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KENNETH K. ELTSCHLAGER
MINING/EXPLOSIVES ENGINEER
USDOI-OSM, THREE PARKWAY CENTER, PITTSBURGH, PA 15220
412.937.2169 / 412.937.3012 KELTSCHL@OSMRE.gov

Assessment of Low-Frequency Blast Vibrations and Potential Impacts on Structures.

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U.S. Department of the Interior, Bureau of Mines,
Twin Cities Research Center
Willard E. Pierce, Steven V. Crum and David E. Siskind

UNITED STATES DEPARTMENT OF THE INTERIOR

BUREAU OF MINES

ASSESSMENT OF LOW-FREQUENCY BLAST VIBRATIONS AND POTENTIAL IMPACTS ON STRUCTURES

BY

Willard E. Pierce[1]
Steven V. Crum[2]
David E. Siskind[3]

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[1]Physical Scientist, U.S. Bureau of Mines, Twin Cities Research Center

[2]Geophysicist, U.S. Bureau of Mines, Twin Cities Research Center

[3]Supervisory Geophysicist, Blasting Technology, U.S. Bureau of Mines, Twin Cities Research Center

CONTENTS

INTRODUCTION

The Bureau of Mines was contracted by the Office of Surface Mining (OSM) to study the existence of low-frequency, long duration ground vibrations from surface coal mine blasting at a selected sites and the effect that these vibrations have on residential structures.

RI 8507 (Siskind et al., 1980) established safe ground vibration levels for residential type structures. The report concluded that peak particle velocity is the best single ground motion descriptor and recommended a safe ground vibration limit for homes of 0.5 in/s for plaster on lath interiors and 0.75 in/s for drywall interiors at frequencies below about 15 Hz or at the resonant frequency. Peak particle velocities below these limits have effectively zero probability of even threshold damage regardless of the state of repair of the structure(Siskind, 1994). Appendix B of RI 8507 was included as an alternative safe blasting level criteria which recommended a particle velocity dependent upon frequency. These recommended safe levels were based upon the Bureau's own measurements and nine previous studies by others.

RI 8896 (Stagg et al., 1984) monitored the long term affects of repeated blasting on a wood frame house along with the environmental effects on crack production. RI 8896 concluded that the rate of threshold cracking when ground vibrations were <0.5 in/s was not significantly different than when motions were between 0.5 and 1.0 in/s. RI 8896 also concluded that the smallest ground vibration that would produce the equivalent of environmental strains in walls was 1.2 in/s.

The RI 8507 safe blast vibration recommendations were based upon structure response and crack inspections from vibrations with frequencies at or above 6 Hz. Dominant frequencies below 6 Hz were not found by the Bureau nor documented by other available sources studying surface mine blasting. The criteria in Appendix B for below 6 Hz was a maximum displacement criteria based upon earthquake-related damage (Crum and Pierce, 1995).

More recently, blast vibrations with frequencies below 6 Hz have been documented in various reports(Siskind et al., 1987, 1989 and 1993). These types of vibrations have caused local concern by citizens and in some cases complaints to the agencies implementing OSM regulatory programs. Siskind et al.(1993) and Crum and Siskind(1993) have monitored structure response from low-frequency blast vibrations. These reports monitored vibrations with amplitudes <0.1 in/s.

Based upon the citizens concern and the lack of structural response data to blast induced ground vibrations of higher amplitude and with frequencies below 6 Hz, questions have been raised concerning the adequacy of current OSM regulatory limits to provide protection to residential structures.

This research entitled "Assessment of low-Frequency Blast Vibration and Potential Impacts to Structures" was funded by OSM through interagency agreement EF68-IA 92-12180. The OSM technical project officer is Ken Eltschlager.

BACKGROUND

Based upon past research the Bureau of Mines has targeted the need for further investigation into the effects of low-frequency blast vibrations to residential structures. Concurrent with this contracted study was the Bureau in-house project entitled "Structure Response and Damage from Low-Frequency Coal Mine Overburden Blasting."

In early 1993 the Bureau contacted various state and federal agencies to inquire about the existence of blasting activities which were producing low-frequency, long duration blast vibrations. For the purposes of this study, low frequency vibrations are those below the natural frequency of residential structures or below approximately 6 Hz. Long duration vibrations are those lasting more than 3 to 5 seconds. The criteria for potential site selection also included vibration amplitudes at or above 0.5 in/s at a residential structure with suitable access for inspection and monitoring.

As part of the joint Bureau in-house and OSM effort, a list of 21 mines in 10 states was compiled from various sources. Over the course of several months Bureau researchers initiated contact with these mines and began making a short list of potential sites to visit in order to determine suitability to study. Eight mines in five states were then visited in order to evaluate the mine, the available structures and the blast vibration characteristics to determine suitability to this study.

Eventually 11 structures at six sites in four different states were monitored for blast induced structure response. This report will describe ground vibrations and structure response at one structure as called for in the contract with OSM. A more comprehensive report, to include data from all structures studied, will be completed in the near future.

SITE VISITS AND SELECTION

For this study, eight mines in five states were selected as potential sites. Eventually all eight were visited in order to determine suitability to the needs of the study.

Bureau personnel visited the potential sites and spent one to two days meeting with mine personnel and gathering relevant information. Blast records and blast design criteria were evaluated to get an idea of how the mine blasted. A review of the available seismograph records was made to determine the amplitude and frequency characteristics of the blast vibrations being generated at the mine. During each visit a blast was monitored to obtain a current seismograph record, usually at the structure of interest.

The site selection process contained many variables, including the following:

- Vibration characteristics at the structure to be monitored.
- The condition of the structure available for study.
- Access to the structure for instrumentation and pre- and post-blast inspections for damage.
- Cooperation from the mine, including:
    - Coordination of blasting schedule and times.
    - Availability of blast records and blast designs.
    - Distance from blast to the structure being monitored.
    - Maps of the blast area and the surrounding area.

After a review of the available information and consultation with the OSM project officer, the AMAX Coal Company's Penndiana Mine was selected as the site to be used in this study.

SITE DESCRIPTION

The AMAX Coal Company Penndiana Mine is a surface coal mine located in west-central Indiana just southwest of Dugger, Indiana. The Bureau monitored blasts at this mine from August 1993 to September 1994. During this time the mine was in the process of extending their pit from approximately 5,000 to 8,000 feet in length. The pit was orientated in an east-west direction and was progressing to the south.

The Bureau monitored ground vibrations, airblast and structure response at four houses all owned by the mine at the Penndiana site and therefore vibrations at the structures were not restricted by regulatory limits. All four houses were south of the pit and were in the direct path of the advancing mine. Although four houses were monitored, ground vibrations, frequencies and structure response characteristics were similar. Because it was closest to blasting, the "Shack" was chosen as the primary study house.

This mine was the site of a previous Bureau study, RI 9523 (Siskind et al., 1994), which investigated the response of buried pipelines to surface mine blasting.

STRUCTURE DESCRIPTION

The Shack, a name arbitrarily assigned to the house, was owned by AMAX Coal Company and was unoccupied at the time of the study. The windows and doors were removed, probably by vandals, once it became obvious that the house would not be occupied due to the advance of the pit.

The Shack was a small single story, wood frame Bungalow style house, 50 - 60 years old built on a cement block foundation without a basement. The interior of the house was finished with plaster on lath, although as is the custom in the area, many interior walls were covered with paneling to conceal the many existing cracks in the plaster. For purposes of this study the paneling was removed from the walls to facilitate the pre- and post-blast inspection of the plaster. The Shack was 26.2 feet wide by 35.3 feet long and divided into six rooms (Figure 1).

ANALYSIS AND FINDINGS

Monitoring

The Bureau monitored a total of 35 blasts at the Shack. Ground vibration and airblast along with structure response was measured for each recorded blast. Table 1 summarizes the recorded ground vibration and structure response at the Shack. Along with the monitoring program, a total of seven cracks, at various locations on interior walls, were inspected with a 7X optical comparitor before and after each blast to note any changes. Four of the cracks were inspected for possible extensions and three of the cracks were inspected for possible width changes.

Monitoring of vibrations at the Shack was accomplished with White Industrial Seismology, Inc. seismographs. Bureau-owned Mini-Seis and Seismite seismographs were utilized in this study. The Mini-Seis units were modified with an external transducer containing accerlerameters instead of velocity gauges. This allowed the transducer to be orientated flat against the wall when measuring structure response. A minimum of three seismographs were used for each blast.

The ground vibration transducer was installed in the ground adjacent to the house using the standard shallow burial method. The airblast microphone was located approximately 3 feet off the ground near the ground vibration transducer. The structure was monitored by attaching an aluminum plate to the wall to which a seismograph transducer was attached. The structure response transducers were installed in the same corner as the ground vibration was monitored. One transducer was mounted at floor level and the other was installed at ceiling level.

The corner of the house closest to the blast was monitored for each blast. For some blasts transducers were placed in other corners or on other walls to compare characteristics of the structures response at various locations within the house. From this, it was determined that the corner closest to the blast gave worse-case structure response and was therefore used as the primary monitoring location. For the Shack this was the NE corner, which remained consistent throughout the monitoring effort.

Although the seismographs were self triggering it was important to have the records from different locations to be time-synchronized to determine relative excitation response motions. The seismograph manufacturer helped to accomplish this by including a manual trigger feature built into the seismograph. The seismographs were "daisy-chained" together via a cable. The ground vibration or "master" unit was set up using the automatic trigger function and was triggered when the ground vibration exceeded the preset trigger level. The structure response or "slave" units were set on manual trigger. When the "master" unit was triggered, it in turn would trigger the "slave" units thus giving a common time synchronization for all the instruments on the chain.

Waveform Analysis

Ground vibration, airblast and structure response were recorded and analyzed for each of the 35 blasts. The highest ground vibration was 6.0 in/s. The seismograph data for all blasts recorded at the Shack are listed in Table 1.

The transducers were aligned to correlate each direction of motion with the alignment of the structure. The radial direction is in the same direction as the length of the house(east-west) or perpendicular to the roof trusses. The transverse direction is in the same direction as the width of the house(north-south) or parallel to the roof trusses. The vertical component is the vertical direction.

Two sets of time-correlated waveforms are in Figures 2 and 3, representative of the types of vibrations recorded at this structure. The first set, Figure 2, illustrates a single frequency ground vibration of 6 Hz which closely matches the natural frequency of the structure. The second set, Figure 3, illustrates a more complex or multi-frequency ground vibration. In both cases the low-frequency portion of the ground vibration excitation induces a 2-3 times structural amplification.

<u>Frequency</u>

Previous research has implied a potential problem with low frequency ground vibration and the associated structural response. This is due to the higher absolute displacements at lower frequency ground vibration. The question then becomes whether or not the low-frequency ground vibrations also translate into higher differential displacements within a structure.

Low-frequency ground vibrations have been identified at surface mines using large borehole diameters where certain geologic conditions exist (Crum and Pierce, 1993). The natural frequency range of residential structures is 4 to 12 Hz and previous studies have had only a limited amount of structure response data from ground vibrations below 6 Hz.

The peak structural response of the Shack was at a range of frequencies from 3.5 to 9.4 Hz, with an average of 5.3 and 5.1 Hz for the radial and transverse directions respectively. The driving ground vibration had a range of frequencies from 2.7 to 22.2 Hz, with an average of 8.6 and 8.9 Hz for the radial and transverse directions, respectively. The peak response frequencies should correlate closely to the natural frequency of the structure.

Figure 4 illustrates the theoretical amplification of a single degree of freedom (SDOF) system being excited by continuous, sinusoidal, harmonic motion of constant amplitude with damping values of 5 and 10 percent (after Harris and Crede, 1961). This model shows that when excitation frequencies decrease below the natural frequency response will become the same as the excitation, therefore the differential motions will decrease to zero. Reduced differential motions would result in reduced strain and decreased potential for cracking (Crum and Pierce, 1993).

Figure 5 is a plot of measured amplification factors versus the ratio of the frequency of the ground vibration driving the peak structure response and the frequency of the peak structure response. The line for 10 percent damping from Figure 4 has been superimposed onto Figure 5. Due to the range of natural frequency associated with the Shack, the frequency of peak structure response for each event was used for this plot. Comparing Figures 4 and 5 reveals a similar but not close match between the theoretical model and the measured results. Keep in mind however, that the measured results, for most events, are the result of only 1 to 2 cycles of ground vibration excitation at a particular amplitude near the structures natural frequency. Due to the transient nature of blast vibrations, only 1 or 2 consecutive cycles with harmonic, sinusoidal motion near the natural frequency of the structure with significant amplitude is common.

Damping

Damping controls the rate of decay of vibratory oscillation and therefore is one of the most important structure response characteristics. Damping values calculated from free vibration motions are given by:

$$\beta = 100/2\Pi m \ (-\ln(A_n/A_{n+m})$$

Where β is the percent of critical damping, A is the peak amplitude at the n^{th} cycle and m is any number of cycles later (Siskind et al., 1980).

Response waveforms were analyzed to determine the damping values of the structure for both horizontal directions. The measured damping values measured from response to typical blasts ranged from 3.6 to 14.8 percent for both of the horizontal components. The average damping values for the radial and transverse components were approximately 10 and 8 percent respectively. These values are likely lower than true damping values in most cases because the measurements were made where obvious decay of structure response was present but total driving forces had not always ceased. At the point where damping measurements were made the driving force of the ground vibration had fallen off dramatically and was less than .03 in/s, however it was still present.

Referring to Figure 5 one could estimate the damping value of the Shack. Although there are 35 reported blasts in this data set, the estimate is only as good as the number and quality of the data points available. Nonetheless, the apparent damping value illustrated in Figure 5 is approximately 8.5 percent which is very close to the average damping values measured from individual records listed above.

These measured damping values are significant in that they are higher than the previously reported average residential structure damping of 5 percent(Dowding, 1985). Increasing the damping value from 5 to 10 percent changes the number of continuous, harmonic excitation cycles needed to reach maximum structural response significantly. The theoretical values of amplification versus number of vibration cycles, calculated by a finite-duration SDOF system at the natural frequency is plotted in Figure 6. Excitation is once again assumed to be continuous, sinusoidal motion of constant amplitude, but with a finite number of cycles.

At 5 percent damping, 6 continuous, harmonic excitation cycles at the natural frequency are needed to reach 80% of the peak response amplitude but at 10 percent damping only 3 excitation cycles are required. The potential amplification at the natural frequency is reduced from 10 times at 5 percent damping to 5 times at 10 percent damping. At 15 percent damping these values are decreased proportionately.

The higher damping value tends to broaden the response characteristics frequency range of the structure. Velocity transducers are highly damped so that they have a wider range of frequency response. In affect a structure with higher damping will respond naturally to a wider range of frequencies as compared to a structure with a lower damping value. This is illustrated in Figures 2 and 3 where similar structure response amplification is induced by different ground vibration frequencies.

Peak Particle Velocity Influences on Structural Response

The structure response reported in this paper is driven by blast induced ground vibration. Past reports have indicated amplification factors up to 4 with 1.5 being a typical value(Siskind et al., 1980). Referring to Figure 7 all high corner response amplification factors are plotted against the driving phase of the ground vibration. The amplification values were calculated by taking the peak structure response for each component of motion and dividing that value by what was determined to be the corresponding ground vibration driving the peak structure response.

The structure exhibited very little if any amplification of the ground vibration once the ground vibration exceeded 2.0 in/s. Figure 7 illustrates this phenomena, even though there are 11 of the 25 driving ground vibrations less than 12 Hz (Figure 8).

Reduced amplification of structure response with increasing ground vibration particle velocities was not found at all houses studied. However it can be noted that Stagg et al. in RI 8896 also reported reduced amplification with increased ground vibration particle velocities. Therefore this is probably not unique to the Shack but analysis of data at other structures must be evaluated prior to drawing broad conclusions to this observation.

Duration of Ground Vibration

Theoretically, structure response should increase with each continuous, harmonic, sinusoidal excitation cycle at the natural frequency as shown in Figure 6, up to a certain amplification, determined in part by the damping of the structure. Because of the transient nature of ground vibrations with exponentially decaying amplitudes, the peak structure response very often occurs with the first 1 or 2 cycles at or near the natural frequency of the structure.

However, when the ground vibration becomes sinusoidal as illustrated in Figures 3 and more so in Figure 2, peak structure response occurs at approximately 3 cycles just as the model in Figure 6 suggests. From this same Figure, 3 cycles to peak structure response also correlates to a damping value in the range of 10 percent.

Three ground vibration cycles at a frequency of 6 Hz occurs in 0.5 seconds. Using this example, sinusoidal waveforms lasting more than 0.5 seconds increase the amplitude of structure response only slightly. A structure with a damping value of 5 percent will reach the majority of its response potential within the first second of the ground vibration at the natural frequency of the structure. In theory, measurement of ground vibrations and structure response with longer durations and greater number of cycles than shown in Figure 6 will not produce larger amplifications.

Damage Inspections

Seven cracks at various interior locations on the walls were inspected immediately before and after each blast with a 7X magnification optical comparitor. This optical comparitor had a scale with 0.1 mm per division and a resolution of 0.05 mm (0.002 in). Three of the cracks were monitored for width changes and four of the cracks were monitored for extensions or a change in length. It was important that the same person doing the preblast inspection would also do the postblast inspection, to minimize the influence of subjective interpretation on the measurements.

At the Shack a total of 245 pre- and post-blast inspections of cracks were conducted. The number of blasts, cracks inspected and the level of ground vibration recorded at all the homes in the Bureau in-house study of structure response is reported in Table 2.

Only two observations of crack change were noted, both from a single blast. This first was near a window where two cracks opening from a lower and upper corner of the window and progressing diagonally to the floor and the ceiling respectively.

A post blast inspection revealed that the upper crack opened 0.05 mm and the lower crack closed 0.05 mm. The blast produced a recorded peak ground vibration of 1.28 in/s at 5.6 Hz in the transverse direction with a peak structure response of 2.24 in/s, which calculates to an amplification factor of 1.75.

This was the only crack change caused by blasting for either width or length even though blast induced ground vibrations up to 6.0 in/s were recorded. On several occasions when pre-blast inspections were conducted and then the blast was delayed for a couple of hours, prompting a new pre-blast measurement to be taken, changes in crack widths were noted. Crack width changes up to 0.10 mm were measured and were probably caused by normal daily temperature and humidity changes.

As part of the parallel Bureau in-house project, another structure, the Lhemkuhler home was also instrumented and inspected for damage. A blast induced crack was observed at this home adjacent to a window in a crack free area. The home is a plaster on lath interior and a crack formed after a blast of approximately 1.50 in/s. Although the crack was visible with a strong light on the area, the size was much smaller than the 0.05 mm resolution of the optical comparitor used for this study. This structure may be discussed in a future paper and is only mentioned here because it coincides with the ground vibration level at the Shack that caused a crack to change. Also, the changes listed above are at or above the smallest ground vibration that would produce the equivalent of environmental strains in walls (1.20 in/s) as reported by Stagg et al. in RI 8896.

Summary

The Bureau contacted various state and federal regulatory agencies to inquire about the existence of blasting activities which were producing low-frequency, long duration blast vibrations. Eventually 21 mines in 10 states were contacted and eight mines in five states were actually visited in order to determine the suitability to this study.

The difficulty in finding a suitable site for this study leads one to believe that the existence of sites having low-frequency, long duration blast vibrations at amplitudes approaching or exceeding the regulatory limits is, at best, rare. The Penndiana mine was determined to have the best match of characteristics to this study due to the multiple cycles of frequencies near 6 Hz at amplitudes that would exceed 1 in/s. However this was at a structure controlled by the mine and therefore had no regulatory vibration limits.

Although there is a lack of actual measurements below about 50 percent of the natural frequency of the structure studied, there is an abundance of data at the natural frequency of the structure. Excitation at the natural frequency of a structure is the worse-case scenario for highest cracking potential. Theoretical models along with data in this report and others support the premise that maximum response is generated when the excitation matches the natural frequency of the structure. When excitation frequencies decrease below the natural frequency the response will be the same as the excitation and strain inducing differential motions will decrease to zero.

Based on observation, the duration of the ground vibration does not affect the maximum structural amplification beyond the first 1 to 3 cycles of excitation at the natural frequency. This is due to the transient nature of the ground vibration with decaying amplitudes and complex frequency content. This is consistent with calculations using a finite duration SDOF model assuming continuous, harmonic, sinusoidal excitation at damping values, determined for this study, which are 50 to 100 percent greater than those commonly used and determined from previous studies. At worse-case, 3 to 6 cycles of continuous, harmonic and sinusoidal excitation at the natural frequency of structures would be at most 1 second (assuming 6 cycles at 6 Hz) and therefore longer durations would not create higher structural response amplitudes.

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Blast Number	Channel	Peak Gnd. Vibration (in/s)	Peak Gnd. Frequency (Hz)	Peak Struc. Response (in/s)	Peak Struc. Frequency (Hz)	Driving Gnd. Vibration (in/s)	Driving Gnd. Frequency (Hz)	Amplification Factor
1	R	0.21	13.8	0.41	8.2	0.13	9.5	3.15
1	V	0.09	11.9	0.17	13.4	0.08	10.7	2.13
1	T	0.14	8.2	0.22	9.4	0.14	10.0	1.57
2	R	0.62	5.4	2.00	5.6	0.31	6.0	6.45
2	V	0.15	28.4	0.28	15.5	0.04	9.3	1.87
2	T	0.65	5.8	2.52	5.5	0.52	6.0	4.85
3	R	0.07	6.1	0.14	6.0	0.05	5.7	2.80
3	V	0.04	6.0	0.05	6.6	0.04	6.0	1.25
3	T	0.14	5.3	0.26	5.6	0.04	4.2	6.50
4	R	1.14	6.3	3.12	5.8	0.94	6.2	3.32
4	V	0.45	6.7	0.64	9.8	0.45	6.7	1.42
4	T	1.48	7.7	2.80	5.6	1.15	6.4	2.43
5	R	0.62	6.7	1.74	6.1	0.42	6.7	4.14
5	V	0.25	9.3	0.34	10.2	0.24	10.4	1.42
5	T	0.57	7.5	1.08	6.2	0.42	7.4	2.57
6	R	0.93	9.1	1.72	5.6	0.61	7.4	2.82
6	V	0.34	8.9	0.36	5.9	0.34	8.9	1.06
6	T	0.86	8.3	1.00	5.8	0.45	6.9	2.22
7	R	1.09	5.9	2.40	5.9	0.76	5.3	3.16
7	V	0.51	30.1	0.62	14.2	0.51	30.1	1.22
7	T	0.95	7.7	2.00	5.3	0.95	7.6	2.11
8	R	0.82	22.2	1.20	5.7	0.53	9.8	2.26
8	V	0.56	36.5	0.67	0.3	0.56	36.5	1.20
8	T	0.71	24.3	0.85	6.2	0.39	18.2	2.18
11	R	0.65	6.6	2.08	5.3	0.55	8.5	3.78
11	V	0.36	7.4	0.60	7.1	0.36	7.4	1.67
11	T	1.28	5.6	2.24	5.0	1.28	5.6	1.75
12	R	0.07	13.8	0.21	7.1	0.04	9.8	5.25
12	V	0.06	24.3	0.07	24.3	0.06	24.3	1.17
12	T	0.12	11.3	0.32	6.3	0.08	6.2	4.00
13	R	0.67	5.3	1.92	4.9	0.53	5.1	3.62
13	V	0.28	6.1	0.44	7.5	0.19	8.5	2.32
13	T	0.77	7.2	1.80	5.3	0.61	5.8	2.95
14	R	0.96	5.9	2.76	5.5	0.69	5.9	4.00
14	V	0.33	12.8	0.48	10.8	0.33	12.8	1.45
14	T	1.04	8.0	1.84	5.2	0.86	6.9	2.14
15	R	0.83	6.0	1.88	5.4	0.56	5.6	3.36
15	V	0.51	5.1	0.54	7.6	0.51	5.1	1.06
15	T	0.84	7.0	1.44	5.4	0.62	5.8	2.32
16	R	1.48	9.8	1.78	5.5	0.90	2.7	1.98
16	V	0.88	23.2	0.74	23.2	0.84	17.4	0.88
16	T	1.44	15.5	1.18	5.3	0.86	16.2	1.37
17	R	2.00	18.2	2.00	6.0	1.28	10.7	1.56
17	V	1.32	30.1	0.98	10.0	0.88	9.0	1.11
17	T	2.52	16.0	0.94	6.7	0.93	13.4	1.01
18	R	2.32	16.0	2.40	5.3	1.00	17.5	2.40
18	V	1.88	32.0	1.48	18.9	1.56	24.5	0.95
18	T	2.08	15.0	1.10	5.0	0.81	13.5	1.36
19	R	1.88	18.9	1.86	6.0	1.09	8.3	1.71
19	V	2.16	28.4	1.32	36.5	2.12	30.1	0.62

Table 1. Recorded ground vibration and structure response at the Shack.

Blast Number	Channel	Peak Gnd. Vibration (in/s)	Peak Gnd. Frequency (Hz)	Peak Struc. Response (in/s)	Peak Struc. Frequency (Hz)	Driving Gnd. Vibration (in/s)	Driving Gnd. Frequency (Hz)	Amplification Factor
19	T	2.48	16.0	1.24	6.0	0.47	7.0	2.64
20	R	2.08	17.6	1.04	4.8	1.24	10.0	0.84
20	V	1.80	23.2	1.56	30.1	1.10	32.7	1.42
20	T	3.72	8.6	2.14	4.9	1.24	19.7	1.73
21	R	2.60	9.3	2.10	5.0	2.24	8.5	0.94
21	V	1.64	30.1	2.00	11.9	0.56	25.6	3.57
21	T	2.36	20.4	1.62	4.9	2.36	4.9	0.69
22	R	2.36	9.4	1.74	4.8	2.36	9.4	0.74
22	V	2.08	32.0	1.84	30.1	1.44	34.2	1.28
22	T	2.96	11.9	1.22	6.7	2.96	11.9	0.41
23	R	0.45	12.8	0.58	6.4	0.11	6.7	5.27
23	V	0.20	12.1	0.25	12.8	0.20	12.1	1.25
23	T	0.33	10.0	0.58	5.5	0.24	6.6	2.42
24	R	0.25	4.8	0.88	5.6	0.14	5.7	6.29
24	V	0.11	9.1	0.14	7.0	0.07	17.6	2.00
24	T	0.32	5.2	0.90	5.2	0.31	6.2	2.90
25	R	0.59	5.6	1.88	6.0	0.31	7.2	6.06
25	V	0.28	7.5	0.48	7.2	0.22	18.3	2.18
25	T	0.82	7.3	2.44	4.7	0.59	5.6	4.14
26	R	0.64	5.6	1.11	4.7	0.51	5.2	2.18
26	V	0.34	9.4	0.37	15.5	0.16	9.5	2.31
26	T	1.12	5.6	3.64	4.7	1.02	5.5	3.57
27	R	1.03	8.9	1.96	5.5	1.03	8.9	1.90
27	V	0.52	10.6	0.56	9.3	0.13	14.6	4.31
27	T	1.28	5.4	3.40	4.6	1.24	4.9	2.74
28	R	0.82	5.1	2.00	5.5	0.41	5.5	4.88
28	V	0.31	5.9	0.56	12.1	0.18	20.5	3.11
28	T	0.83	6.4	1.56	4.7	0.25	6.7	6.24
29	R	3.12	7.0	2.24	6.0	2.88	10.5	0.78
29	V	2.80	8.6	3.28	10.2	1.76	46.7	1.86
29	T	4.16	21.3	1.86	4.7	1.28	10.9	1.45
30	R	6.00	13.8	4.08	4.5	6.00	13.8	0.68
30	V	4.16	24.3	3.44	15.0	4.16	24.3	0.83
30	T	3.92	11.1	2.24	4.4	3.92	11.1	0.57
31	R	4.56	10.6	3.44	4.0	4.56	10.6	0.75
31	V	3.04	17.0	2.96	13.4	2.48	41.0	1.19
31	T	3.36	6.9	2.48	3.5	2.96	11.5	0.84
32	R	3.92	16.0	2.80	6.5	3.92	16.0	0.71
32	V	4.40	30.1	5.12	20.4	3.04	19.7	1.68
32	T	4.64	13.8	3.28	5.7	2.46	10.7	1.33
33	R	3.92	10.0	2.24	5.2	2.00	21.8	1.12
33	V	1.06	16.5	1.16	11.9	0.70	48.8	1.66
33	T	3.44	10.4	1.78	3.5	1.70	22.2	1.05
34	R	4.40	13.1	2.32	4.8	3.52	18.6	0.66
34	V	1.88	17.0	2.00	9.3	1.52	22.7	1.32
34	T	5.76	11.3	2.32	4.0	3.20	16.0	0.73
35	R	3.76	10.4	2.14	7.3	3.76	10.4	0.57
35	V	4.16	24.3	4.00	11.1	3.36	34.1	1.19
35	T	5.36	10.0	2.12	6.8	5.36	10.0	0.40

Table 1. Recorded ground vibration and structure response at the Shack.

Table 2. Tabulation of Blasts and Crack Inspections at Recently Monitored Homes

House I. D.	Number of Monitored Blasts	Number of Blasts with Pre- and Post-Blast Crack Inspecitons (Number of Inspection Sites in Home)	Number of Blasts within Range of Peak Particle Velocities (Number of Blasts with Crack Inspections)			
			<0.5 in/s	0.5-0.99 in/s	1.0-4.00 in/s	>5.00 in/s
Lhemkuhler	106	7 (10)	33	36 (4)	36 (3)	1 (0)
Shack	35	35 (7)	8 (8)	8 (8)	15 (15)	4 (4)
Smith	20	0	8	10	2	0
Arvida	11	0	11	0	0	0
Jordan	4	4 (4)	4 (4)	0	0	0
McConnell	4	4 (5)	4 (4)	0	0	0
Hoover	5	0	5	0	0	0
Manor	5	0	3	2	0	0
Hole	9	0	1	1	7	0
Pritcher	2	0	1	1	0	0

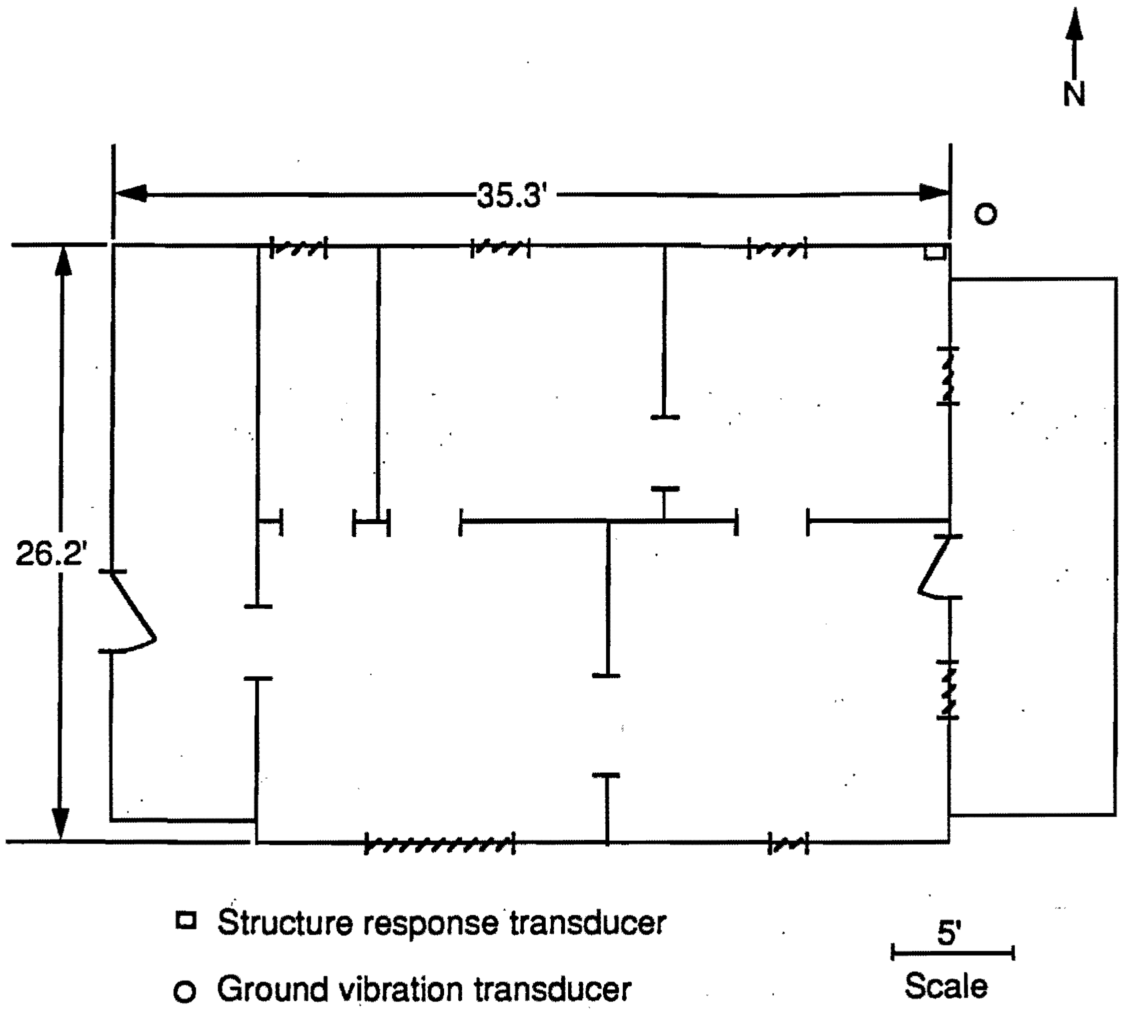

Figure 1. Floor plan of the 'Shack'.

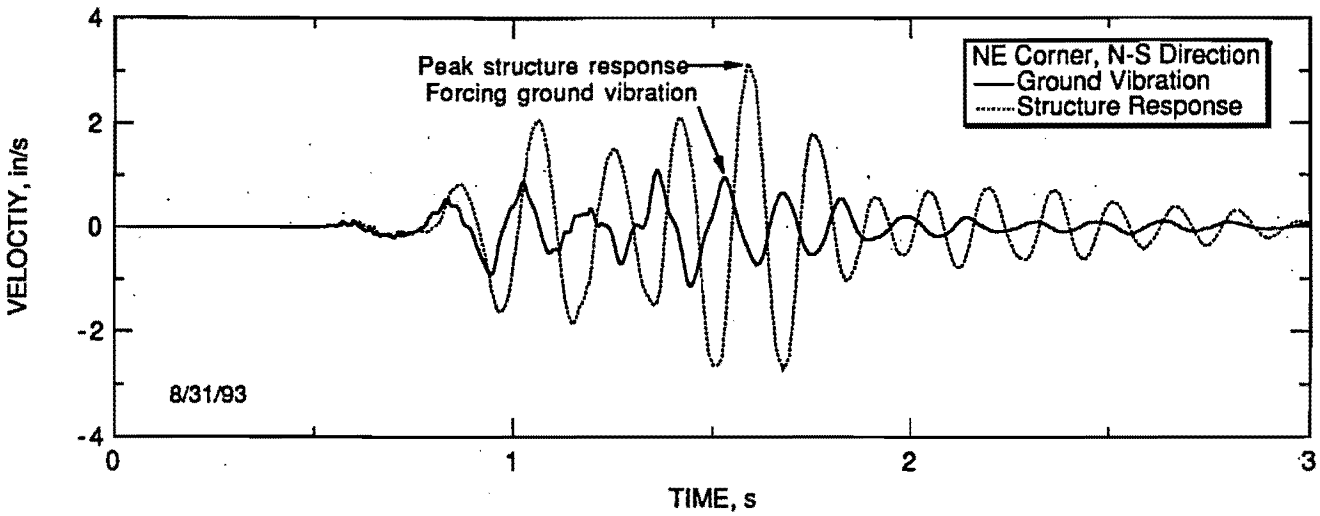

Figure 2. Ground vibration and resulting structure response at the Shack (transverse direction).

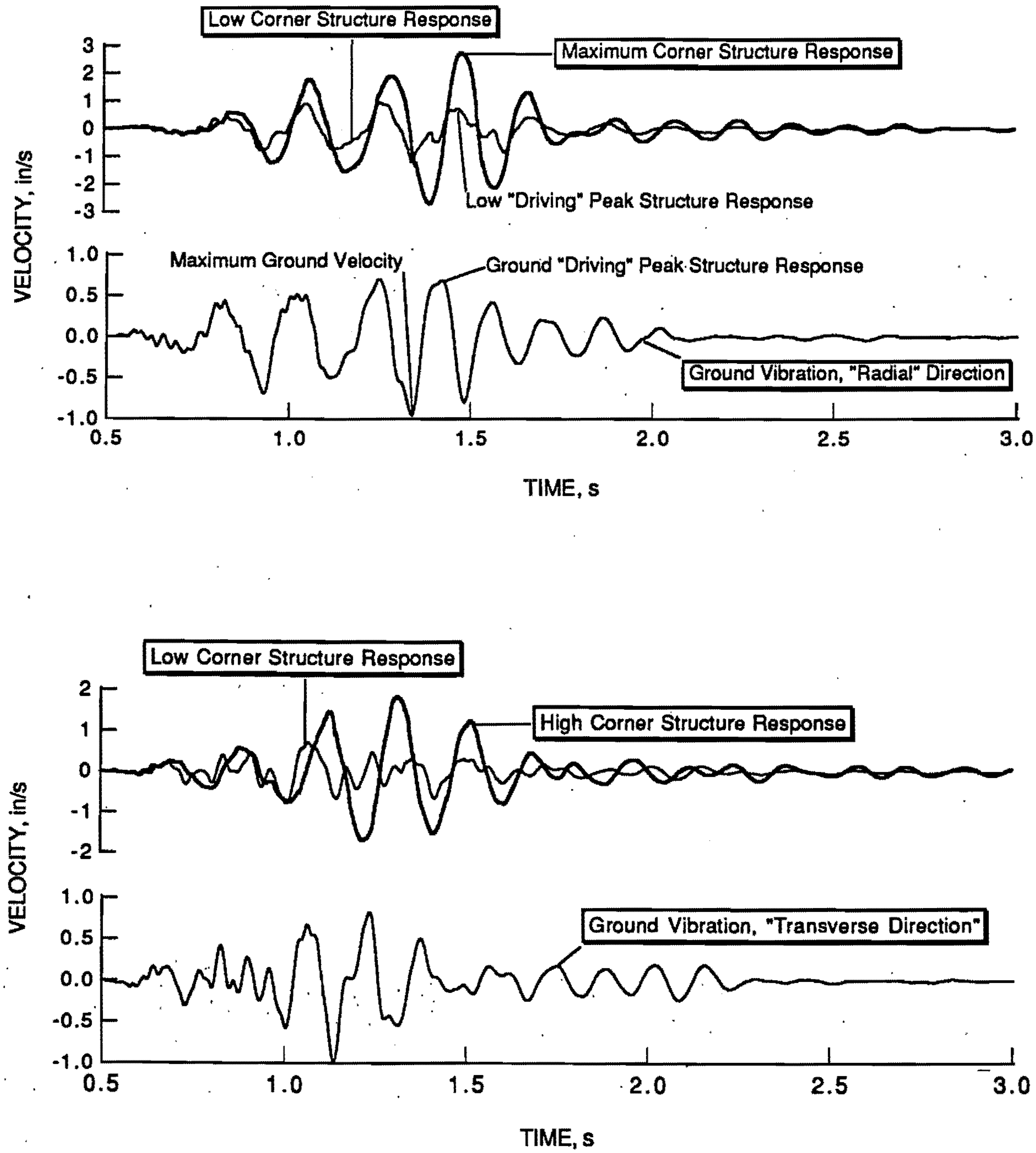

Figure 3. Ground vibration and resulting high and low corner structure response at the Shack along the radial (top) and transverse (bottom) directions (see text for definition of radial and transverse directions).

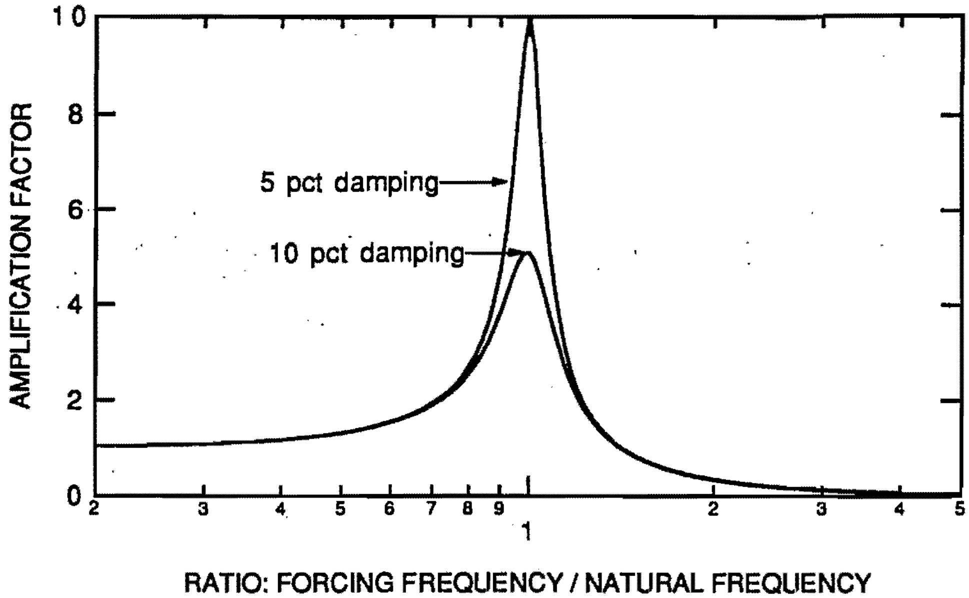

Figure 4. Theoretical response of a SDOF system with damping of 5- and 10-pct of critical (after Harris and Crede, 1961). Excitation is assumed to be infinitely long, continuous, sinusoidal and of constant amplitude and frequency.

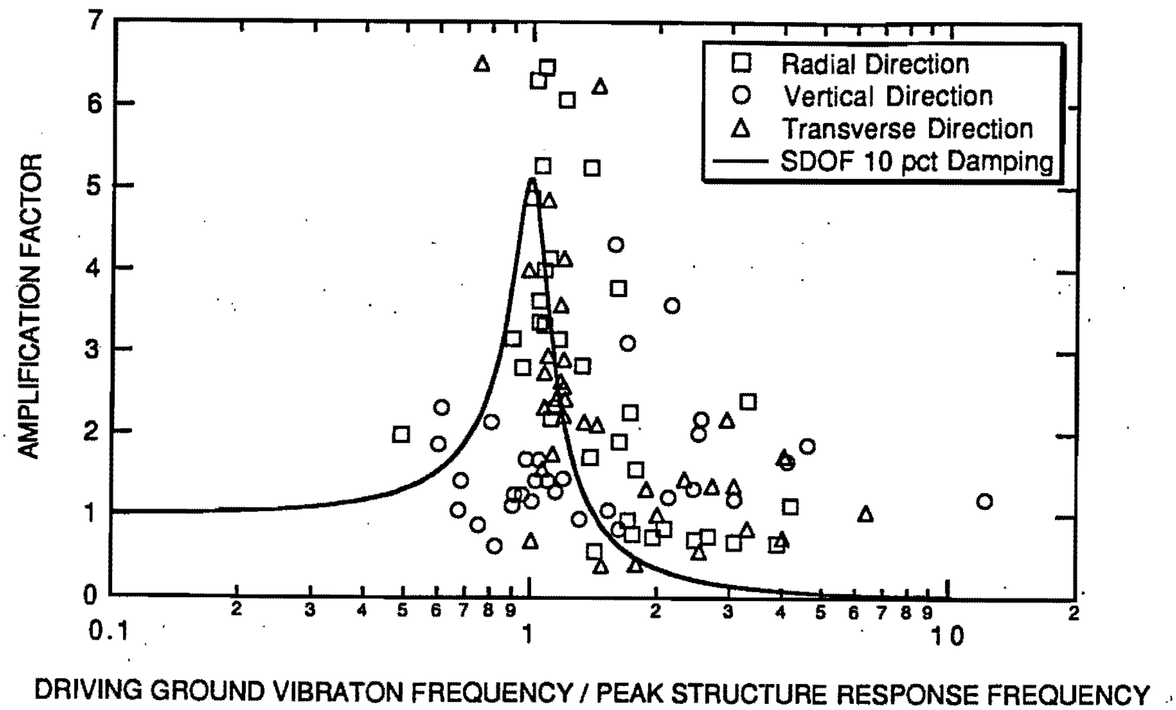

Figure 5. Measured high corner structure response amplification at the Shack versus the ratio of driving ground vibration frequency divided by the peak structure response frequency represented by individual data points. Superimposed on the response measurements is the single degree of freedom (SDOF) response curve for 10 pct damping as shown in figure 4.

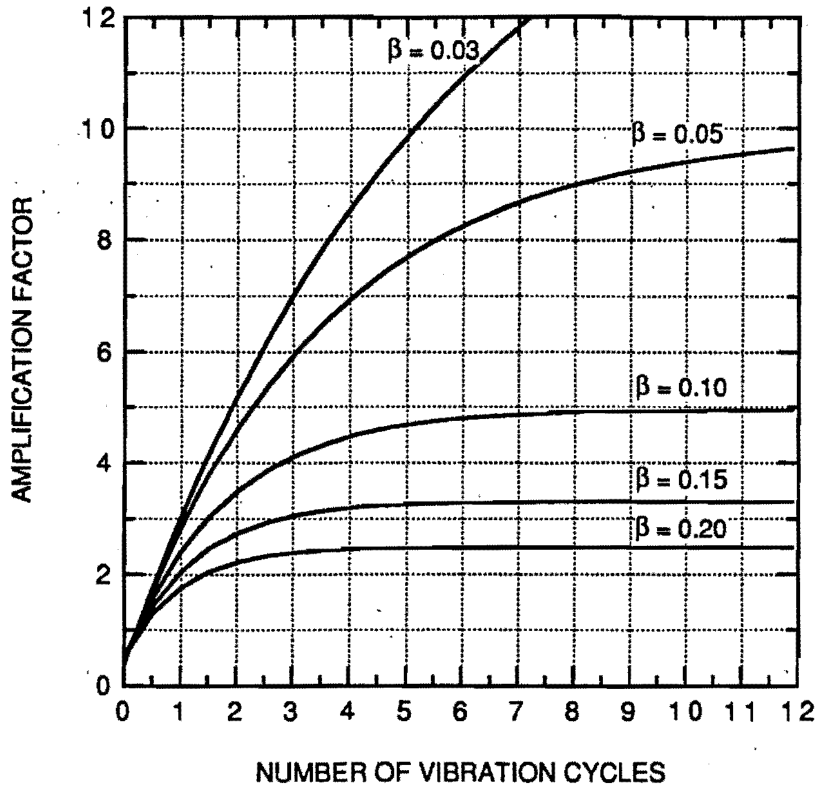

Figure 6. Finite-duration SDOF system assuming continuous, sinusoidal and harmonic excitation at the natural frequency. Curves depict the effect of differing damping values on response characteristics.

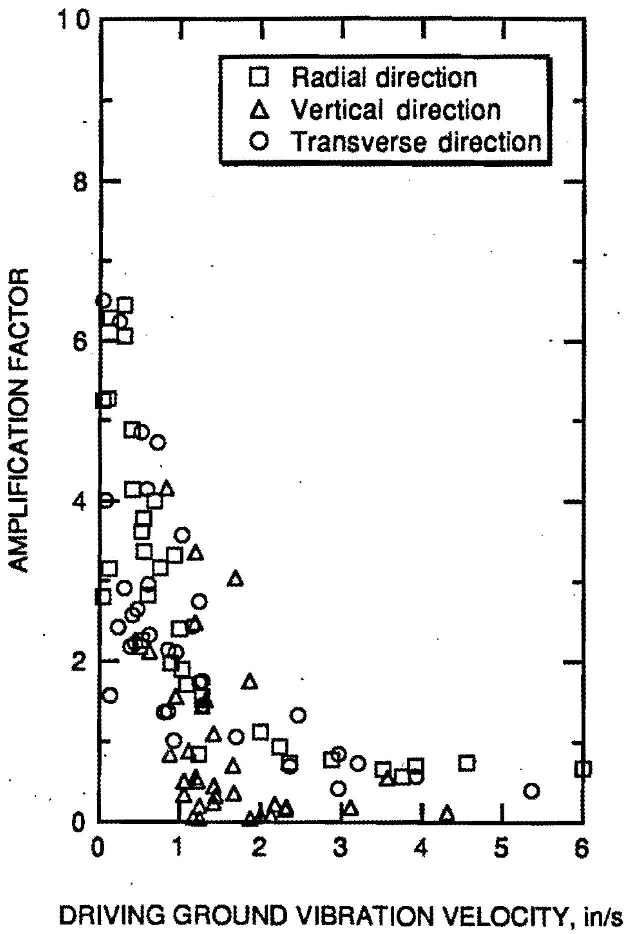

Figure 7. High corner structural amplification versus driving ground vibration velocity (in/s) at the Shack.

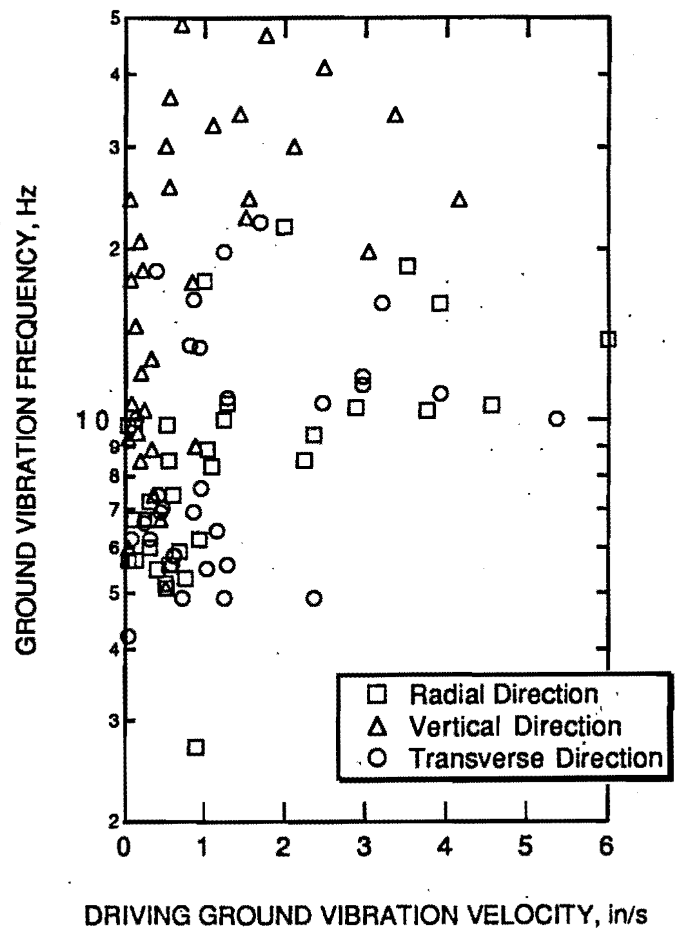

Figure 8. Ground vibration frequency (Hz) versus driving ground vibration velocity (in/s) at the Shack.