Fifth U.S. National Conference on Earthquake Engineering, Earthquake Awareness and Mitigation Across the Nation, Vol. 1, 1994

ANALYTICAL MODELING FOR SOIL-STRUCTURE INTERACTION OF A 6-STORY COMMERCIAL OFFICE BUILDING

L.T. Phan[1], E.M. Hendrikson[1], R.D. Marshall[2], and M. Çelebi[3]

ABSTRACT

Strong-motion and ambient vibration data obtained from a 6-story commercial office building in San Bruno, California, were analyzed. Comparison of dynamic characteristics revealed that the first-mode response frequency deduced from the Loma Prieta earthquake records is significantly lower than that deduced from ambient vibration data, and the damping ratio for strong motion is higher than that obtained from ambient vibration. A computer model of the building was developed and analyzed using two boundary conditions. The fixed-base condition was used to simulate the building response to ambient vibration, and the spring-supported condition was used to incorporate soil-structure interaction and thus simulate realistic building response to the Loma Prieta earthquake. Results of analyses showed that the first-mode response frequencies for the two cases differ by essentially the same factor observed from measurements. This suggests that the difference in first-mode response frequencies between ambient vibration and strong motion in this building was due largely to soil-structure interaction.

Introduction

Approximately one year after the Loma Prieta earthquake (LPE) of October 17, 1989, an ambient vibration test program was initiated by the National Institute of Standards and Technology (NIST) in collaboration with the United States Geological Survey (USGS) to study the dynamic characteristics of five buildings in the San Francisco Bay area. The five buildings selected are: (1) the California State University Administration Building at Hayward, (2) the Santa Clara County Office Building in San Jose, (3) the Commercial Office Building in San

[1] Research Structural Engineer, Building and Fire Research Laboratory (BFRL), National Institute of Standards and Technology (NIST), Gaithersburg, MD 20899
[2] Leader, Structural Evaluation Group, BFRL, NIST, Gaithersburg, MD 20899
[3] Civil Engineer, Branch of Engineering Seismology and Geology, U.S. Geological Survey, Menlo Park, CA 94025

Bruno, (4) the Transamerica Building in San Francisco, and (5) the Pacific Park Plaza Building in Emeryville [Marshall, 1991 and 1992]. All five buildings sustained no visible structural damage during the LPE. It was observed that damping ratios computed from the LPE records, ζ_{LPE}, are always higher than those from ambient vibration recordings, ζ_{AMB}, and response frequencies determined from the LPE records (f_{LPE}) are lower than those from ambient vibration (f_{AMB}). The most pronounced difference in the measured frequencies is found in the San Bruno Commercial Office Building ($f_{LPE}/f_{AMB} = 0.69$). Since stiffness is proportional to the square of frequency, the corresponding ratio of stiffness is 0.48. This significant difference calls into question the appropriateness of using ambient vibration test results for seismic design purposes. Thus, the cause of the observed frequency difference is the subject investigated in this paper.

Description of The San Bruno Commercial Office Building

The San Bruno Commercial Office Building is a 6-story, reinforced concrete moment-frame structure, rectangular in plan (61.26 m (N-S)x 27.13 m (E-W)). The total building height is 24 m. All six floors are above ground level. Figure 1 shows a view of this building to the southeast. There are 13 frames in the building. The sixth frame from the south is made up of larger, more heavily reinforced beams and columns which results in the building having almost the same lateral stiffness in the two plan directions, despite the plan aspect ratio of more than 2 to 1. A typical floor plan is shown in Figure 2. Perimeter columns are cast-in-place reinforced concrete, encased by precast wall panels of irregular shape, which results in columns with composite cross sections that are roughly trapezoidal in shape. These encasing wall panels appear to be a proprietary design and little is known as to how they interact with the cast-in-place columns. Interior columns have square cross sections (0.51 m x 0.51 m). Typical perimeter column cross sections are shown in Figure 3. All columns are supported by individual spread footings.

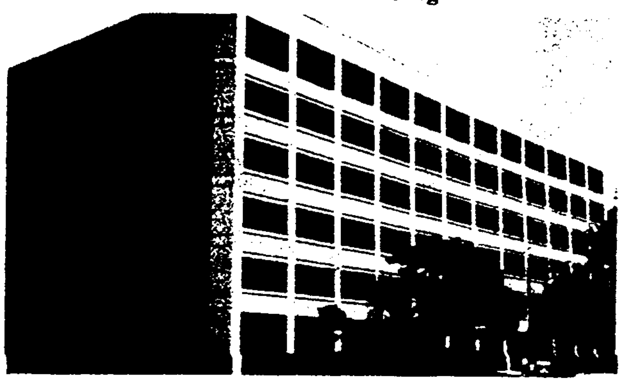

Figure 1. A View of the Commercial Office Building

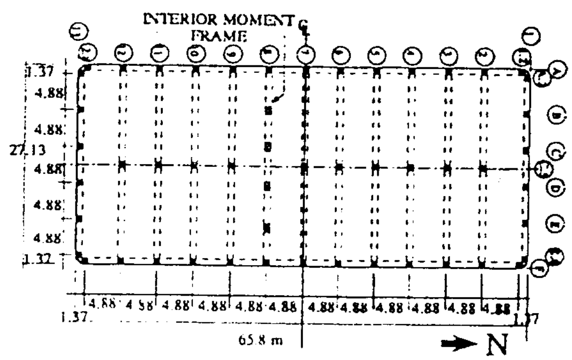

Figure 2. Typical Floor Plan

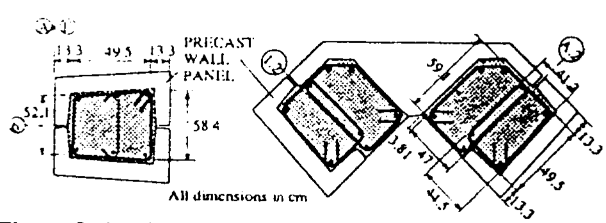

Figure 3. Typical Perimeter Column Cross Section

The perimeter beams are cast-in-place rectangular sections. The interior beams are precast, post tensioned with a span of 12.8 m. The floor slabs are 0.14 m thick reinforced concrete. The ground floor is a 0.13 m-thick slab on grade. A steel-framed mechanical penthouse floor, 29.3 m x 10.8 m in plan and 4.0 m in height, is located at the center of the roof. The estimated mass of the penthouse plus mechanical equipment is 19,000 kgf.

Existing Strong-Motion Instrumentation

There were 13 existing force balance accelerometers (FBA) in this building (see Figure 4). Ambient vibration recording were carried out by connecting these FBAs to the NIST's portable data acquisition system. Data were recorded at a rate of 50 Hz per channel.

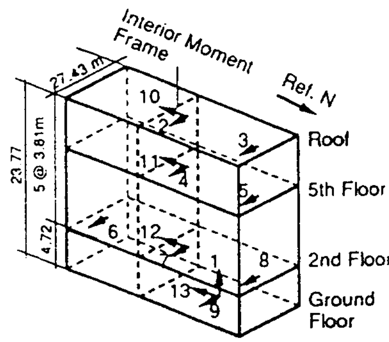

Figure 4. Building Instrumentation Scheme

Structural Response to Strong-Motion (LPE)

An acceleration history of a roof-level FBA (#10) and Fourier amplitude spectra of two roof-level FBAs (#10 and 2) are shown in Figures 5 to 7. In the N-S direction, a peak acceleration of 240.9 cm/sec^2 was recorded for the center of the roof 11.62 seconds after triggering. The Fourier amplitude spectrum for this location shows four peaks at 0.76, 1.17, 1.37, and 1.87 Hz, respectively. The first peak of 0.76 Hz has been identified as the soil system resonance frequency, and the peak at 1.17 Hz has been identified as the first N-S translational mode [Marshall et al., 1992]. The damping ratio, calculated by the system identification technique for the first N-S translational mode, is 7.2 percent of critical damping [Çelebi et al., 1991, 1993].

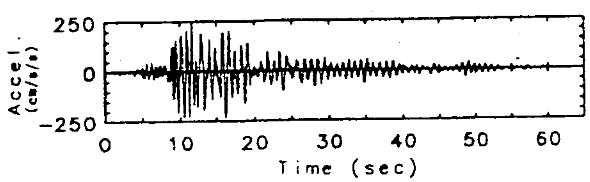

Figure 5. Acceleration at Roof Center (NS) Due to the LPE

In the E-W direction, a peak acceleration of 309.8 cm/sec^2 was recorded at the center of the roof 13.82 seconds after triggering, and a peak acceleration of 443.0 cm/sec^2 was recorded at the north end of the roof 14.3 seconds after triggering. Three spectral peaks are identifiable on the Fourier spectra. The 0.6 Hz peak appears to be due to soil system resonance. The 0.98 Hz peak is identified as the first E-W translational mode. The 1.32 Hz peak is identified as a torsional mode. The damping ratio (for first E-W translational mode) is 4.1 percent of critical damping .

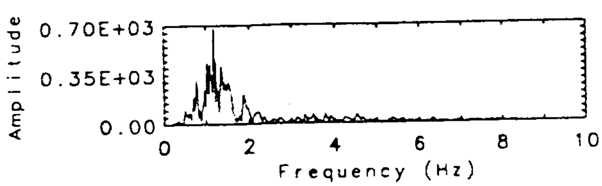

Figure 6. Fourier Spectrum at Roof Center-NS

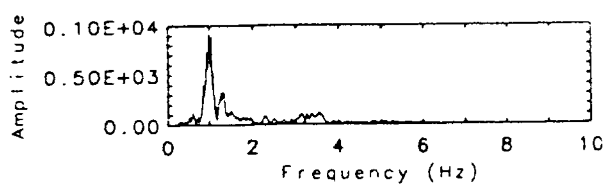

Figure 7. Fourier Spectrum at Roof Center-EW

Structural Response to Ambient Vibration

Figures 8 to 10 show 42-seconds of ambient vibration data and the corresponding Fourier amplitude spectra recorded by the same two roof-level FBAs discussed above. For the N-S direction, the Fourier spectrum shows a major spectral peak at 1.72 Hz. This frequency has been identified as the first N-S translational mode. For the E-W direction, both Fourier spectra at the center and at the north end of the roof show two identifiable spectral peaks, at 1.41 and 1.95 Hz. The 1.41 Hz peak is judged to be the first E-W translational mode and the 1.95 Hz peak is identified as a torsional mode.

Logarithmic decrements (see Figure 11) of the autocorrelations yield overall structural damping ratios, ζ_{AMB}, of 2.2 and 2.3 percent of critical in the N-S and E-W directions, respectively. Table 1 summarizes the measured dynamic properties of this building.

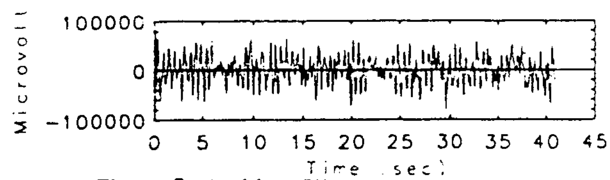

Figure 8. Ambient Vibration Response at Roof Center-NS

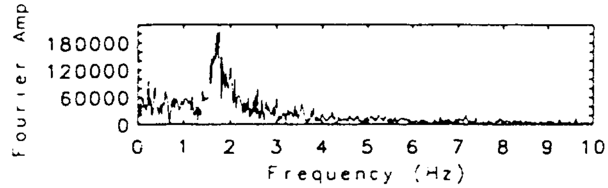

Figure 9. Fourier Spectrum at Roof Center-NS

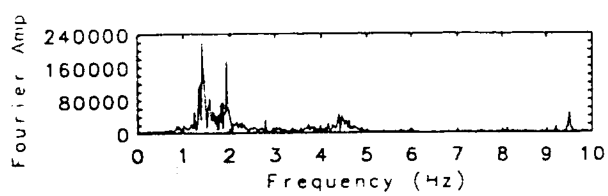

Figure 10. Fourier Spectrum at Roof Center-EW

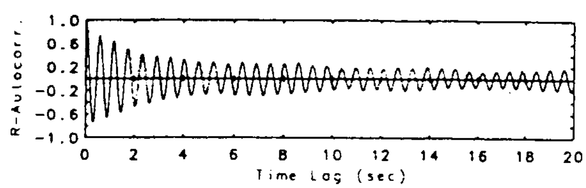

Figure 11. Autocorrelation of Ambient Vibration Response at Roof Center-NS

TABLE 1. SUMMARY OF FIRST-MODE RESPONSE OF THE BUILDING

Mode Description	Loma Prieta		Ambient Vibration		Frequency Ratio
	f_{LPE}(Hz)	ζ_{LPE}^{1}(%)	f_{AMB}(Hz)	ζ_{AMB}^{2}(%)	(f_{LPE}/f_{AMB})
First E-W Translation	0.98	4.1	1.41	2.3	0.70
First N-S Translation	1.17	7.2	1.72	2.2	0.68

[1] Obtained by system identification technique.
[2] Obtained by logarithmic decrement of auto-correlation function.

Computer Model Of The San Bruno Office Building

Modeling of Structural Systems

A 3-dimensional computer model was created by discretizing all structural components into finite elements [Phan et al., 1992]. Columns and beams were modeled using two-noded bar elements whose section properties were calculated using transformed, uncracked sections. Concrete modulus of elasticity of 28,090 MPa was used based on a designed f_c of 34.5 MPa. Because little is known about the properties of the precast panels which encased the perimeter columns and how they are connected, only the cast-in-place portion of the perimeter columns was used in computing section properties. Thus, if the encasing panels were effectively connected to and performed integrally with the cast-in-place columns, then the building stiffness would be underestimated by the model. Floor and roof slabs were modeled using 4- and 3-noded plate elements. These elements were given actual slab thickness to accurately model the stiffness, and modified mass densities were used to account for the floor and roof loads. The penthouse was modeled as 22 added masses located at the bases of the penthouse columns (18 masses of 907 kgf each and 4 masses of 680 kgf each for a total estimated mass of 19,000 kgf).

Modal Analyses of Comp uter Model

Modal analyses were performed to compute the natural frequencies and associated mode shapes of the model. Two sets of boundary conditions, fixed-base and spring-supported, were used. To perform the analyses, a large mass was placed at the geometric center of the building at ground level. This mass was linked to the model at the base of each column by rigid link elements. The rigid link elements were used to transmit motion resulting from the application of acceleration histories at the artificial mass to the upper structure. The magnitude of this mass was selected so that the response accelerations calculated for column bases would not differ significantly from the acceleration record used to drive the mass. Idealized models with large mass, rigid links, and springs are shown in Figures 12 and 13.

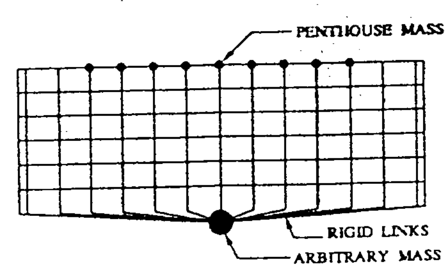

Figure 12. Fixed-Base Model

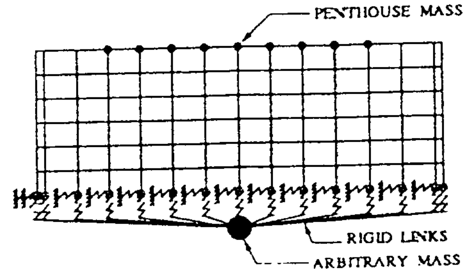

Figure 13. Spring-Supported Model

Fixed-base Model

This boundary condition is thought to simulate best the ambient vibration condition since soil-structure interaction is ignored. The first five natural frequencies obtained from modal analyses are listed in Table 2. The first mode frequency of 1.217 Hz is identified as the E-W translational mode. The second mode of 1.283 Hz is N-S translation. The third, 1.599 Hz, is a torsional mode. The remaining modes of 3.74 Hz and 3.99 Hz are E-W and N-S translation combined with floor twisting.

TABLE 2. NATURAL FREQUENCIES OF THE FIXED-BASE MODEL

Mode No.	Natural Frequencies (Hz)	Mode description
1	1.217	E-W Translation
2	1.283	N-S Translation
3	1.599	Torsion
4	3.740	E-W Translation with Floor Twisting
5	3.990	N-S Translation with Floor Twisting

The modal frequencies of the fixed-base model may be compared to frequencies obtained from ambient vibration measurements (Table 1). It is observed that the analytically obtained frequencies, on average, are approximately 25 percent lower than those obtained from measurements. It is thought that refinement of the model to include the stiffnesses of the precast panels would certainly result in better agreement between the predicted and the measured frequencies. However, since the purpose of this study is to identify the possible cause (or causes) of the reduction in response frequencies due to strong motion (LPE), no refinement of the model was attempted. Instead, the same model, only with different boundary condition (spring-supported model), was developed to model the San Bruno Building under strong motion.

Spring-Supported Model

For the spring-supported model, a set of discrete, two-noded spring elements consisting of one vertical and two lateral springs with finite lengths and stiffnesses was created at the base of each column. The results of this model are to be compared with those of the fixed-base model to allow an assessment of the effect of soil-structure interaction on the response frequencies. For the vertical springs, a finite extensional stiffness was given to model the modulus of subgrade reaction. This stiffness was computed by dividing the average load supported by each footing by the estimated maximum subgrade settlement (193,000 kgf/4.2 mm equals 45,714 kgf/mm) [Bowles, 1977]. For the lateral springs, both extensional and rotational stiffnesses were computed. The building lateral stiffness at the foundation level ($k = 12EI/L^3$) was computed to be 3.04×10^6 kN/m (N-S) and 3.00×10^6 kN/m (E-W). The extensional stiffness of each lateral spring was then computed by dividing the total stiffness by the number of columns per floor (63,292 kN/m for N-S springs and 62,542 kN/m for E-W springs). For rotational stiffness, because of many uncertainties involving the estimation of this quantity, five different rotational spring stiffnesses, ranging from a low value of 10^3 kN-m/radian to a high value of 10^8 kN-m/radian, were used. The modal analyses results are shown in Table 3.

From Table 3, it may be observed that varying the rotational stiffnesses of the spread footings, even by a factor of 10^5, results in only minor change in the first mode frequency of the spring-supported model (3%). Thus, even though a precise value of rotational stiffness could not be computed, its effect on the response frequency is insignificant. However, the effect of

TABLE 3. NATURAL FREQUENCIES OF THE SPRING-SUPPORTED MODEL

Mode No.	Rotational Stiffnesses (kN-m/rad) $1\text{x}10^8$	$1\text{x}10^7$	$1\text{x}10^6$	$0.5\text{x}10^6$	$1\text{x}10^3$	Mode Description
1	0.895	0.873	0.871	0.871	0.870	E-W Translation
2	0.932	0.910	0.908	0.908	0.908	N-S Translation
3	1.153	1.131	1.129	1.129	1.129	Torsion
4	3.064	3.035	3.032	3.032	3.032	E-W Bending/Floor Twisting
5	3.238	3.211	3.210	3.208	3.208	N-S Bending/Floor Twisting

the modulus of subgrade reaction on the response frequency appears to be pronounced, as can be seen by comparing the results in Table 3 with the results of the fixed-base model (Table 2). Assuming a rotational stiffness of 10^3 kN-m/rad for the base columns, the first mode (E-W translation) frequency ratio between the spring-supported condition and the fixed-base condition, f_s/f_f, is 0.72 (0.87/1.22). For the second mode (N-S translation), this frequency ratio is 0.71 (0.91/1.28). These analytically obtained frequency ratios (f_s/f_f) can be compared with the measured frequency ratios between the LPE and the ambient vibration (f_{LPE}/f_{AMB}) listed in Table 1. From Table 1, the first mode (E-W translation) f_{LPE}/f_{AMB} ratio is 0.70 which is approximately 3% different from the first mode f_s/f_f ratio (0.72), and the second mode (N-S translation) f_{LPE}/f_{AMB} ratio is 0.68 which is approximately 4% different from the second mode f_s/f_f ratio (0.71). The good agreement between the measured and analytical frequency ratios indicates that, for this building, the observed reduction in frequency between ambient vibration and strong motion may be attributed in large part to the effect of soil-structure interaction.

Transient Dynamic Analyses of Computer Model

Transient dynamic analyses were performed to (1) study the sensitivity of the first-mode frequency to changes in overall structural damping, and (2) study the sensitivity of the first-mode frequency to different assumptions of building rotational stiffnesses. The model was excited using the E-W component of acceleration recorded at the center of the ground floor during the LPE. The first five modes obtained from modal analyses were used for mode superposition.

Fixed-base Model

Four different damping ratios (0.5, 3.0, 7.0, and 10.0 percent) were used in the transient dynamic analyses of the fixed-base model. Figures 14 to 17 show the E-W acceleration histories and corresponding Fourier spectra of nodes located at the center and at the north end of the roof for the case of 7.0 percent damping. The first-mode frequencies were determined from the half-power bandwidth method and are listed in Table 4.

The response acceleration history at the center of the roof compare well with the building responses recorded during the LPE (Figure 5). For each of the four different damping ratios, the elapsed time to maximum acceleration at the center of the roof is between 13 and 14 seconds, which compares well with 13.82 seconds observed in the LPE.

TABLE 4. FIRST MODE FREQUENCY WITH VARYING DAMPING RATIOS FIXED-BASE MODEL

	Damping Ratios (% of Critical)			
	10.0	7.0	3.0	0.5
First Mode Frequencies (Hz)	1.212	1.216	1.227	1.235

At the north end of the roof, the elapsed time ranges from 14 to 15 seconds which compares well with the observed 14.3 seconds for the LPE. Furthermore, the peak accelerations at the north edge are higher than those at the center of the roof; similar to the response of the San Bruno Building for the LPE. This demonstrates the ability of the model to simulate the three-dimensional behavior, i.e. the torsional response, of this building. Moreover, from the Fourier spectra, it is apparent that damping ratios have little effect on the first mode response frequency (1.235 Hz with 0.5 percent damping vs. 1.212 Hz with 10 percent damping, or a difference in response frequency of approximately 2%).

Spring-Supported Model

The results of modal analyses of the five cases listed in Table 3, where the rotational spring constants were varied between 10^3 to 10^8 kN-m/radian, were utilized in the modal superposition process for transient analyses. An overall damping ratio of 7 percent of critical was used in all five cases. The resulting first mode response frequencies corresponding to the five cases are listed in Table 5. The response

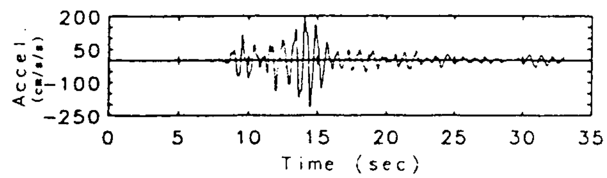

Figure 14. EW Acceleration History at Roof Center 7% Damping, Fixed-Base Model

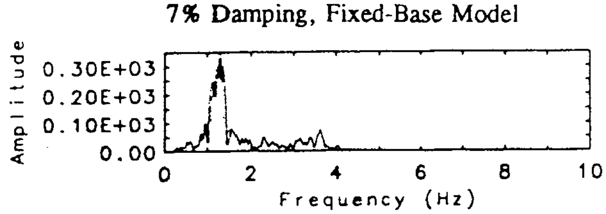

Figure 15. Fourier Spectrum of EW Acceleration at Roof Center, 7% Damping, Fixed-Base Model

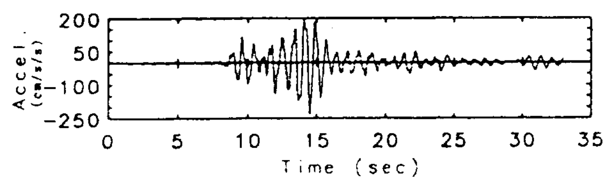

Figure 16. EW Acceleration History at North End of Roof, 7% Damping, Fixed-Base Model

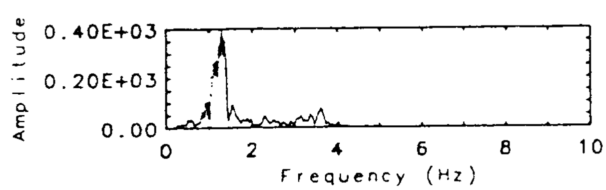

Figure 17. Fourier Spectrum of EW Acceleration at Roof's North End, 7% Damping, Fixed-Base Model

acceleration history and Fourier spectrum corresponding to the case of 10^3 KN-m/radian rotational stiffness and 7 percent damping are shown in Figures 18 and 19. As can be seen from Table 5, different assumptions of building rotational stiffnesses at the ground level appear to have little effect on the first-mode response frequency of this building.

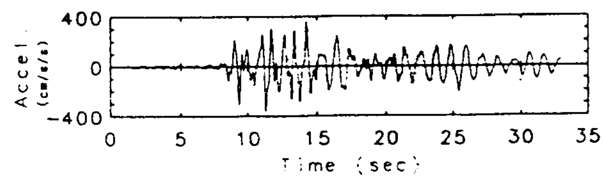

Figure 18. EW Acceleration at Roof Center, Spring-Supported Model

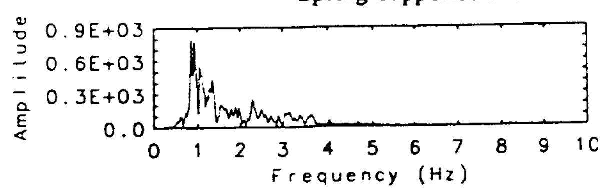

Figure 19. Corresponding Fourier Spectrum

TABLE 5. FIRST MODE FREQUENCY WITH VARYING ROTATIONAL STIFFNESS

	Rotational Stiffnesses (kN-m/rad)				
	$1\text{x}10^8$	$1\text{x}10^7$	$1\text{x}10^6$	$0.5\text{x}10^6$	$1\text{x}10^3$
First Mode Frequencies (Hz)	0.893	0.873	0.869	0.869	0.867

Summary and Conclusions

The measured responses of the building to the LPE and to ambient vibrations are presented and compared. A significant difference in the first and second mode response frequencies between ambient vibration and the LPE was observed. The measured frequency ratio, f_{LPE}/f_{AMB}, is 0.70 for the first mode response (E-W translation), and is 0.68 for the second mode response (N-S translation). Damping estimates deduced from ambient vibration response records by auto-correlation techniques are always smaller than those deduced from the LPE response records by system identification techniques (the ζ_{LPE}/ζ_{AMB} ratios range between 1.0 to 19.3).

A computer model of the building was developed and analyzed using two sets of boundary conditions; fixed-base and spring-supported conditions. Subsequent modal analyses showed that the models underestimated the measured response frequencies of the building by approximately 25 percent. This difference may be attributed to several factors, one of which is the exclusion of the precast wall panels that formed the exterior columns. Better agreement might be obtained with refinement of the model to include these panels. However, since the interest here is in the relative difference in response frequency between the LPE and ambient vibration, refinement of the model was not performed. Comparisons between the response frequencies of the two models indicate an analytical frequency ratio equal to the observed frequency ratio obtained from measurements. Further, transient dynamic analysis of the models with varying damping ratios and rotational stiffnesses indicates that these factors have little effect on the response frequency of the building. Thus, it is concluded that, for this particular building, the frequency difference resulting from ambient vibration and from the LPE is due mainly to soil-structure interaction.

The following conclusions are drawn from the current study:

- For this building, soil-structure interaction appears to be the primary reason for the frequency difference observed from ambient vibration and from the LPE.

- Damping ratios within the range of 0.5 to 10.0 percent have little effect on the response frequencies of the building.

- The linear elastic computer model of the San Bruno Commercial Office Building was developed primarily to study the relative effects of various assumptions and factors affecting the dynamic response of the building, and in this sense the model was successful. However, the model did not include elements to represent the precast panels which encased the exterior columns. This appears to have resulted in lower stiffness in the model than in the real building, which in turn results in a difference of approximately 25 percent in modal frequencies. This serves to illustrate that careful attention must be paid to specific details of the building in the modeling process to obtain the required accuracy if the interest is in the absolute values of the dynamic properties of the building.

References

Bowles, J.E., "Foundation Analysis and Design," 2nd Edition, McGraw-Hill Book Co., 1977.

Çelebi, M.; Phan, L.T.; Marshall, R.D., "Dynamic Characteristics of Five Buildings During Strong and Low-Amplitude Motions", International Journal of Structural Design of Tall Buildings, Volume 2, 1-15, 1993.

Çelebi, M., Phan, L.T., Marshall, R.D., "Comparison of Responses of a Select Number of Buildings to the 10/17/1989 Loma Prieta (California) Earthquake and Low-Level Amplitude Test Results," NIST SP 820, Proceedings of the 23RD Joint Meeting of the U.S.-Japan Coorperative Program in Natural Resources, Panel on Wind and Seismic Effects, p. 475-499, September 1991.

Marshall, R.D., Phan, L.T., Çelebi, M., "Measurement of Structural Response Characteristics of Full-Scale Buildings: Comparison of Results from Strong-Motion and Ambient Vibration Records", NISTIR-4884, National Institute of Standards and Technology, 1992.

Marshall, R.D., Phan, L.T., Çelebi, M., "Measurement of Structural Response Characteristics of Full-Scale Buildings: Selection of Structures", NISTIR-4511, National Institute of Standards and Technology, February, 1991.

Phan, L.T.; Hendrikson, E.M.; Marshall, R.D., "Measurement of Structural Response Characteristics of Full-Scale Buildings: Analytical Modeling of the San Bruno Commercial Office Building", NISTIR-4782, National Institute of Standards and Technology, March, 1992.