Identification of Site Frequencies from Building Records

M. Çelebi,[a)] M.EERI

A simple procedure to identify site frequencies using earthquake response records from roofs and basements of buildings is presented. For this purpose, data from five different buildings are analyzed using only spectral analyses techniques. Additional data such as free-field records in close proximity to the buildings and site characterization data are also used to estimate site frequencies and thereby to provide convincing evidence and confirmation of the site frequencies inferred from the building records. Furthermore, simple code-formula is used to calculate site frequencies and compare them with the identified site frequencies from records. Results show that the simple procedure is effective in identification of site frequencies and provides relatively reliable estimates of site frequencies when compared with other methods. Therefore the simple procedure for estimating site frequencies using earthquake records can be useful in adding to the database of site frequencies. Such databases can be used to better estimate site frequencies of those sites with similar geological structures. [DOI: 10.1193/1.1542618]

INTRODUCTION

Reliable calculations and/or estimates of the fundamental frequency of a building and its site are essential during analysis and design process. Various code formulas based on empirical data are generally used to estimate the fundamental frequency of a structure. Alternatively, if dynamic modal analysis is performed, fundamental frequencies and mode shapes are obtained. For existing structures, in addition to code formulas and available analytical tools such as modal analyses, various methods of testing including ambient and forced vibration testing procedures may be used to determine dynamic characteristics. Reliable strong-shaking dynamic characteristics are obtained when and if the structures are instrumented and their on-scale responses are recorded during strong shaking events. Spectral procedures and system identification techniques applied to the recorded strong shaking response data yield very accurate assessments of the actual dynamic characteristics.

While structural frequencies can be calculated using mathematical models or determined from records, identification of site frequencies are not as straightforward; hence, often, the estimation of site frequencies are made using simple empirical relationships with rough parametric values and without mathematical modeling. Recent codes provide approximate estimates of site frequencies if geotechnical logs are available. Furthermore, there are always some uncertainties in prediction of site frequencies because of the assumptions made in establishing representative site characteristics. The frequently

[a)] U.S. Geological Survey, MS 977, 345 Middlefield Rd., Menlo Park, CA 94025

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used simple formula, $T_s = 4H/V_s$, requires minimal but reasonable characterization of depth to bedrock and representative average shear-wave velocities of layered media (*International Building Code* [ICC 2000]).

In this paper, the objective is to show that when and if structural response records are available from both the roof and ground floors or basements of building structures, simple spectral analyses procedures can be used to convincingly identify not only structural frequencies but also site frequencies.

For all of the five cases included in this study, detailed assessments of structural characteristics as well as assessments of site frequencies were included in previous studies with much wider scopes (Çelebi 1992; 1993a, b, c; 1994; 1998). It is recognized herein that studies of assessments of structural frequencies from building records and related data include but are not limited to the studies by Foutch and Jennings (1978), Goel and Chopra (1997, 1998), Trifunac et al. (2001a, b). In particular, Trifunac et al. (2001a, b) dwell upon studies of apparent structural periods defined to be larger than fixed-based structural periods and incorporates translational and rocking interaction periods by the relationship $T_{apparent} = \surd(T^2_{fixed} + T^2_{trans} + T^2_{rocking})$. Numerous other studies are too long to list herein. However, unlike this present study concentrating primarily on identification of site frequencies using recorded building responses motions, others concentrate on structural frequencies.

The benefits of such a procedure to identify site frequencies are considerable. Identified site frequencies can be used in databases that aim to classify site characteristics against dynamic characteristics, and in assessing other techniques used to identifying site frequencies. Furthermore, the procedure can be applied to sets of data available from code-type instrumented buildings (three tri-axial accelerographs placed at the roof, mid-height, and basement of buildings) as well as from buildings instrumented with multiple sets of sensors in different floors. It is envisioned that databases of site frequencies extracted from building responses records may be developed for future use. A database that results from such assessments can be used for similar sites when, otherwise, there is insufficient information to infer site frequencies.

The scope of this paper is limited to demonstration of the procedure with five sets of building response data, four obtained during the 1989 Loma Prieta (California) earthquake ($M_s = 7.1$) and another obtained during the 1987 Whittier, California, earthquake ($M_s = 5.8$). The intent here is to identify structural frequencies and site frequencies from synchronized strong-shaking data recorded during strong-shaking events from instrumented structures. Low-amplitude data from ambient and forced vibration test are not used in this study because the objective is to extract site frequencies (in addition to structural frequencies) during shaking caused by strong-motions. Such studies provide wide variations in structural characteristics inferred from low-amplitude motions as compared to those determined from strong shaking data (Çelebi et al. 1993).

THE PROCEDURE

In order to identify site frequencies, the following steps are essential:

1. At least two pairs of horizontal components of recorded data, one pair from the

roof and the other from the ground floor or basement are required. Either parallel and/or orthogonal pairs of data from roof or basement can be used.

2. The structural frequencies are identified first. The roof and/or upper-floor records are naturally best suited for this. The following well-known methods are used to identify structural frequencies:
   a. spectral analyses (amplitude spectra and spectral ratios, cross-spectra, and coherence and phase relationships) and
   b. system identification methods. In the event that system identification procedures are used, the roof and/or upper-floor data constitute the output and basement and/or ground-floor records are adopted as the input motion.
3. Once structural frequencies are confidently identified, then the site frequency is distinguishable:
   a. if one of the nonstructural dominant peaks of cross-spectrum of ground-floor (or basement) motions is different than the structural frequency, then that frequency is likely the site frequency,
   b. if the spectral ratio cancels out a dominant frequency that clearly appears in amplitude and/or cross-spectrum, then that frequency is not a structural frequency but it most likely is the site frequency, and/or,
   c. cross-spectra or normalized cross-spectra $S_{xy}/\max(S_{xy})$ calculated from pairs of roof and basement (or ground floor) data exhibit site frequencies.
4. Availability of free-field records from a free-field station that is in the proximity of the building adds further confidence in confirming the identified site frequency. The amplitude spectra of the components of and/or cross-spectrum of orthogonal horizontal components of free-field motions usually reveals the site frequency.
5. If, in addition, site characterization data (depth to bedrock, geological borehole data, shear-wave velocities of different layers of soil below the foundation) are available, transfer functions can be calculated to add further confidence (as described below).

The question may arise as to whether the frequency identified as belonging to site is possible due to source effects. The site frequencies identified are larger than expected source frequencies for the two main earthquakes (Loma Prieta and Whittier) considered. The corner frequency (an expression of the source frequency) is approximately equal to inverse of the source duration. Source duration is the amount of time from start to the end of rupture (as determined from records that are in close distances to the rupture area and not affected by site response) (Hanks and McGuire 1981). Therefore, for both the Loma Prieta and Whittier earthquakes (records from which are used in this paper), the strong-shaking duration is longer than 5 seconds. That makes the corner frequencies less than 0.2 Hz. None of the frequencies identified as belonging to the site is less than 0.2 Hz. Therefore the identified site frequencies cannot be source frequencies.

COMPUTING SITE TRANSFER FUNCTIONS USING SITE CHARACTERIZATION DATA

It is noted herein that the inclusion in this paper of computed transfer functions based on available site characterization data is not the main objective of the paper. Rather, computations of transfer functions are made only to supplement the identification of site frequencies from building records and therefore serve to verify and add confidence in the approach used.

Although other methods are available, computation of site transfer functions is performed by using a software program developed by Mueller (pers. comm. 1997) using Haskell's shear-wave propagation method (Haskell 1953, 1960). In this method, the transfer function is computed using linear propagation of vertically incident SH waves and as input data related to the layered media (number of layers, depth of each layer, corresponding shear-wave velocities [V_s], damping, and density), desired depth of computation of transfer function, sampling frequency, half-space substratum shear-wave velocity and density. Damping (ξ) in the software is provided as Q, a term used by geophysicists and is equivalent to damping by the relationship $\xi = 1/(2Q)$. Q values used in calculating the transfer functions are between 25 and 60 for shear-wave velocities between 200 and 600 m/s—having been approximately interpolated (rounded off) to vary linearly within these bounds.

For all of the cases described below, the following assumptions and interpolations are made, based on past analyses and in consultations with others (T. Fumal, pers. comm. 1997; J. Tinsley, pers. comm. 1997; D. Boore, pers. comm. 2002):

- Q values used in calculating the transfer functions are between 25 and 60 for shear-wave velocities between 200 and 600 m/s—having been approximately interpolated and rounded off to vary linearly within these bounds.
- Half-space substratum shear-wave velocities have been assumed to be between 700 and 1200 m/s. These values are included in Table 3 for each case. It should be mentioned herein that the impact of substratum V_s in determination of frequency is negligible—even though it affects the amplitude of the computed transfer function. The significant parameters are the depths and shear-wave velocities of the layered media above the substratum.
- Whenever possible, values for depth to bedrock were used. In cases where such information was not available, interpolations and assumptions were made. In each case that follows, if available, depth to bedrock information is cited with references.

CASE 1: PACIFIC PARK PLAZA (PPP) AT EMERYVILLE, CALIFORNIA

Detailed analyses of 1989 Loma Prieta earthquake ($M_s = 7.1$) response recordings of the 30-story, reinforced concrete framed Pacific Park Plaza (PPP) in Emeryville, California, have been presented by Çelebi and Safak (1992), Anderson, Mirando and Bertero (1991), and Çelebi (1998). Recently compiled borehole and site characterization information is also available (Gibbs et al. 1994a). Figure 1 shows plan view, instrumentation scheme of the building, and the location of the free-field stations associated with this

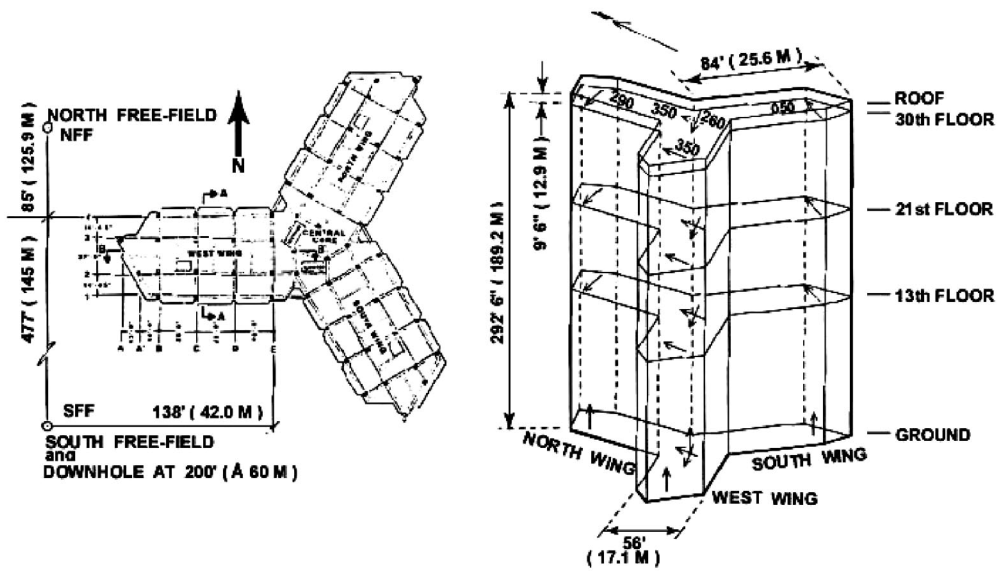

Figure 1. General layout and instrumentation scheme of Pacific Park Plaza, Emeryville, CA.

building instrumentation. The available free-field strong motion recording is pertinent to the convincing identification of the site frequency. Figures 2a and b depict building accelerations recorded at the core of the top instrumented level, at the core of the ground floor and the associated south free-field of the three-winged building. Corresponding amplitude spectra are provided in Figures 2c and d. The first three modal structural frequencies (periods) clearly identified from the recordings are 0.38, 0.95, and 1.95 Hz (2.63, 1.05, 0.34 s). The peak at 0.7 Hz that appears in the amplitude spectra of the roof also appears as the dominant peak in the amplitude spectra of the ground floor and the south free-field (SFF). However, this peak at 0.7 Hz disappears in the spectral ratios calculated from the amplitude spectra of the roof and ground floor as depicted in Figures 2e and f. This indicates that 0.7 is the site frequency as, although it appears in the roof spectra, it cancels out when ratios are calculated.

In Figure 3, cross-spectra, calculated from pairs of orthogonal components of acceleration recorded at the (a) roof, (b) ground floor, and (c) free-field are presented. The roof cross-spectrum clearly identifies the aforementioned frequencies of the first three modes. These modes are coupled torsional-translational modes (Çelebi 1998). The peak at 0.7 Hz that appears in the cross-spectrum of the roof appears as the dominant peak in the cross-spectra of the ground floor and the south free-field (SFF). Next, when the normalized cross-spectra are calculated for the ground floor and free-field, the site frequency at 0.7 Hz is distinguishable from the structural frequencies in the normalized cross-spectrum of the roof (Figure 2d). This is further confirmed by the lowest frequency peak at 0.7 Hz of the transfer function (Figure 4) calculated by using (previously described) Haskell's shear-wave propagation method (Haskell 1953, 1960) using site char-

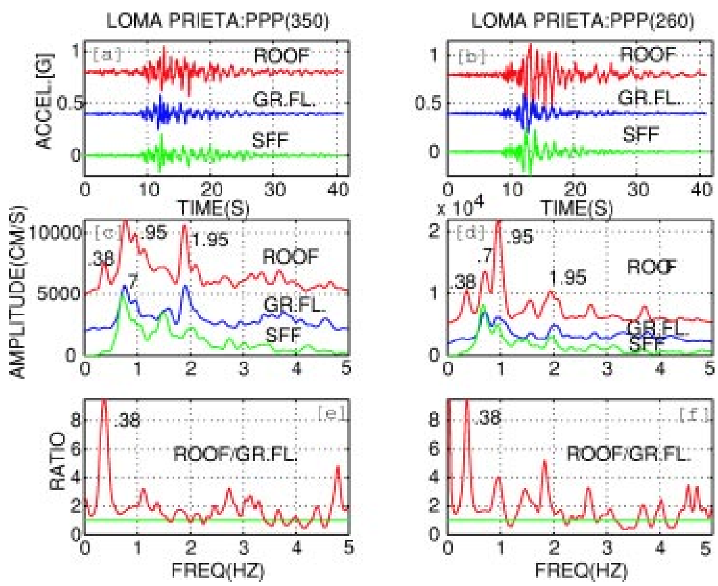

Figure 2. (a, b) Recorded orthogonal accelerations at the roof, ground floor, and south free-field of Pacifica Park Plaza; (c, d) corresponding amplitude spectra; and (e, f) ratios of amplitude spectra.

acterization data related to variation of shear-wave velocities with depth (Gibbs et al. 1994a). The depth to bedrock has been adopted from a map by Hensolt (1993) as 150 m (∼500 ft).

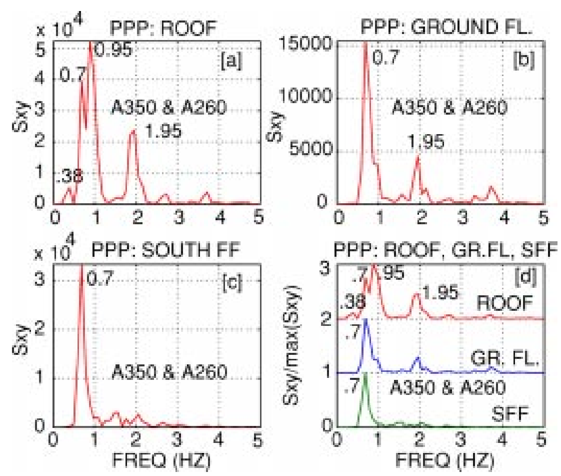

Figure 3. Cross-spectra of motions at the (a) roof, (b) ground floor, (c) free-field of Pacific Park Plaza, Emeryville, CA, and (d) normalized cross-spectra.

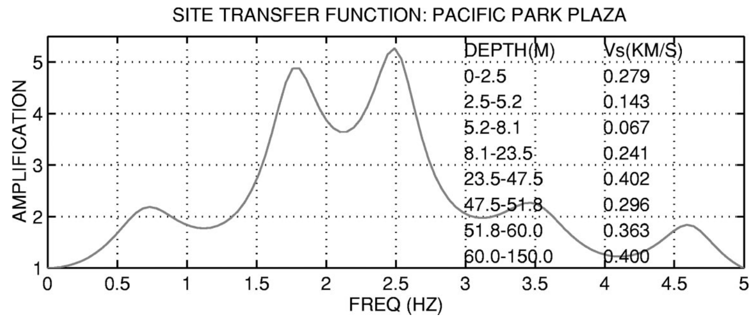

Figure 4. Site transfer function for described characterization at Pacific Park Plaza.

CASE 2: TWO-STORY OFFICE BUILDING (OAK), OAKLAND, CALIFORNIA

McClure (1991) provides detailed particulars of the 2-story office building in Oakland, California. The instrumentation scheme of this building as well as accelerations recorded during the 1989 Loma Prieta earthquake from the roof, second floor, and ground floor are provided in Figure 5. Ambient tests of the building performed in the 1965 yielded first-mode frequency (period) as 2.13 Hz (0.47 sec) and 2.08 Hz (0.48 sec) for the NS and EW, respectively, and forced vibration tests, also performed in 1965, yielded 2.35 Hz (0.426 sec) (Bouwkamp and Blohm 1966). These and other assessments of modal frequencies of the building are summarized in Table 1.

Figure 6 shows amplitude spectra of recorded accelerations in both the NS and EW directions and rotational accelerations (difference between parallel records) at the three structural levels (roof, second floor, and ground floor). Figure 7 shows time histories, amplitude spectra, and spectral ratios for pairs of recorded accelerations at the roof and

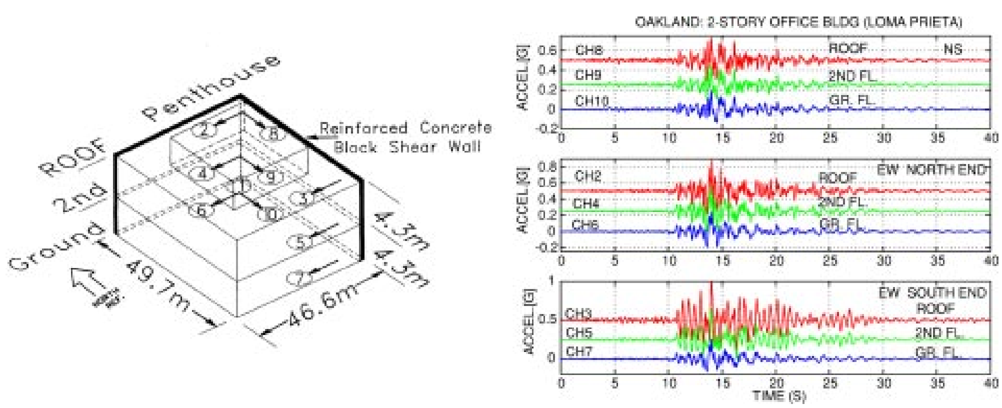

Figure 5. (Left) Instrumentation scheme of the torsionally eccentric 2-story building in Oakland, CA, and (right) accelerations recorded at the roof, second floor, and ground floor during the 1989 Loma Prieta earthquake.

Table 1. Modal frequencies (periods) of the 2-story Oakland building

Assessment Method	Modal Frequencies (Hz) (Periods [seconds]) NS	EW	Comments
1965 Ambient Test (McClure 1991)	2.13 (1st) (0.47)	2.08 (0.48)	Walls in, no plaster
1965 Forced Vibration Test (Bouwkamp and Blohm 1966)	(1st) 2.35 (0.426), (2nd) 7.70 (0.130)		Walls in with plaster
1988 *UBC* (McClure 1991)	2.29 (0.437)		Code Formula
McClure Computer Model (1991)	(1st) 1.59 (0.63), (2nd) 5.0 (0.20)		w/o plaster
	(1st) 2.16 (0.463), (2nd) 7.69 (0.130)		with plaster
Spectral analyses (this study)	1.95 (0.51) Translational, 1.65 (0.61) Torsional		Loma Prieta (1989) data

ground floor. From the spectra, three distinctive frequencies (0.82–0.85, 1.65, and 1.95 HZ) are identified. The frequencies 1.65 Hz and 1.95 Hz are structural frequencies determined by the fact that they have a very high ratio amplitude as seen in the spectral ratio plots (Figures 7g–7i) calculated from the pairs of amplitude spectra of the roof and ground floor motions (Figure 7d–7f). These two frequencies are very close to one another. Therefore, given the structural irregularity created mainly by the north and east end walls, the structure responds in a closely coupled translational-torsional mode with frequency between 1.65 and 2 Hz. The 0.82–0.85 Hz (NS) and 0.65–0.85 Hz (EW) frequencies that appear in the amplitude spectra do not appear in the spectral ratios because they cancel out. Therefore, it is safe to declare that the site frequency is between 0.65 and 0.85 Hz.

Figure 8 depicts coherence, phase angle, and cross-spectrum of the pairs of parallel motions at the (a) roof (CH2 and CH3) and (b) ground floor (CH6 and CH7). Because

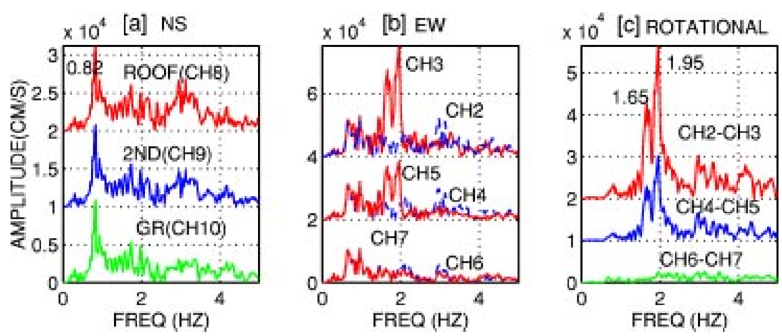

Figure 6. Amplitude spectra of translational and torsional accelerations recorded at Oakland (2-story building).

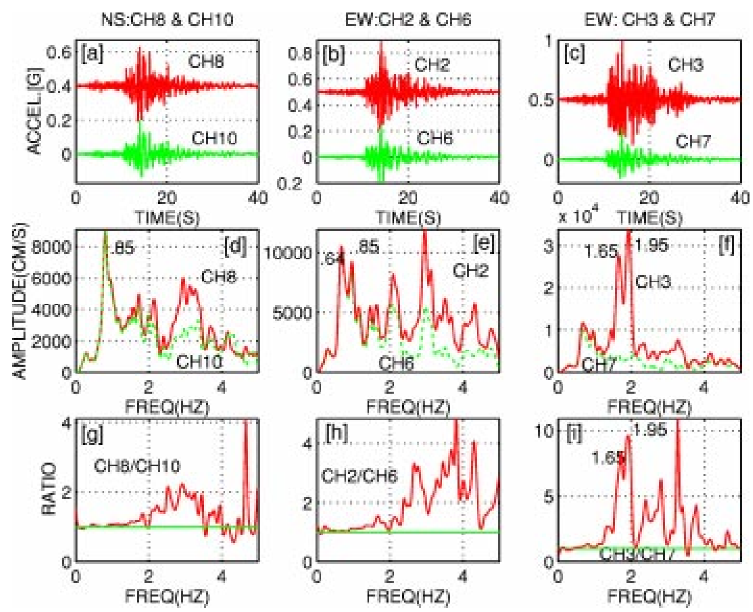

Figure 7. (a–c) Time-histories of roof and ground floor acceleration pairs, (d–f) corresponding amplitude spectra, and (g–i) corresponding roof/ground floor spectral ratios.

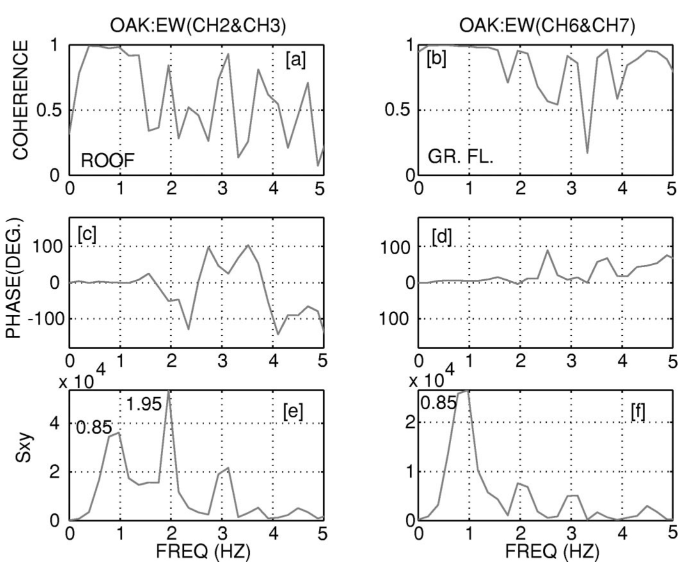

Figure 8. (a, b) Coherence, (c, d) phase, and (e, f) cross-spectra plots of pairs of parallel motions at the roof (CH2 and CH3) and ground floor (CH6 and CH7).

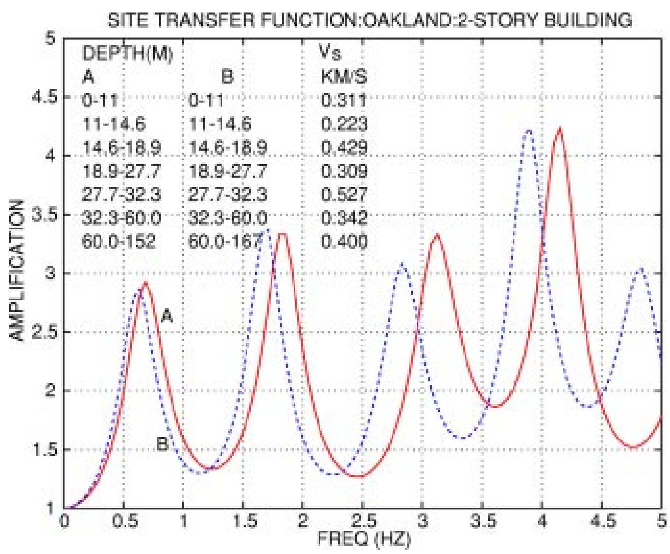

Figure 9. Computed site transfer function for described site characterization of the site of the 2-story building, Oakland, CA (see Figure 5).

the 2-story building is very rigid, both the structural frequency and the site frequency appear in the cross-spectrum plots of the roof and ground floor motions, although the amplitude of the site frequency at the ground floor is much larger than that of the structural frequency.

There is no free-field station associated with the building; however, recently documented site characterization data in proximity to the building (Gibbs et al. 1993) allows determination of site transfer function (Figure 9). Depth to bedrock (two cases) have been estimated from Hensolt map (1993). There is a good match between the lowest frequency (0.6–0.7 Hz) in Figure 9 and the site frequency (0.65–0.85) extracted from the building records (Figures 6 and 7).

CASE 3: SANTA CLARA COUNTY OFFICE BUILDING (SCCOB), SAN JOSE, CALIFORNIA

The building for Case 3, the Santa Clara County Office Building (SCCOB) in San Jose, California, is perhaps the most complex response cases caused by three close frequencies (0.38, 0.45, and 0.57 Hz) (Çelebi 1998). Figure 10 depicts the instrumentation scheme and the relative location of the building and the epicentral locations of the three earthquakes that were recorded on and before the 17 October 1989 Loma Prieta earthquake.

Figure 11 shows the very unique responses of the roof of the building to the three different earthquakes with different azimuths (1984 Morgan Hill [M_s=6.1], 1986 Mt. Lewis [M_s=5.5], and 1989 Loma Prieta [M_s=7.1], earthquakes). These are typical exhibitions of coupled torsional and translational responses with significant beating effect

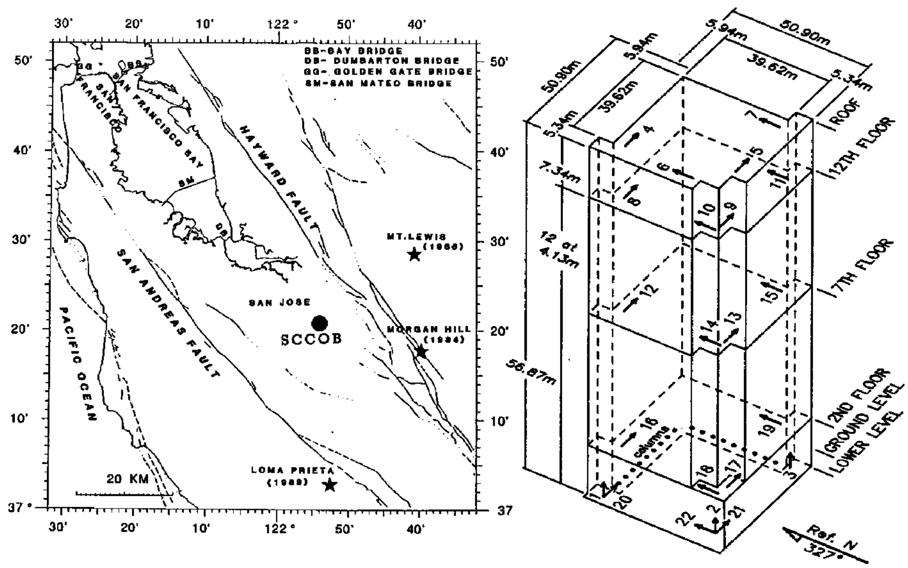

Figure 10. Location map of Santa Clara County Office Building in San Jose, CA, and the epicenters of the three earthquakes that were recorded by the instrumentation array within the building.

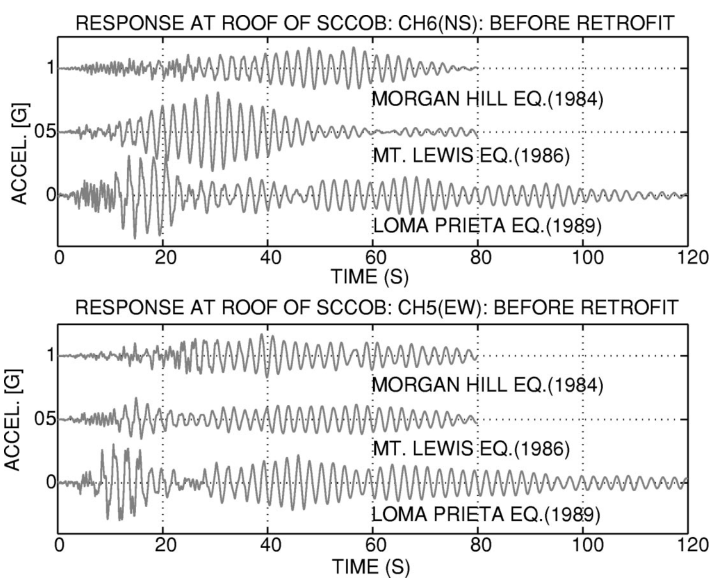

Figure 11. Time histories of accelerations recorded at the roof of the Santa Clara County Office Building during the 1989 Loma Prieta, 1984 Morgan Hill, and 1986 Mt. Lewis earthquakes. The building was retrofitted in 1994. The responses exhibit the beating effect caused by closely coupled translational and torsional modes and low damping.

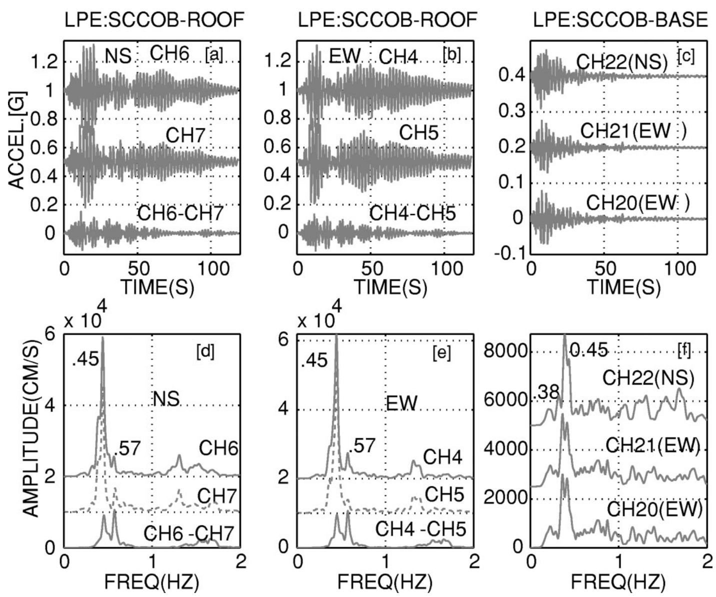

Figure 12. (a–c) Translational and torsional accelerations at the roof and translational accelerations at the basement, and (d–f) their amplitude spectra.

caused by closely spaced translational frequency (0.45 Hz) with the torsional frequency (0.57 Hz) and low critical damping of approximately 2% of the structural system (Çelebi 1994, 1998; Boroschek and Mahin 1991). Due to this type of behavior, the building was retrofitted by adding viscous elastic dampers (Crosby et al. 1994). Although strong shaking data has not been recorded since the retrofit in 1994, it is expected that in the future the response of this building will not resemble those in Figure 11 and due to expected shift in building frequency and increased damping, the beating effect will disappear.

Figures 12a–c show pairs of parallel translational accelerations at the roof and their differences representing torsional accelerations (NS: CH6, CH7, and CH6-CH7) and (EW: CH4, CH5, and CH4-CH5) and translational accelerations (NS: CH22, EW: CH20 and CH21) at base of the SCCOB. Corresponding amplitude spectra of these motions are provided in Figures 12d–f. The fundamental frequency (period) of the building at 0.45 Hz (2.22 sec) belongs to the translational mode and the frequency (period) at 0.57 Hz (1.75 s) belongs to the torsional mode, hence, the closely coupled translational-torsional response of the building that causes the beating effect. Details of these effects are provided by Çelebi (1994, 1998) and Boroschek and Mahin (1991). The frequency at 0.38 Hz (2.63 sec) belongs to the site.

The attributions to structural and site frequencies are confirmed by the spectral ratios of roof motions with respect to base motions (Figures 13a and b). The site frequency (0.38 Hz) cancels out in the spectral ratios of roof/base motions. This frequency (0.38

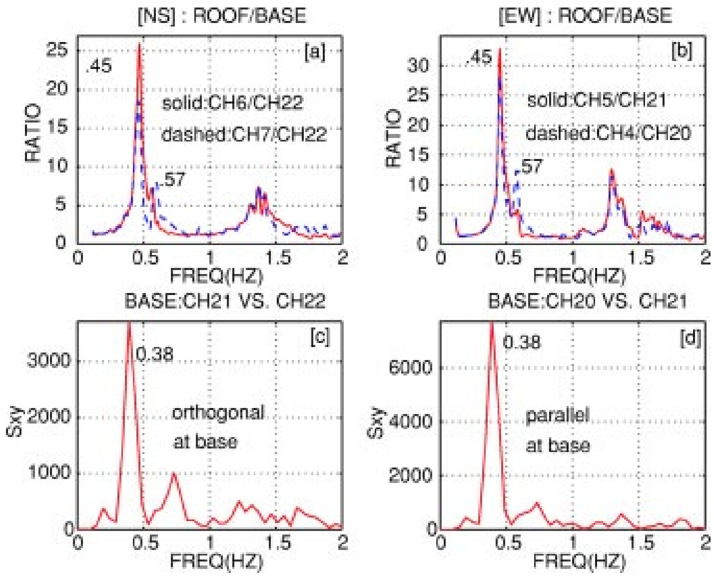

Figure 13. (a, b) Spectral ratios of roof/basement motions indicate structural frequencies (translational [0.45 Hz] and torsional [0.57 Hz]), and (c, d) cross-spectrum of basement motions indicate site frequency (0.38 Hz).

Hz) also appears in the cross-spectra of orthogonal (CH21 and CH22) and parallel (CH20 and CH21) pairs of motions at the base; hence, indicating that it is site related and not structural related.

In addition to the identification using the 1989 Loma Prieta earthquake records, the site peaks are also identified from records obtained from this building during the 1984 Morgan Hill and 1986 Mt. Lewis earthquakes (Figure 11). For the sake of brevity and without providing repetitious plots (as in Figures 12 and 13), the site frequencies as well as the structural frequencies (and damping) determined from the records are summarized in Table 2 (Çelebi 1994, 1998). It is noted herein that observation of site frequencies at this building location from repeated earthquakes provides exceptionally similar frequencies.

Limited geological logs (Earth Sciences Associates 1971) available allow approximate calculation of site transfer function using estimated shear-wave-velocities and depth to bedrock estimated to be anywhere between 270 and 500 m. Figure 14 shows that the site, in the Santa Clara (California) basin, is capable of generating motions with periods between 2 and 3 seconds, depending on the assumed depth to bedrock. The long-period site characterization is also confirmed by a study of the basin effect in Santa Clara by Frankel and Vidale (1992), who concluded that 2–5-second long-period motions can be generated in this particular basin.

Table 2. Accelerations and structural and site frequencies as determined from records (MHE–Morgan Hill earthquake, MLE–Mt. Lewis earthquake, LPE–Loma Prieta earthquake)

		Earthquake		
	Location	MHE	MLE	LPE
Acceleration (Peak) (g)	Roof (NS)	0.17	0.32	0.34
	Roof (EW)	0.17	0.37	0.34
	Base (NS)	0.04	0.04	0.10
	Base (EW)	0.04	0.04	0.09
Translational	Frequency (HZ)	0.46	0.48	0.45
	Period (s)	2.17	2.08	2.22
	Damping (%)	1.98	2.10	2.70
Torsional	Frequency (HZ)	0.59	0.58	0.57
	Period (s)	1.70	1.72	1.69
Site	Frequency (HZ)	.8–.40	.38–.40	0.38

CASE 4: EMBARCADERO BUILDING (EMB), SAN FRANCISCO, CALIFORNIA

Figure 15 shows a three-dimensional view and the instrumentation scheme of the Embarcadero Building (EMB) in San Francisco.

Figures 16a and b show the acceleration responses recorded at the roof and basement of EMB in the NS and EW directions, respectively. The normalized amplitude spectra of

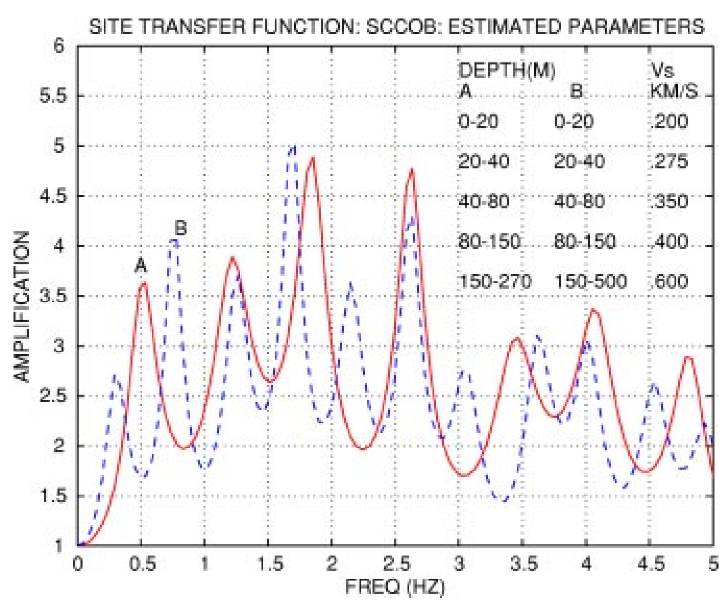

Figure 14. Site transfer functions for two postulated depths to bedrock (SCCOB) site.

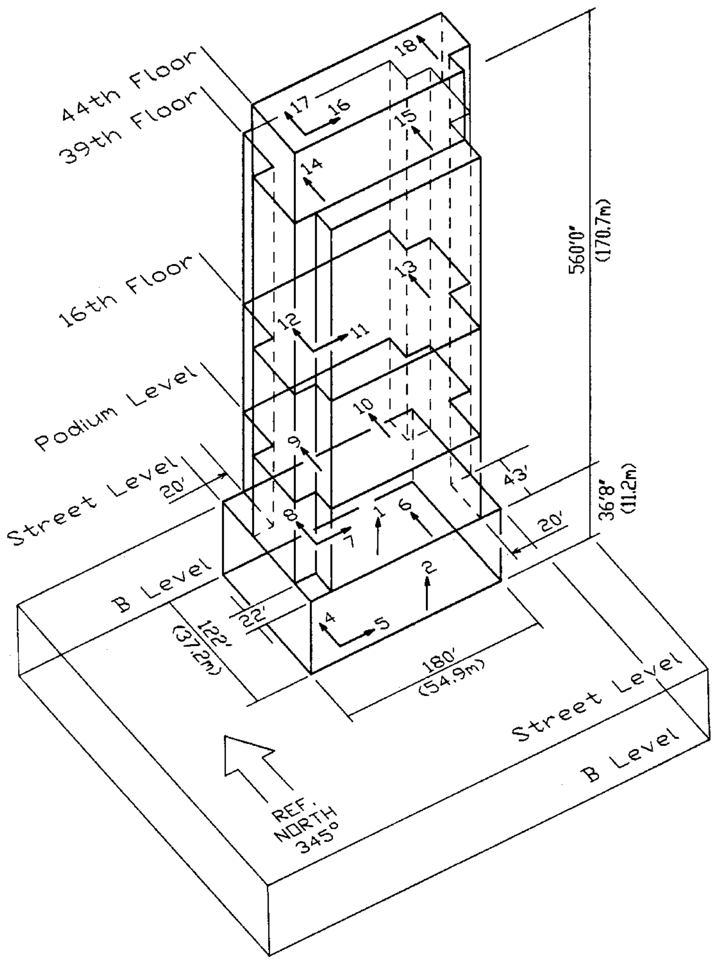

Figure 15. Three-dimensional schematic of Embarcadero Building (EMB) and its instrumentation scheme.

these motions are depicted in Figures 16c and d. The reason as to why these spectra are normalized is to show the significant peaks of both the roof and basement in the same plot. Otherwise, since the building is tall, the basement spectra would not clearly be seen if they are plotted on an equal scale. The fundamental frequencies (periods) of the building are identified as 0.19 Hz (5.26s) in the NS and 0.16 Hz (6.25s) in the EW directions respectively. Detailed analyses of recorded data from this building are presented by Astaneh et al. (1991) and Çelebi (1993a). Figures 16e and f show the ratios of amplitude spectra of pairs of roof and basement motions. The site frequency (period) at 0.7–0.8 Hz (1.25–1.43 sec) clearly seen in the normalized amplitude spectra of basement motions cancels out in the spectral ratio plots. In the spectral ratio plots (Figures 16e and f), the

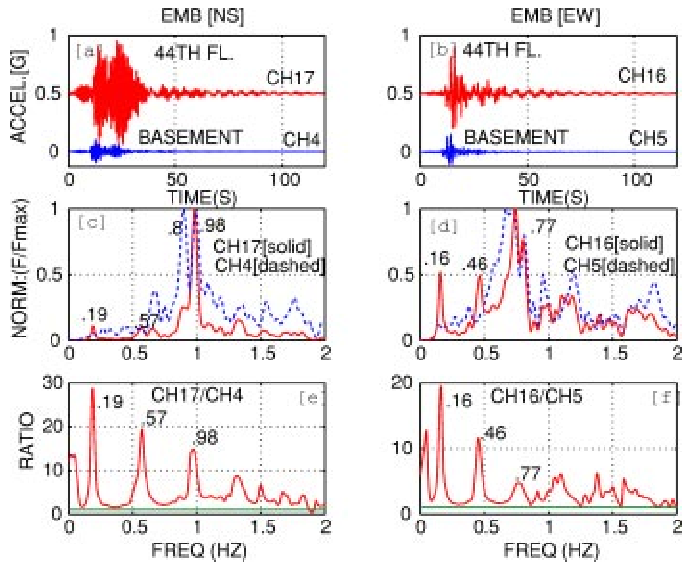

Figure 16. (a, b) Time histories of roof and ground floor acceleration pairs of EMB in the NS and EW directions, (c, d) corresponding normalized amplitude spectra, and (e, f) corresponding roof/ground floor spectral ratios.

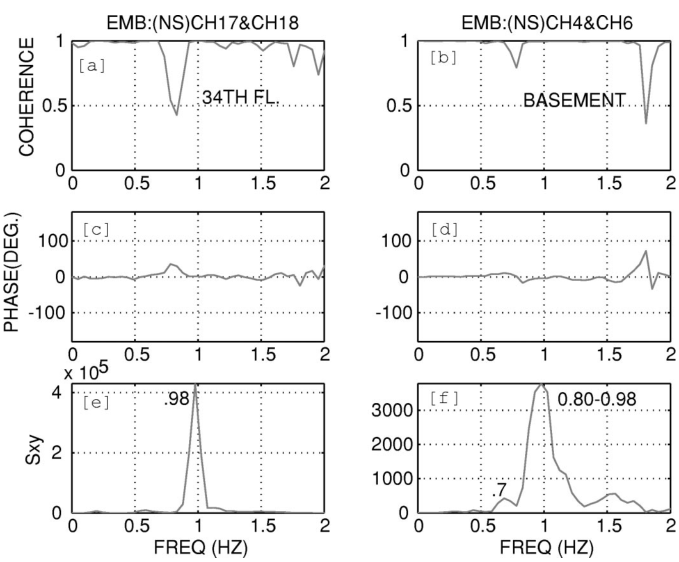

Figure 17. (a, b) Coherence, (c, d) phase angle, and (e, f) cross-spectra plots of pairs of parallel motions at the roof (CH17 and CH18) and basement (CH4 and CH5).

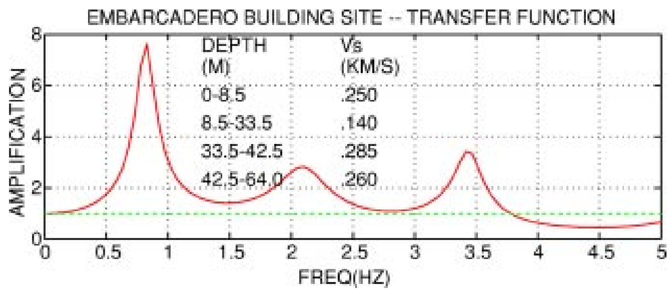

Figure 18. Site transfer function for EMB site.

second (NS: 0.57Hz [1.75 s], EW: 0.46 Hz [1.02 s]) and third (NS: 0.98 Hz and EW: 0.77 Hz) modal frequencies are clearly identifiable. These periods, in general, follow the T, T/3, T/5 rule of thumb.

Figure 17 shows coherence, phase angle, and cross-spectrum plots of the pairs of motions at the roof (NS: CH17 and CH18) and at the base (NS: CH4 and CH6). Cross-spectrum of the pair of roof motions, shown in Figure 17e clearly indicates the third modal frequency (0.98 Hz). The same for basement motions has a much wider frequency band that includes the 0.98 Hz frequency. This implies that the site frequency (~0.7–0.9 Hz) that appears in the amplitude spectrum of the basement motions is very close to the third modal frequency in the NS direction and possibly causes resonance—which may explain why the peak of the third modal frequency is much higher than the peaks of the first and second modes.

The site frequency is 0.7–0.8 Hz (1.25–1.43 sec) and is identifiable in the cross-spectra of basement motions. The site transfer function, presented in Figure 18, confirms this identification. Site characterization data has been adopted from Gibbs et al. (1994b).

CASE 5: NORWALK BUILDING (NOR), NORWALK, CALIFORNIA

Figure 19 shows the extensive instrumentation for the two buildings and the site at 12400 block of Imperial Highway, Norwalk, California. In this study, only Building B (hereby called NOR) and the south free-field (SFF) are considered. Responses of both buildings and three of the four free-field stations were recorded during the 1987 Whittier, California, earthquake (M_s=5.8). Detailed studies of these records are reported elsewhere (Çelebi 1993b, c).

Figures 20a and b show NS and EW accelerations at the roof and the basement of NOR and its south free-field. Figures 20c and d show the corresponding amplitude spectra calculated from these accelerations. Figures 20e and f show the corresponding spectral ratios calculated from the amplitude spectra. The fundamental structural frequencies of Building B are identified as 0.76 Hz (NS) and 0.83 Hz (EW). These structural frequencies are also clearly identified in the normalized cross-spectra ratio (Figure 21a) of parallel motions at the roof and cross-spectra of orthogonal motions at the roof (Figure 21c). The very small peak at 0.3 Hz observed in the amplitude spectra (Figure 20c) as

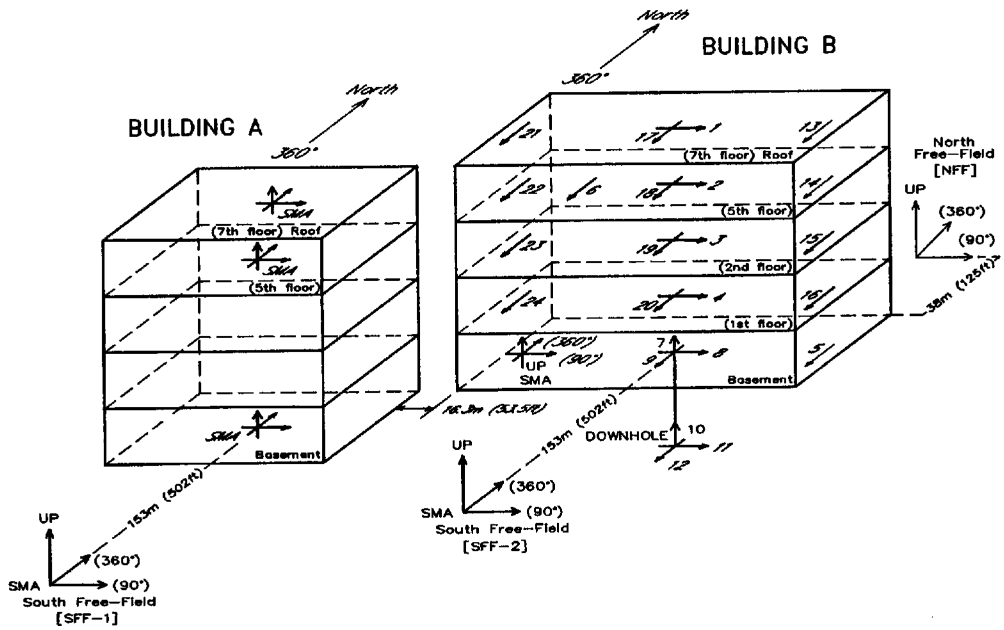

Figure 19. Instrumentation of the two buildings and the site at 12400 block of Imperial Highway, Norwalk, CA. In this study, Building B (NOR) and the south free-field (SFF) are considered.

well as in the basement cross-spectra (Figures 21b and d) is the site frequency. It is noted that this frequency cancels out in the spectral ratio plots (Figures 20e and f). Another possible larger-mode site frequency is detectable at approximately 1.7 Hz. This frequency also cancels out in the spectral ratio plots.

The depth to bedrock in the vicinity of the buildings is not well described; however, logs from existing nearby oil wells indicate that below 500 m depth, well-cemented sandstone and marine rock are prevalent. The site transfer function using estimated shear-wave velocities is shown in Figure 22 and exhibits that the site is capable of generating motions with low frequencies (e.g., 0.3 Hz).

ESTIMATES OF SITE FREQUENCY USING CODE FORMULA

In order to assess the reliability of the extracted site frequencies and to facilitate comparisons, the approximate site formula ($T_s=4H/V_s$) is used to calculate approximate periods of the sites using the site characterization data displayed in the site transfer function plots (Figures 4, 9, 14, 18, and 22). These calculations are summarized in Table 3 and compared with the site frequencies assessed from earthquake records and site transfer functions. It is noted herein that the above simple formula uses an average shear-wave velocity, $V_s(\text{ave})=H/(\Sigma(h_i/V_{si})$, and that while the *Uniform Building Code* (ICBO

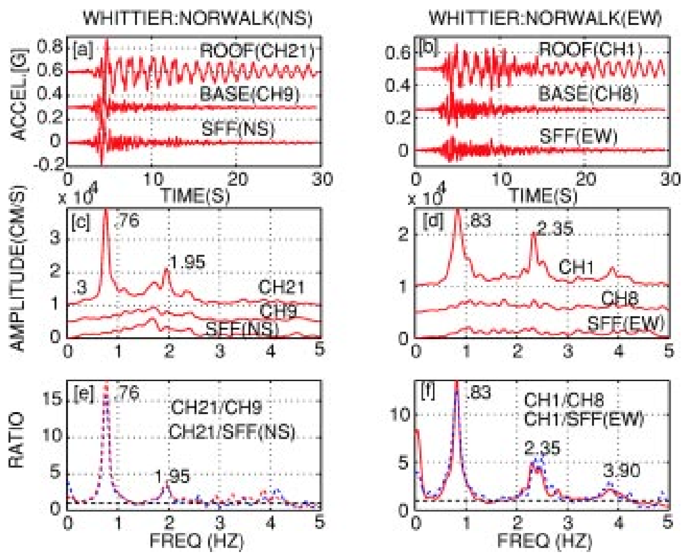

Figure 20. (a, b) NS and EW time histories of roof, basement, and SFF accelerations of NOR building; (c, d) corresponding amplitude spectra; and (e, f) corresponding roof/ground and roof/free-field spectral ratios.

1997) does not state a limitation on the total depth for which this formula can be used, the new *International Building Code* (ICC 2000) limits the use of this relationship up to a depth of ~30 m (100 ft).

DISCUSSIONS AND CONCLUSIONS

A procedure that is used to identify site frequency from building responses recorded during earthquakes has been presented. The procedure distinguishes site frequency among several frequencies that appear in amplitude spectra and cross-spectra of horizontal records from roof and basements of buildings. These results are further substantiated by analyses of additional data from associated free-field records of each building and site transfer function calculated by using site characterization data. In one case, available records from three different earthquakes provide comparatively similar site frequencies as well as structural frequencies. This indicates that identified site frequencies are repeatable for different strong shaking events.

The site frequencies identified from records are compared with simple code formula computation even though the formula, $T=4H/V_s$, may be limited to layered media with depths to 30 meters (100 ft) (ICC 2000). All results presented for five cases are summarized in Table 3 and comparatively plotted in Figure 23. As noted in both Table 3 and Figure 23, the site frequencies identified from building records are higher than those estimated by transfer functions and the simple code formula. The reason for this is possibly due to various assumptions that are introduced into the code formula as well as the transfer function calculation based on one-dimensional linear wave propagation. The

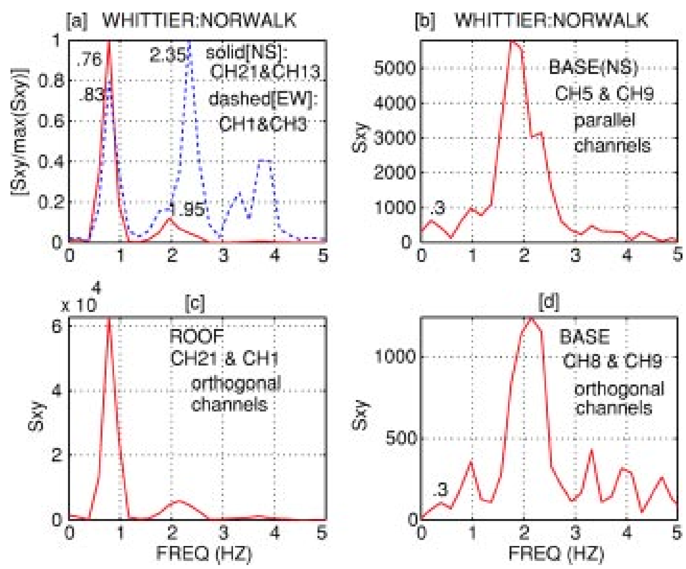

Figure 21. (a) Normalized cross-spectra of pairs of parallel NS accelerations (CH21 and CH13) at the roof and EW accelerations (CH1 and CH3) at the roof and second floor; (b) cross-spectrum of parallel NS accelerations (CH5 and CH9) at the base; (c) cross-spectrum of orthogonal accelerations (CH21 and CH1) at the roof; and (d) cross-spectrum of orthogonal accelerations at the base (CH8 and CH9).

code formula is based on average shear-wave velocity for the total depth considered. On the other hand, the site frequencies identified from the records are most likely the correct ones as they are inferred from actual on-scale response records and, therefore, include any inherent nonlinearities caused by the level of shaking.

The procedure can be used to process numerous sets of accumulated data from instrumented structures and promises to be an effective and simple technique to identify site frequencies from actual building responses recorded during earthquakes.

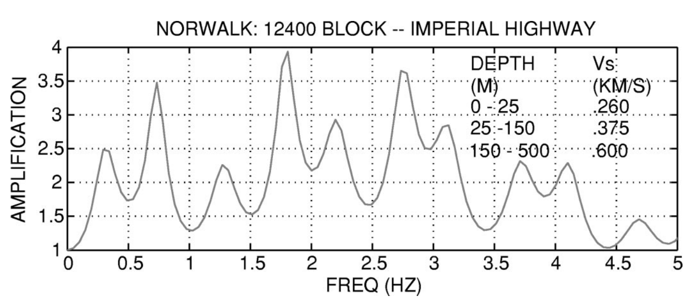

Figure 22. Site transfer function for NOR site.

Table 3. Assessment of site period (frequency) using code formula (V_s[ave]=H/[Σ(h_i/V_{si}]), earthquake records and transfer functions

			OAK		SCCOB			
Method	Parameter H=Σh_i (m)	PPP 150	A 152	B 169	A 270	B 500	EMB 64	NOR 500
Formula	Σ (h_i/Vs_i)	0.46	0.41	0.46	0.66	1.05	0.33	1.01
	Vs(Ave) (m/s)	329.5	366.5	369.6	407.8	478.3	195.8	493.7
	Site T=4H/Vs (s)	1.82	1.65	1.83	2.65	4.18	1.31	4.05
	Site f=1/T (Hz)	0.55	0.60	0.55	0.38	0.24	0.76	0.25
Records	Structural T(s)	2.63	0.51 (trans.), 0.61 (tors.)		2.22 (trans.), 1.75 (tors.)		5.26 (NS) 6.25 (EW)	1.32 (NS) 1.20 (EW)
	Structural f(Hz)	0.38	1.95 (trans.), 1.65 (tors.)		0.45 (trans.), 0.57 (tors.)		0.19 (NS) 0.16 (EW)	0.76 (NS) 0.83 (EW)
	Site T (s)	1.33	1.18–1.25		2.63		1.11–1.42	3.3
	Site f(Hz)	0.7	0.8–0.85		0.38		0.7–0.9	0.3
Transfer Function	Substratum Vs (m/s)	700	1000		1200		1000	1000
	Site T (s)	1.18–1.54	1.43–1.54		2.0–4.0		1.25–1.42	3.3
	Site f(Hz)	0.65–0.85	0.65–0.70		0.25–0.5		0.7–0.8	0.3

The only real difficulty in applying this procedure could arise when and if the site and structural frequencies are identical or if they are too close to one another, in which case further examination of the data by other procedures can be applied or estimates of free-field data or site data can be used to clarify the situation.

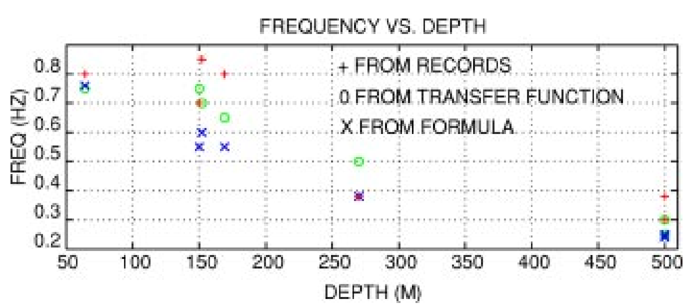

Figure 23. Variation of site frequency with depth using three methods.

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