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STRONG-MOTION INSTRUMENTATION AND STRUCTURAL RESPONSE OF ATWOOD BUILDING IN DOWNTOWN ANCHORAGE, ALASKA

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SUMMARY

This paper describes the strong-motion instrumentation of the Atwood Building (AB) in downtown Anchorage and results of data analyses for ambient background input and low level earthquake shaking. This 20-story office building is underlain by an approximately 100-foot thick cohesive facies of Bootlegger Cove Formation (BCF). Due to loss of strength during cyclic loadings caused by earthquake shaking, the sensitive cohesive facies of BCF was responsible for the catastrophic landslides occurred in Anchorage during the 1964 Great Alaska earthquake.

There are 32 channels of accelerometers installed at 10 levels in AB to monitor its seismic responses, including torsional effects and rocking behavior. Besides ambient vibration measurements, an earthquake (M_L = 3.7) on Dec. 15, 2003 triggered the building sensors. The epicenter of this event was located 18.6 km from downtown Anchorage. Analyses of the data from ambient test and earthquake shaking were carried out by conventional Fourier techniques to identify dynamic characteristics of the structure. The identified dynamic properties from ambient test and earthquake shaking agree well with each other. We anticipate that this instrumented building will provide valuable data in the future for the investigation of soil-structure interaction (SSI).

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INTRODUCTION

Although Soil-Structure Interaction (SSI) has been an area of research for a long time, it was confined to a limited understanding due to lack of enough strong motion records and adequate instrumental coverage in soils as well as in the structures. Therefore, many critical questions concerning the dynamic response and behavioral characteristics of soil-structure interaction (SSI) in an in-situ environment remained to be resolved. Simplified substructure-based SSI provisions are included in the National Earthquake Hazard Reduction Program (NEHRP 2000) and Applied Technology Council (ATC) reports (ATC [1]). However, these provisions need to be calibrated on site specific basis. The best experimental method for investigating SSI and the overall dynamic response is to monitor the earthquake response in and around buildings and conduct comprehensive full-scale tests of structures (Trifunac [2]).

This paper describes the strong-motion instrumentation of a high-rise (20-story) building in downtown Anchorage and results of analyses of data from ambient testing and earthquake (M_L=3.7) shaking. At first, the general information regarding the building and its site is described. Next, the strong-motion instrumentation is described in detail. Data from ambient testing and earthquake shaking is shown. Finally results of data analyses are presented.

BUILDING DESCRIPTION

This 20-story state office building (officially named the Robert B. Atwood Building) is located in downtown Anchorage. The building is a steel moment-resisting frame structure with reinforced-concrete spread footing. Figure 1 shows the elevation and plan views of its design. Figure 2 shows a picture of the building and borehole array site in close proximity to the building (inside a public park). Instrumentation of the downhole array is in progress. Extensive site investigation has been conducted at the building site. Nine boreholes with the depth varing from 40 ft to 102 ft were drilled and logged befoe the construction of the building. Two test holes were drilled and sampled to determine the soil properties in 1981.

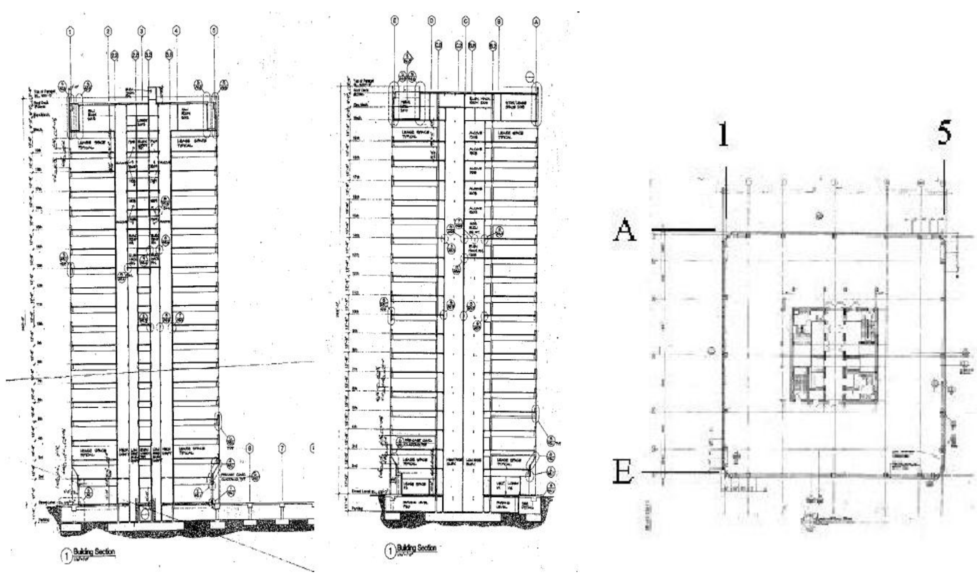

Figure 1. Elevation view and plan view of the Atwood Building.

Shear wave velocity profile at the above site is shown in Figure 3. Roughly, the site can be divived into five layers. The first layer is about 35 ft thick and consists of poorly graded sand and gravel. The second layer extends from 35 ft to 53 ft and is mainly composed of silt and lean clay. Unlike the first two layers, the third layer reaches 147 ft depth and is mainly composed of soft, locally sensitive clay and silty clay. This layer is also called the Bootlegger Cove Formation (BCF) (Updike and Carpenter [3]). Below 140 ft, the soil becomes very stiff with layered silt, sand and clay. Starting from 171 ft, older, dense and hard glacial till is encountered which extends to the bottom of drilled hole. It is may be noted that this is a typical site condition in downtown Anchorage area.

Figure 2. View of the Atwood Building from nearby Delany Park. The white fenced area in this park shows the downhole array site.

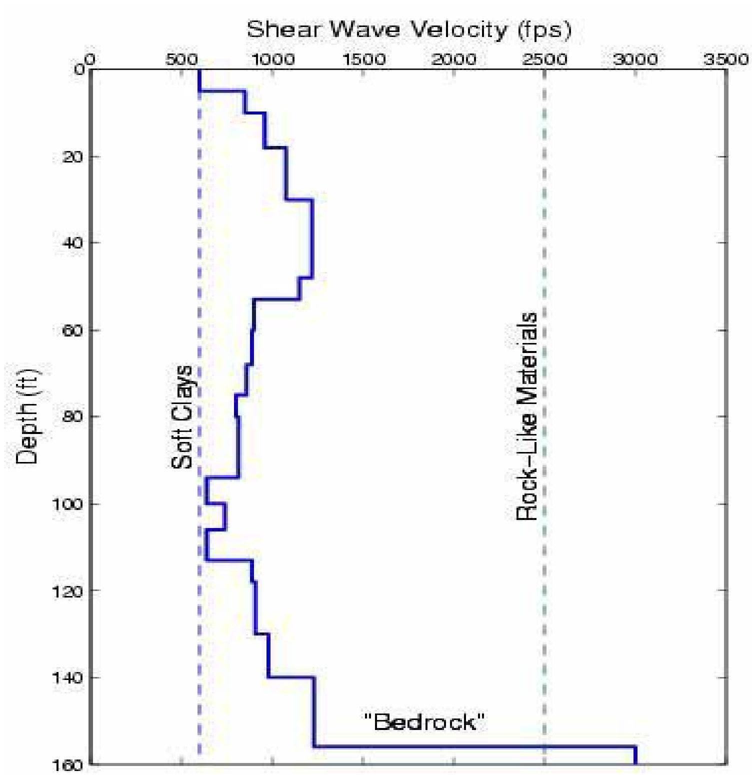

Figure 3. Shear wave velocity profile at the Atwood Building site.

BUILDING ARRAY CONFIGURATION

In order to obtain adequate building response measurements, the Atwood Building has been instrumented (Figure 4) during the summer of 2003 in cooperation with ANSS[8]. There are 32 channels of accelerometers (episensors of Kinemetrics, Inc.) installed on 10 levels of the building to monitor seismic responses of the building, including torsional effects and rocking behavior. The data recorders are housed on the 18th floor. A telephone line and internet jack is installed in the same room to enable the remote access of the data. Cable routing are selected at the most efficient way. A free-field station associated with this building is to be installed at the downhole array site. The instrumentation system in the building was tested in November 2003 and has been operating since then.

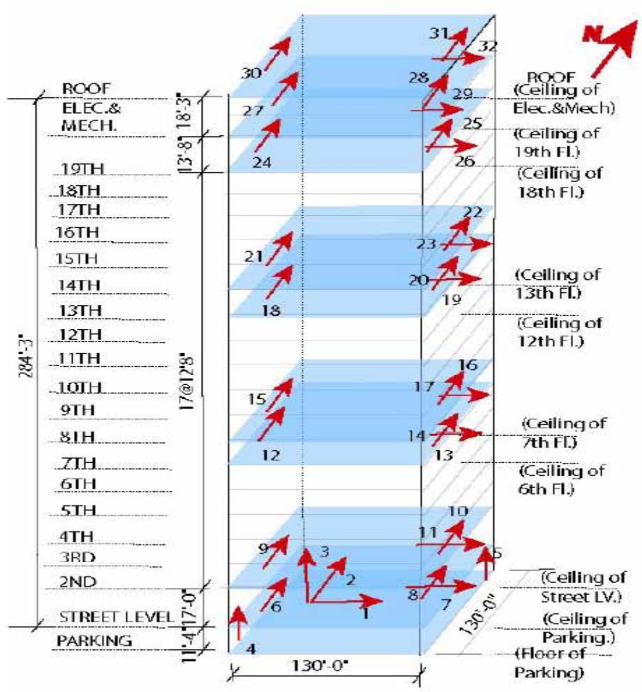

Figure 4. The seismic sensor configuration in the Atwood Building.

SEISMIC DATA ANALYSES

After the installation of the instruments (summer of 2003), building responses were recorded during a local earthquake (M_L=3.7) with an epicentral distance of 18.6 km (as mentioned earlier) on December 15, 2003. Figure 5 shows the time histories and Fourier amplitude spectra (FAS) of the input motions at the basement. The peak accelerations recorded in the basement are 6.35 cm/sec^2 (~0.64 % of g) for the E-W component, and 8.95 cm/sec^2 (~0.9 % of g) for the N-S component. From the FAS plots, it can be seen that the input energy is concentrated in the frequency range of 1~10 Hz. Although the shape of FAS plots is similar for E-W and N-S components, there are two peaks at 4.35 Hz (0.23 sec) and 7 Hz (0.14 sec) in the N-S component.

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[8] ANSS: Advance National Seismic System – an initiative to expand and upgrade the seismic network system in the United States, authorized by U.S. Congress and administered by the U.S. Geological Survey (USGS Circular 1188).

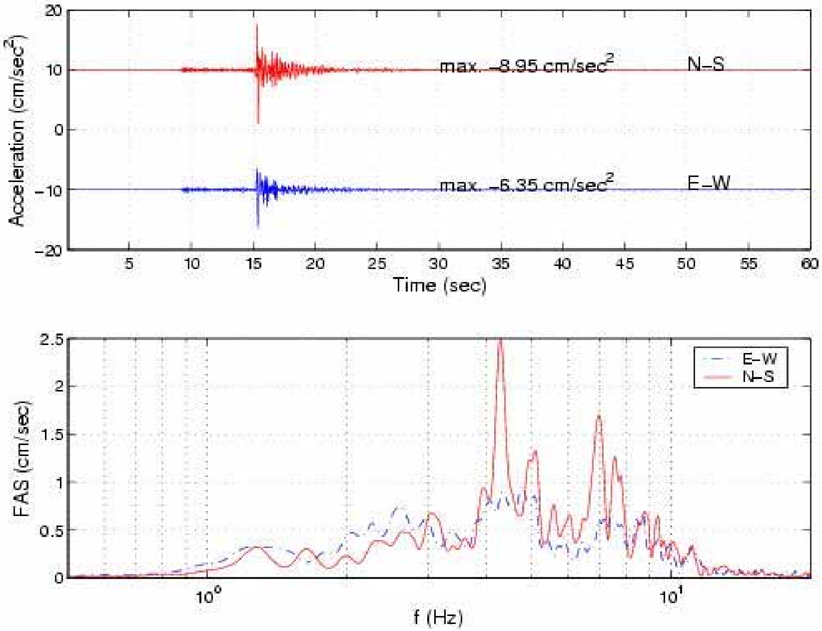

Figure 5. Acceleration time histories and FAS of the input motions for the December 15, 2003 earthquake.

The recorded acceleration and double integrated displacement responses at different levels of the building during the above earthquake are shown in Figures 6 and 7. Figure 6 shows the acceleration and displacement time histories in the E-W direction, while Figure 7 shows these responses in the N-S direction. Since the input motion is very weak, the building's torsional and rocking motions are not prominent for this earthquake, and therefore not considered in this study.

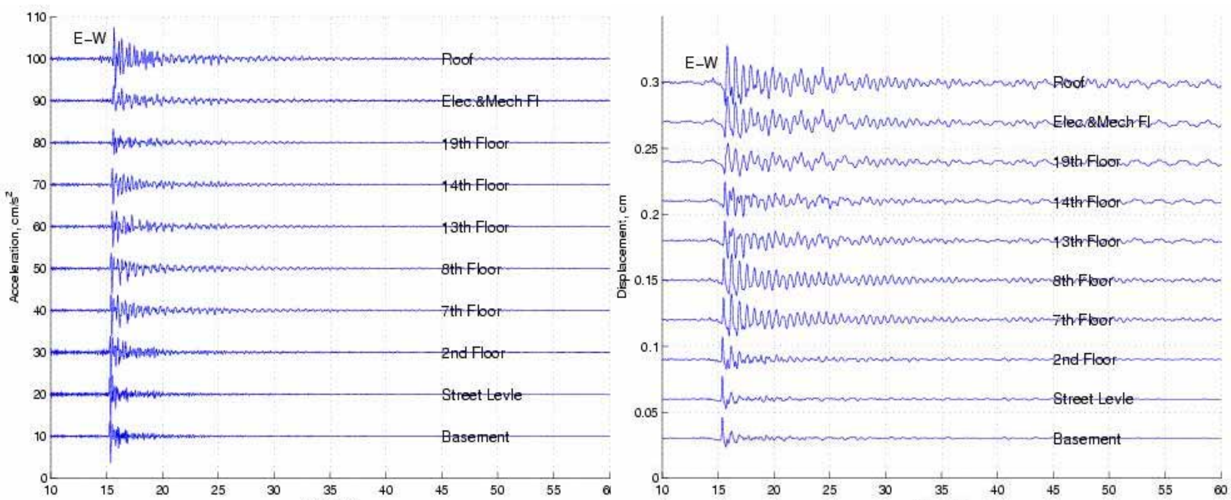

Figure 6. E-W components of acceleration and displacement time histories for the December 15, 2003 earthquake.

Based on the acceleration time histories, the dynamic properties of the building, such as fundamental periods, have been studied using spectral analysis. Figure 8 shows the Fourier spectra of the input motion at the basement, output motion on the roof and the spectral ratios between them for both E-W and N-S components. In order to get smoother spectra, the spectra are smoothed using a moving window with a width of 0.24 Hz. Note the peaks corresponding to the first four modes in each diection in these two figures; the periods and frequencies of these modes are summarized in Table 1. The signal-to-noise ratio was small for modes higher than the above modes. The fundamental period in the E-W direction is 2.21 sec (0.45 Hz), while that in the N-S direction is 1.86 sec (0.54 Hz). This indicates that the building is stiffer in the N-S direction than in the E-W direction. Although the structure is symmetric in geometry, it is associated with braced walls in the N-S diections in four bays which makes the building more stiffer in this direction.

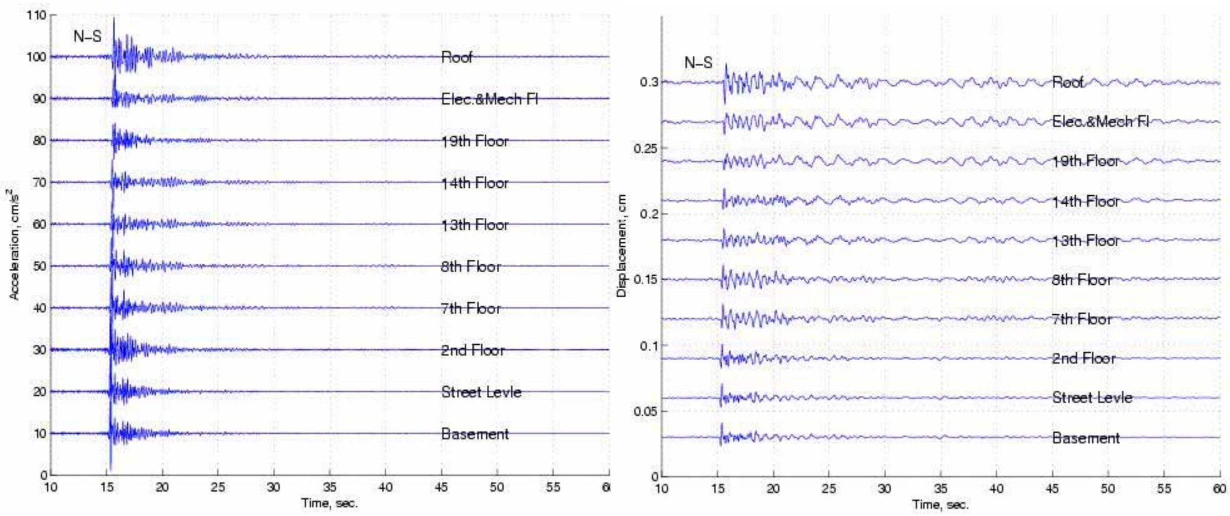

Figure 7. N-S components of acceleration and displacement time histories for the December 15, 2003 earthquake.

Since the input motion has small energy around 2 sec (0.5 Hz), one may argue that the peak around 0.5 Hz for E-W and N-S directions might be due to other factors. To ascertain these peaks, the spectral ratios from recordings at three more levels have been computed and shown in Figure 9. These plots clearly show the exsitence of peaks around 0.5 Hz for both directions and confirm the fundamental periods in both directions.

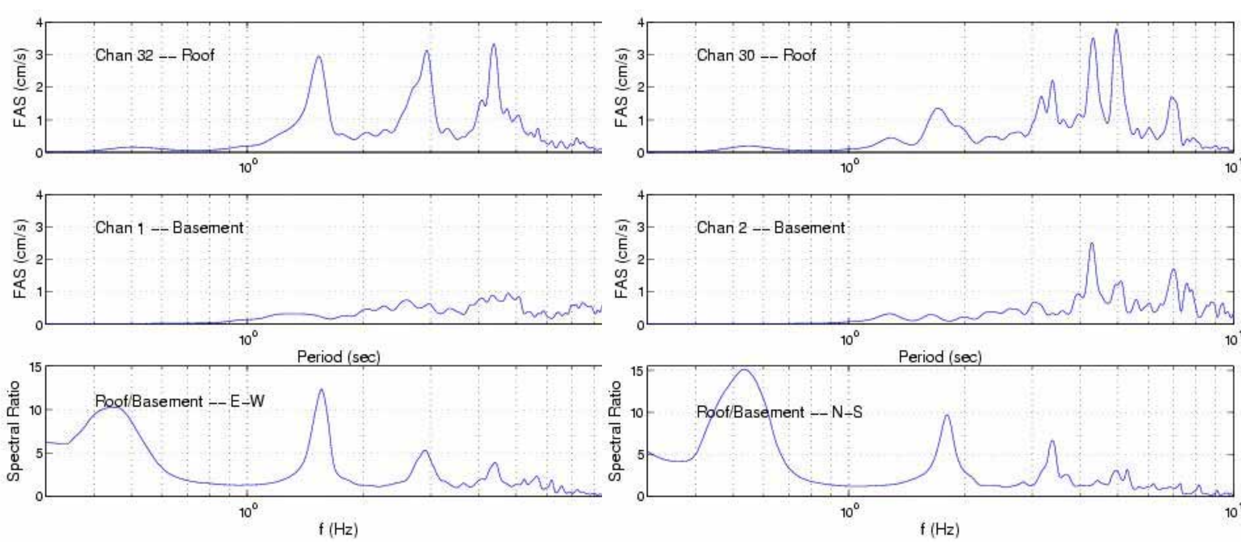

Figure 8. Fourier spectra and spectral ratios based on recordings from the December 15, 2003 earthquake.

Table 1 the first 4 modes based on the December 15, 2003 earthquake recordings.

Direction	Period - sec (f - Hz)			
	1^{st} Mode	2^{nd} Mode	3^{rd} Mode	4^{th} Mode
E-W	2.213 (0.452)	0.640 (1.563)	0.345 (2.900)	0.227 (4.405)
N-S	1.862 (0.537)	0.556 (1.800)	0.295 (3.390)	0.201 (4.975)

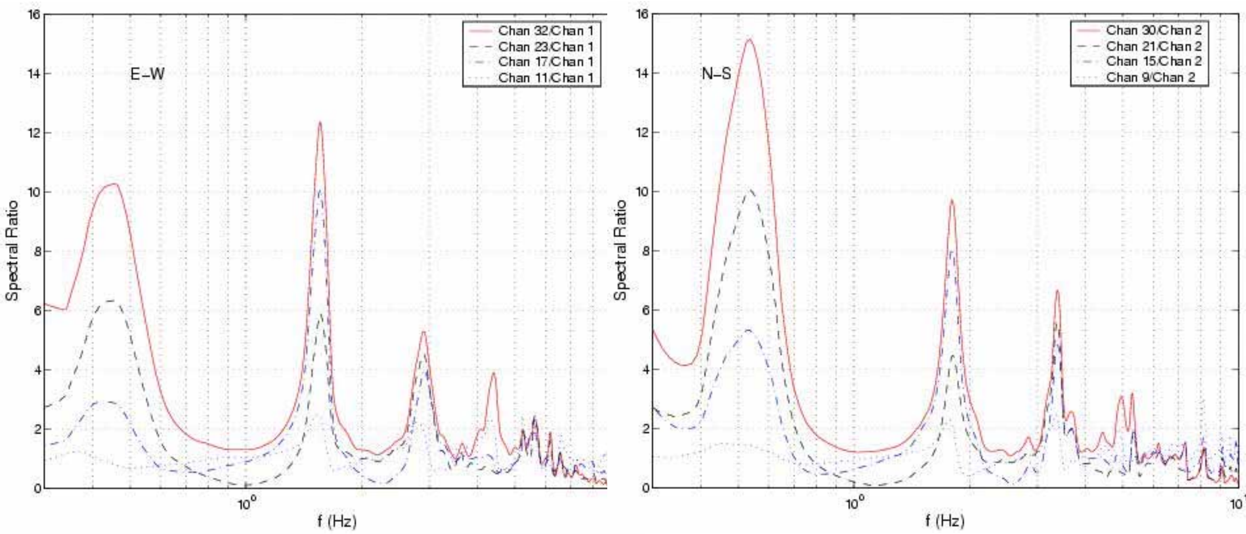

Figure 9. Spectral ratios computed at four different building levels using the December 15, 2003 earthquake data.

AMBIENT TEST AND DATA ANALYSIS

To further test the instrumentation system and study the dynamic properties of the building, an ambient vibration test was carried out on January 6, 2004. The wind on that day was in NE direction and reached 53 km/h. The ambient vibration was recorded for a period of 20 minutes at a rate of 200 samples/sec (Ventura et al. [4]). Figure 10 shows the E-W components of acceleration time histories recorded at the basement and roof levels, and Fourier spectra and spectral ratio in the ambient test. Only a portion of signals is shown. It can be seen that the input motion has a constant FAS value in the frequency range of interest and therefore has white noise characteristics.

In Figure 10, the first natural mode may be noted for the roof motion. Due to small signal-to-noise ratio, the other modes are not so prominent but can still be identified in FAS and spectral ratios. Similar results for the N-S component are shown in Figure 11. The periods and frequencies of the first four natural modes are summarized in Table 2. Comparing with Table 1, it is found that the natural periods of first modes identified in the earthquake records are slightly longer that that from the ambient vibration test. It is believed that this is caused by different levels of shaking in the two events.

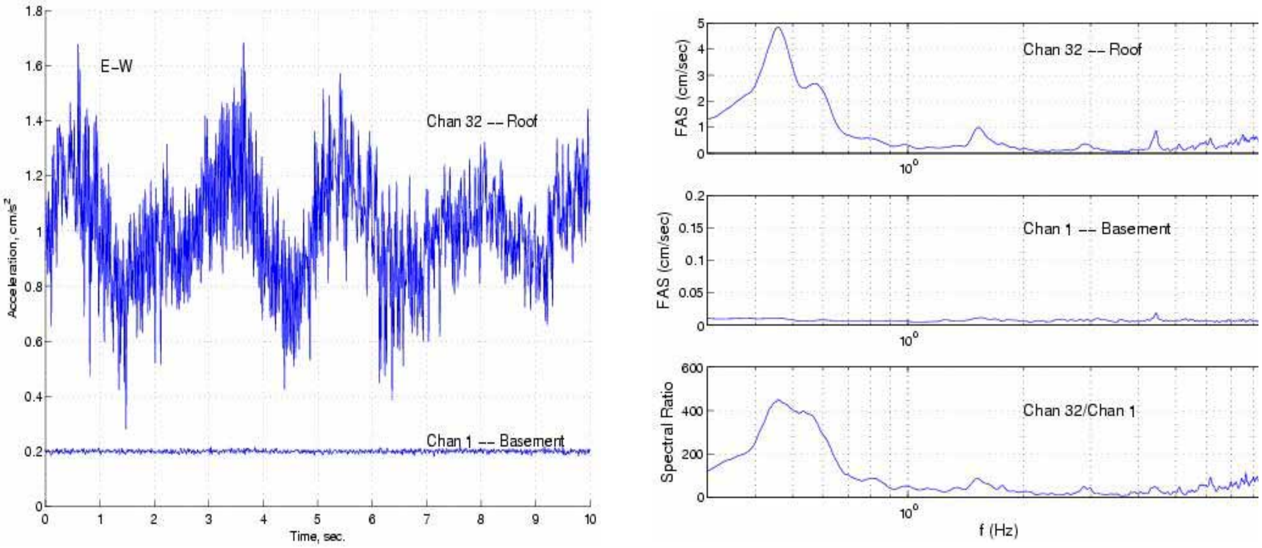

Figure 10. Portion of E-W ambient test signals and their FAS recorded in the basement and roof level of the Atwood Building.

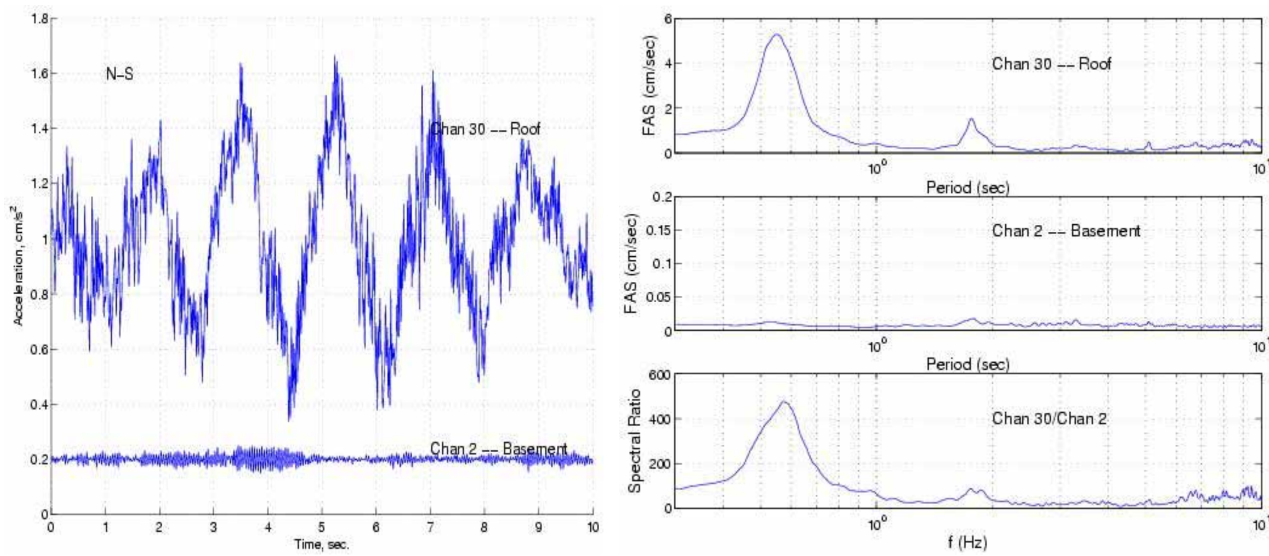

Figure 11. Portion of N-W ambient test signals and their FAS recorded in the basement and roof level of the Atwood Building.

Table 2 the first 4 natural modes based on the ambient vibration test recordings

Direction	Period -sec (f – Hz)			
	1st Mode	2nd Mode	3rd Mode	4th Mode
E-W	2.188 (0.457)	0.655 (1.527)	0.343 (2.915)	0.225 (4.444)
N-S	1.820 (0.549)	0.566 (1.767)	0.303 (3.300)	0.196 (5.102)

SITE CHARACTERISTICS

The characteristic site period is helpful in investigating SSI effects. A simple procedure (Celebi [5]) is utilized to identify the characteristic site period based on the records. The cross-spectra, calculated from pairs of orthogonal components of acceleration recorded at the basement, street level and roof are presented in Figure 12. The aforementioned periods of the first four modes can be noted in the roof cross-spectrum. The small peak at 1.282 Hz (0.780 sec) in the amplitude spectra (Figure 6) as well as in the basement cross-spectra (Figure 12) is the site frequency. It is noted that this frequency cancels out in the spectral ratio plots (Figure 9). To assess the reliability of the characteristic site period obtained in this study, the approximate site formula ($T_s = 4H/V_s$) has been used to calculate characteristic site period based on the shear wave velocity profile shown in Figure 3. This estimated site period is 0.65 sec (1.538 Hz). To further verify the identified site period, the transfer function of the site based on the same shear wave velocity profile has been computed using SHAKE91 (Idriss [6]) and presented in Figure 13. There is a good agreement between the site frequency 1.205~1.600 Hz found by using SHAKE91 and the site frequency 1.282 Hz obtained from the building records.

SUMMARY

This paper presents the strong-motion instrumentation of a highrise office building in downtown Anchorage. Adequate seismic coverage (32 channels on 10 levels of the building) has been successfully installed to monitor the seismic response of the building. Analyses of seismic data from a local small earthquake has been carried out to study the dynamic properties of the building. Ambient vibration testing have also been conducted using this instrumentation system. Results regarding the dynamic properties of the building from earthquake data and ambient vibration testing are consistent and having a difference of approximately 5%. This difference may be larger during strong shaking due to nonlinearities including SSI. It is anticipated that this building instrumentation will produce a valuable database for earthquake engineering studies, especially for SSI effects, when the nearby downhole array becomes operational.

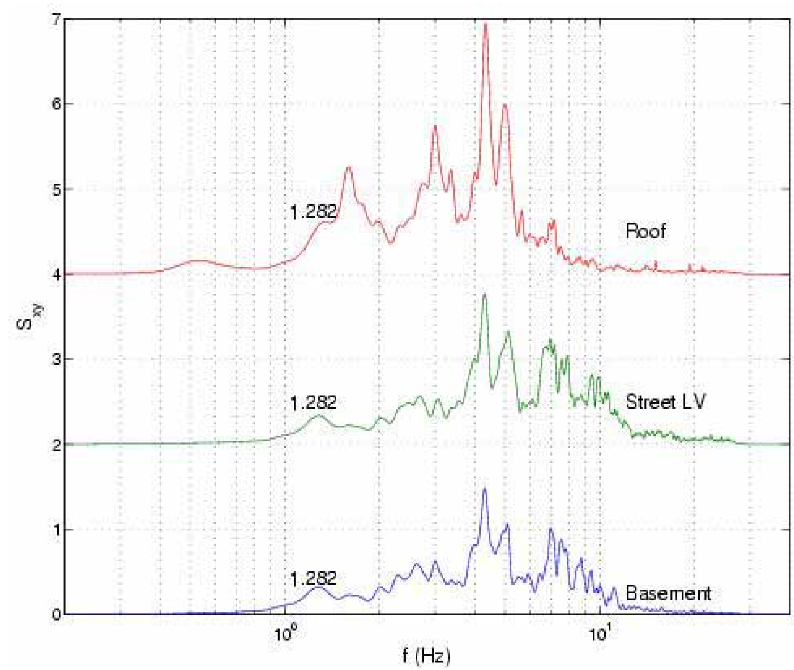

Figure 12. Cross-Spectra of motions at the roof, street level and basement of the Atwood Building.

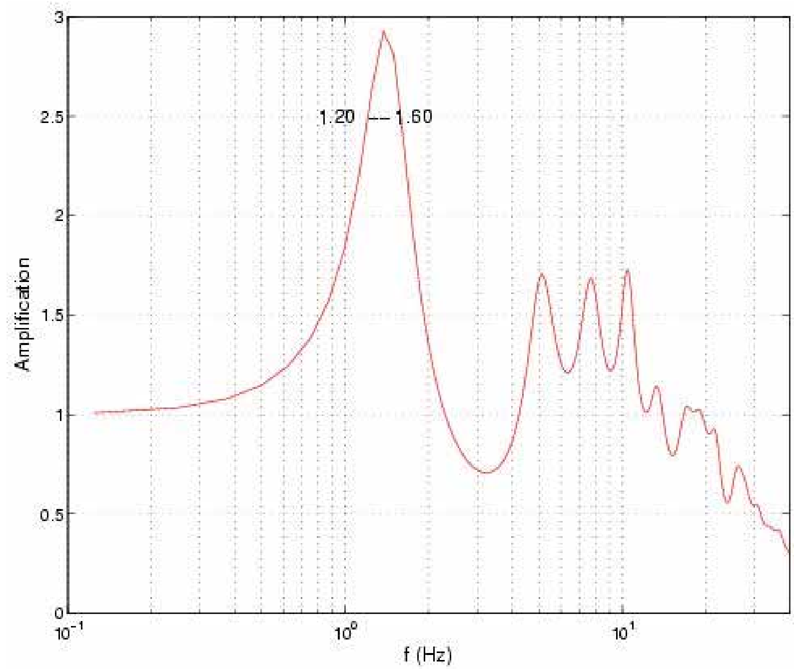

Figure 13. Transfer function at the building site.

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