this paper, there is significant difference between the 0.38 Hz and 0.48 Hz frequencies (approximately 20% less for LPE if 0.48Hz is considered as the baseline and even more if 0.59 Hz is considered). In many studies, establishment of baseline frequency can be an issue and therefore ought to be carefully assessed to prevent erroneous interpretation. Another point to be made is that, in reaching the conclusions in this paper, most of the data analyses were made with data with time increments of 0.005 seconds. It was observed during the data analyses that overdecimating and oversmoothing the data can lead to significant differences in the assignments of values to the fundamental frequencies.

4.0 CONCLUSIONS

Recorded responses of structures serve to expose unusual and unexpected response characteristics that require detailed analyses in order to improve or validate analytical models and design processes and to identify possible methods for retrofit of the structure if necessary. Significant findings, although not limited by the list below, are summarized as:

1. It is shown that there are significant differences in the fundamental frequencies of Pacific Park Plaza Building determined from strong shaking as compared to low-amplitude shaking. Thus, the variation of fundamental frequency (period) is dependent on the amplitude of shaking.
2. System identification procedures are very useful in extracting the dynamic characteristics; in particular, the modal damping ratios (for the defined level of shaking) which otherwise are difficult to determine as they are not constant and increase with the level of shaking.
3. Soil-structure interaction, although neglected in the design-analysis process of this building and as is also neglected for most non-critical buildings, plays a significant role in altering dynamic characteristics and therefore the response of buildings. For this building, the variation of the fundamental period (frequency) is quite substantial.

Additional conclusions may also be stated as:

4. As expected, higher modes are excited for this building during the earthquake events.
5. It is shown in this paper (as also in previous papers) for this building that the translational and torsional responses are coupled. This conclusion may be generalized for buildings with irregularities.
6. In certain cases, as for this building, response spectra of recorded motions exceed design response spectra within some (lower) frequency bands that include structural frequencies. In determination of design response spectra, effect of lower frequency ground motions must be taken into account.
7. Finally, particularly in areas of high seismicity, deployment of seismic monitoring systems particularly for complex and irregular buildings and other types of structures are strongly encouraged since records obtained during future events reveal response characteristics that are not always envisioned or taken into account during design and analysis processes.

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Table 1. Events that have been recorded by the PPP arrays

Event/ Date	UTC	Lat. (N)/ Long. (E)	Dist. (km)	Azim. (deg)	Depth (km)	Mag.
Loma Prieta 10/18/1989	04:15	37.036 -121.883	96	157	18.0	M_s 6.9
El Cerrito 12/04/1998	12:16	37.920 -122.290	9	4	6.8	M_w 4.0
Yountville 09/03/2000	08:36	38.379 -122.413	61	350	10.1	M_w 5.0
Piedmont 09/05/2003	01:39	37.845 -122.222	7	85	12.4	M_w 3.9
Berkeley 03/02/2006	06:08	37.863 -122.245	5	96	11.4	M_d 2.8

Table 2. Peak Accelerations and System Identification Results for (Pre-1991) PPP Data

Peak Accelerations (A[g]))				
	Loma Prieta Eq. (1989) [See Refs]		Low-Amp. Tests & Analyses [See Refs]	
	NS	EW	NS	EW
Roof	0.24	0.38	<0.01	<0.01
Gr. Fl.	0.17	0.21	<0.01	<0.01
FF	0.21	0.26	-	-
Dynamic Characteristics (System Identification)				
f_o (Hz)	0.38	0.38	0.48-0.59	
T_o (s)	2.63	2.63	1.69-2.08	
ξ(%)	11.6	15.5	0.6-3.4	

Table 3. Summary - Events, Levels of Shaking (in g's) and Identified Dynamic Characteristics

Tests, Analyses or Events												
	1985 Tests/ analyses	1989 LPE		1990 Tests	1998 EQ. (1204_ 1216)		2000 EQ. (0903- 0836)		2003 EQ (0905_ 0139)		2006 EQ (0302_ 0608)	
Peak Accelerations (A[g]) [NS & EW represents 350^o and 260^o respectively]												
	NS/EW	NS	EW	NS/EW	NS	EW	NS	EW	NS	EW	NS	EW
Roof	<0.01	0.24	0.38	<0.01	.025	.016	.01	.007	.056	.067	.004	. 003
Gr.Fl.	<0.01	0.17	0.21	<0.01	.016	.037	.005	.004	.037	.041	.003	.003
SFF	-	0.21	0.26	-	.022	.028	-	-	.039	.031	.003	.006
Dynamic Characteristics (System Identification & Spectral Analyses)												
f_o(Hz)	0.59	0.38	0.38	0.48	.48	.48	.48	0.48	0.48	0.48	0.48	0.48
T_o (s)	1.69	2.63	2.63	2.08	2.08	2.08	2.08	2.08	2.08	2.08	2.08	2.08
ξ(%)	.6-3.4	11.6	15.5	.6-3.4	-	-	-	-	.5-2.	.5-2.	.5-2.	.5-2.

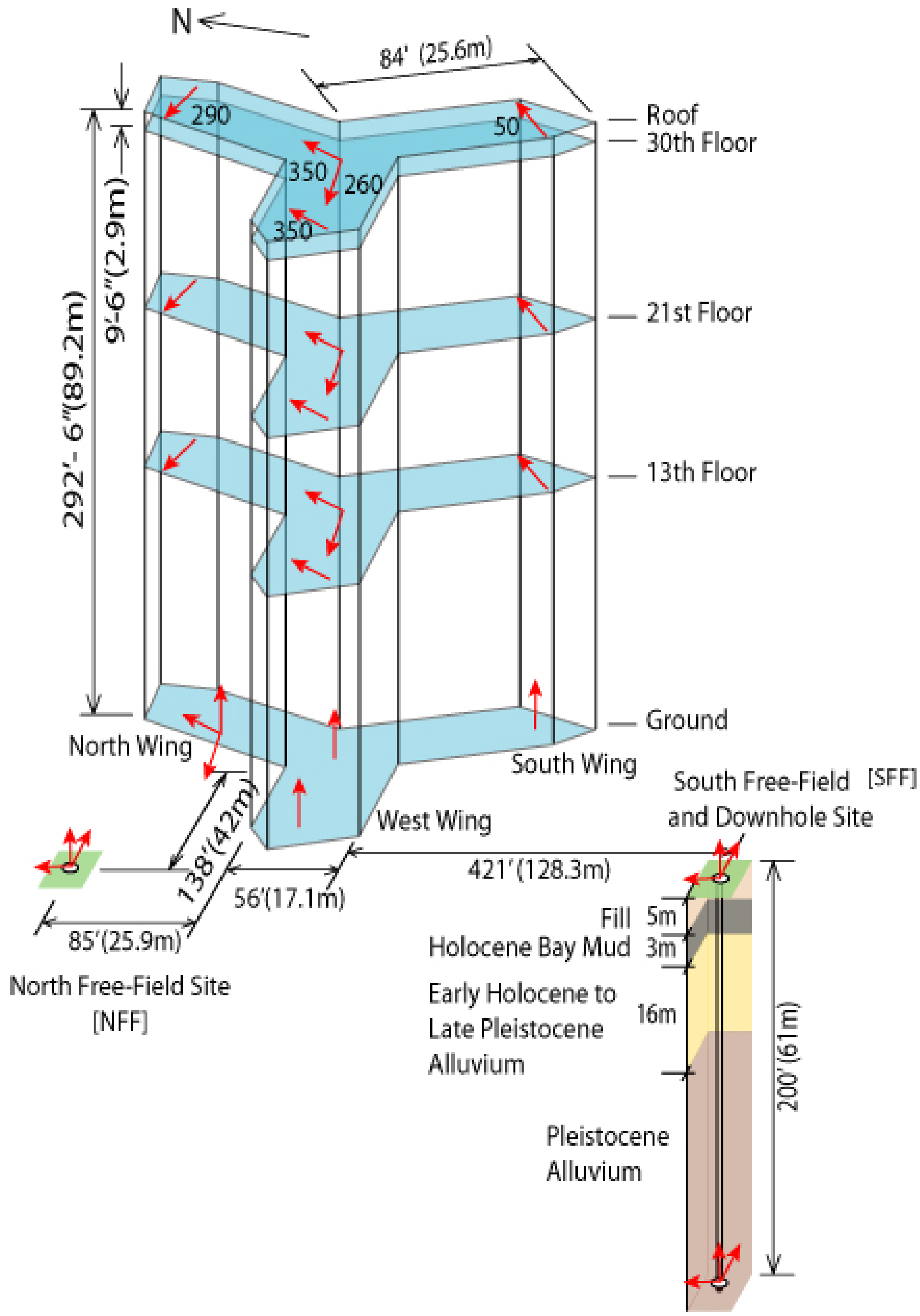

Figure 1. A three-dimensional schematic of the building array with integrated surface and downhole array. Red arrows indicate sensor locations and orientations. The tri-axial downhole accelerograph was added after the 1989 Loma Prieta earthquake.

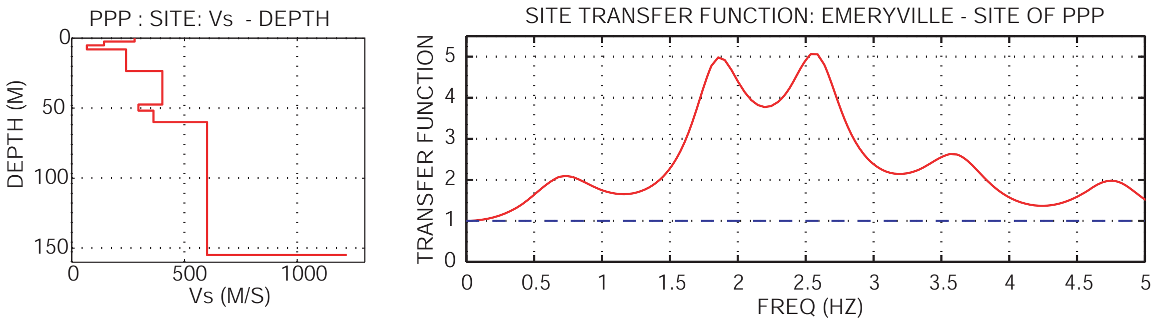

Figure 2. Shear-wave velocity profile and the computed site transfer function. 0.7 Hz is the fundamental frequency and other peaks belong to higher modes.

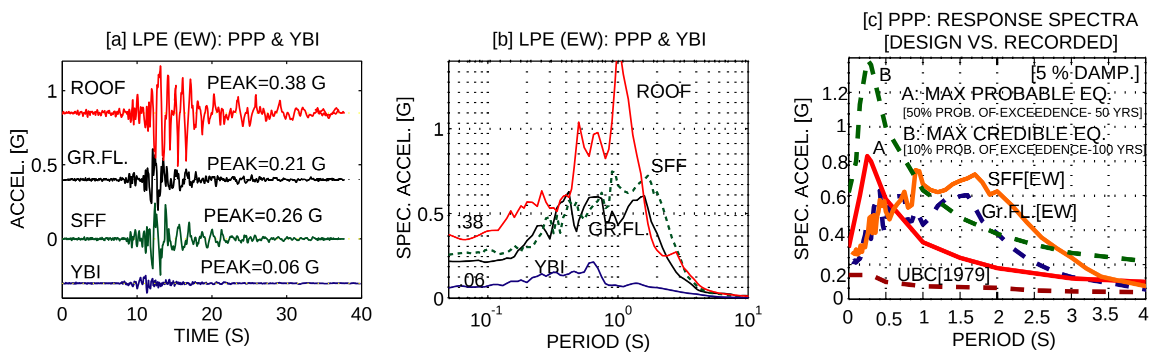

Figure 3. (a,b) Amplified (EW) motions and their corresponding response spectra (5% damped) at the South Free-Field (SFF), ground floor and roof of the Pacific Park Plaza array as compared to the motions at Yerba Buena Island (YBI) at approximately the same epicentral distance as PPP. (c) Design response spectra and response spectra of recorded motions at the ground floor and SFF of Pacific Park Plaza. Also shown is the 1979 UBC response spectrum for comparison. [Note: Curve B is for 10% damping].

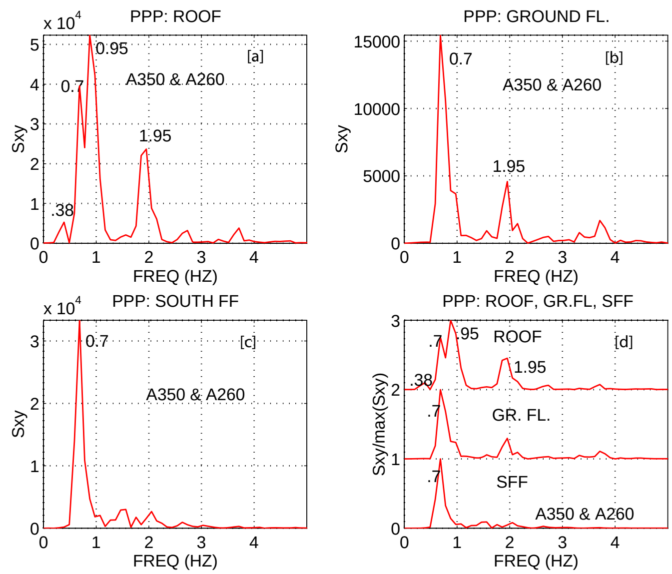

Figure 4. Cross-spectra of orthogonal motions at the [a] roof, [b] ground floor, [c] free-field of PPP, and [d] the normalized cross-spectra depicting structural and site frequency peaks.

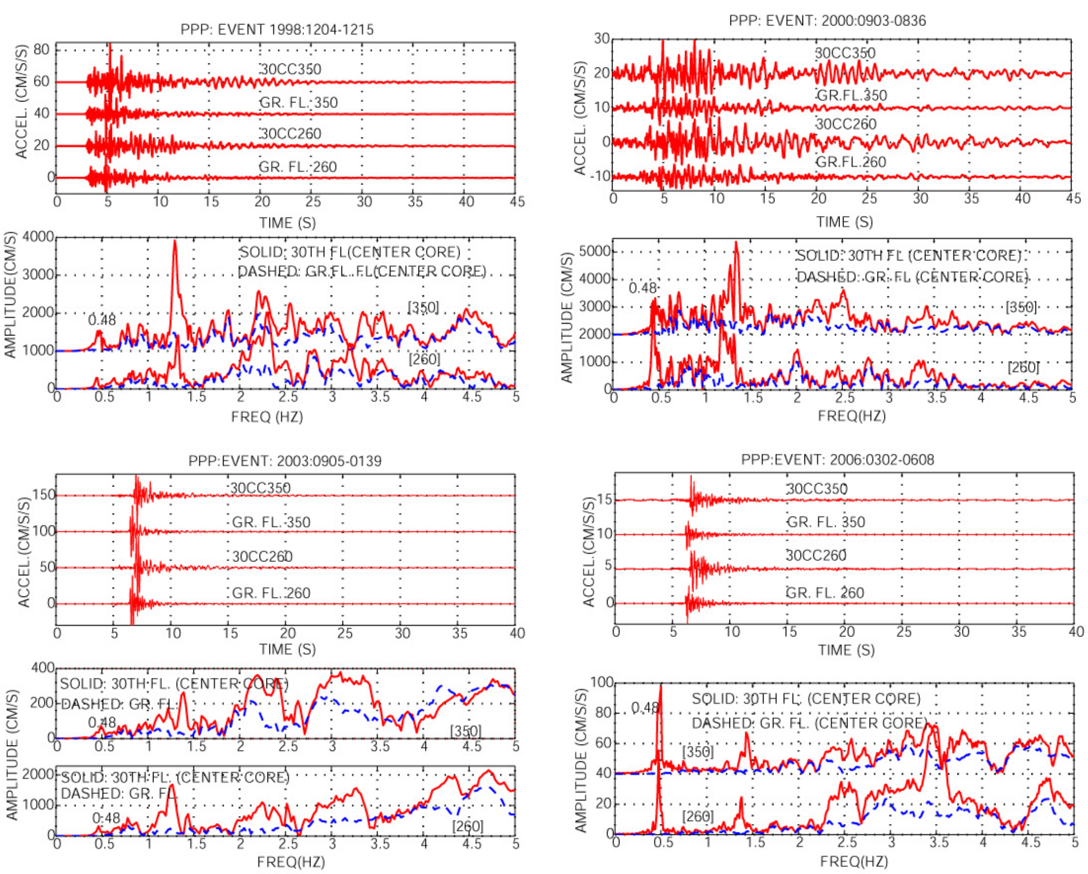

Figure 5. Recorded accelerations at 30th and ground floors and corresponding amplitude spectra.

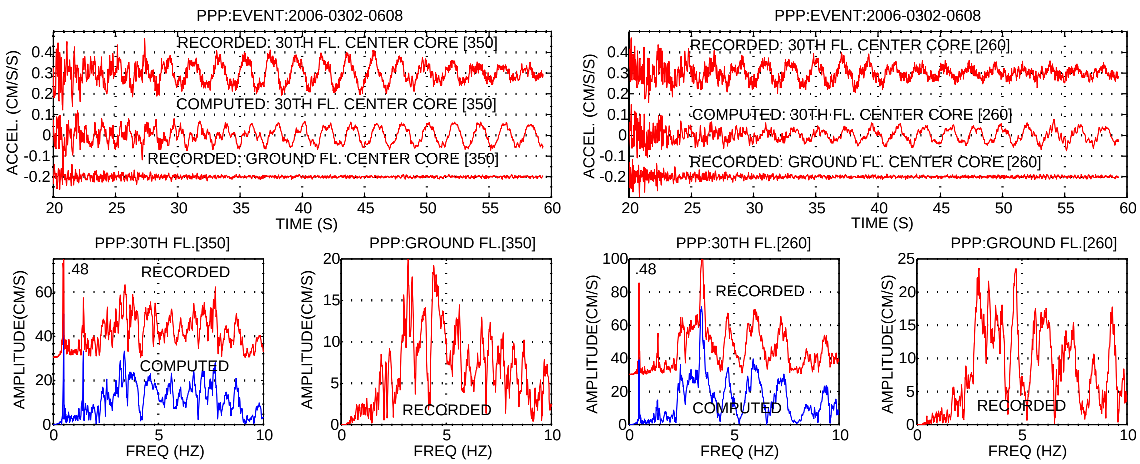

Figure 6. System identification for 2006 event using 40-second window of acceleration data. Ground level motions are used as input and 30^{th} floor motions are used as output.

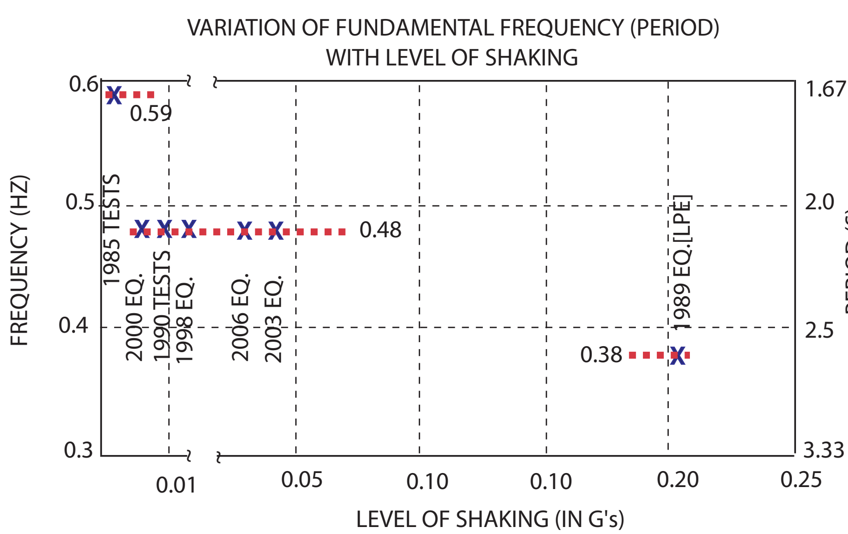

Figure 7. Plot showing variation of fundamental frequency with level of shaking (in g's).