

ADDENDUM II

GUIDELINES FOR MEASURING RESIDENTIAL STRUCTURE RESPONSE

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1.0 INTRODUCTION

Occasionally, structures near mines may be instrumented to determine their response to incoming ground vibrations and airblast. These guidelines will assist with instrumentation and analyses to measure and evaluate structure response from blasting. The methodologies described in these guidelines are intended to ensure consistent measurement of structural response and evaluation of structure characteristics. The field techniques employ traditional vibration instrumentation with exact time correlations.

The methods detailed herein allow for the proper:

- comparison of velocity vibration time histories within structures relative to ground vibrations,
- evaluation of the influence of ground vibrations and airblast on the structure,
- evaluation of structure response to determine the natural frequency,
- determination of structure response amplification of ground vibrations,
- computation of differential displacements of construction components and corner motions to estimate global and in-plane tensile wall strains, and
- comparison of results with historical vibration observations.

2.0 STRUCTURE RESPONSE

2.1 Background

Current regulations to control blasting vibrations are based upon measurements of ground vibrations. There may be occasions to directly measure structure motions. In such cases the most exact procedure will involve measurements of motion that allow back calculation of wall strains. These wall strains are most exactly calculated from displacements measured near the top and bottom corners of walls of uniform construction.

The below ground portion of a structure or basement to which the superstructure is attached is normally well coupled to and vibrates with the ground for most dwelling. Often the lower first floor wall corner motions are similar to those of the ground. When this is the case, ground motions may be used to estimate lower wall corner motions. *However, trailers may be an exception.* Trailers and many manufactured homes normally rest on uncemented concrete masonry unit (CMU) pillars without perimeter wall support and are not tied to interior piers. This results in poor coupling to the ground because each block is free to move independently and ground velocities are not effectively transmitted into the structure.

The above ground portion of a structure shakes or responds to ground vibrations and/or airblast with three degrees of freedom: front to back, side to side, and torsionally. However, trailers tend to “bounce” and are free to readily vibrate in the vertical direction at amplitudes that may be greater than ground vibrations. When the frequency of the incoming vibrations matches the natural frequencies of the house, the whole (or gross) structure horizontal response may be

amplified and sustain velocities greater than the horizontal velocities measured in the ground. The greater the difference in frequencies between the vibration of the ground and the house, the less the house responds in amplitude. The natural frequency of typical homes is between 4-12 Hertz (Hz).

Figure 1 shows simplified diagrams of whole (gross or corner) structure and mid-wall responses. Figure 1(a) is a plan view of a typical structure showing the convention used for velocity sensor orientation with respect to the long axis of the structure. Vertical wall diagrams are shown of wall 1 (Figure 1b) and wall 2 (Figure 1c). Whole structure deformations are represented by upper corner (labeled as S2) displacements minus those at the lower corner (S1). Wall bending response (mid-wall, MW, motions normal to the wall surface) is approximated in the wall section view Figure 1(d) and applies to both walls 1 and 2.

A sensor placed in the ground with the proper orientation to compare with structure response is shown in Figure 1. Normally the longitudinal or radial component is directed parallel to the longest axis of the structure. *Note that this sensor orientation may be contrary to the convention that is traditionally used when the radial or longitudinal velocity transducer is pointed toward the blast.*

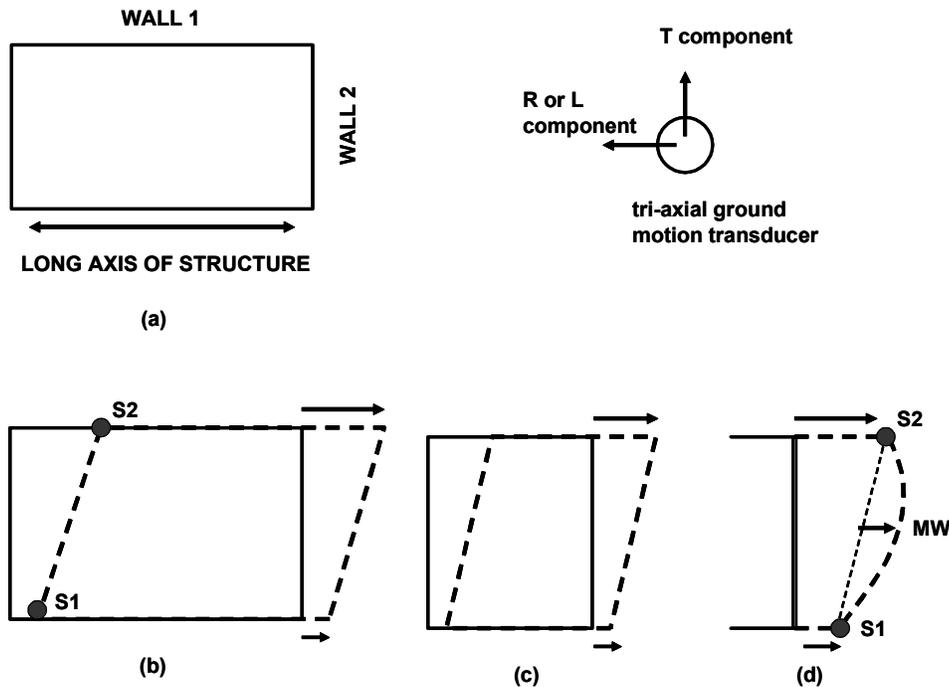


Figure 1 (a) Plan view of structure showing nomenclature for walls, (b) wall 1 and (c) wall 2 whole structure shearing, and (d) typical mid-wall bending

2.2 Natural Frequency

The natural frequency of a structure is most easily determined as it comes to rest once ground motions have stopped as discussed later. Full waveform records allow visual interpretation of the natural frequency by simply counting the number of cycles per second during these free vibrations. When no free response of the structure is obvious because of continued ground motion or airblast, Fast Fourier Transform (FFT) spectral analysis may be necessary to approximate the natural structure frequency.

2.3 Amplification

The above ground portion of each structure will respond more than the ground when excited at its natural frequency. Amplification is a comparative measure of the maximum structure response to ground velocity (GV) at the same point in time. Amplification occurs when the motion at S2 becomes larger than that at GV. Amplification varies for typical and atypical structures. When the ground vibration frequency is significantly higher than that of the structure the motion is equal to that of the ground

2.4 Strains

Strains determine the likelihood of cosmetic cracking. Global (whole) structure strains may be estimated from the measurements of differential structure motions calculated in terms of displacements. The process of calculating displacements involves integrating the velocity time histories at S1 and S2 to obtain displacement time histories and finding the largest time-correlated difference between corner responses (S2 minus S1) over the recorded time history.

Lower corner motions (S1) near the structure foundations often have the same response relative to ground vibrations (GV). If this is the case, GV approximates S1 and may be used for strain calculations. For structures that are not well-coupled to the foundation and the ground (e.g. trailers), the lower corner may move in a different manner than the ground vibrations (GV). If this is the case, S1 monitoring is necessary. Measurements of existing crack motions correlate best with the difference in integrated velocity time histories from the upper and lower corners (S2-S1) as shown in Addendum I of “Comparative Study of Structure Response to Coal Mine Blasting” (Aimone-Martin, et al., 2003).

3.0 INSTRUMENTATION

3.1 Blasting Seismographs

Methodology in this Addendum assumes that commonly-available blasting seismographs are used and that they adhere to the “ISEE Performance Specifications for Blasting Seismographs”. *It is recommended that the seismograph manufacturer be contacted to ensure that the necessary hardware and software are available.*

While the results presented by Aimone-Martin, et al., (2003) were obtained using small single axis transducers, small tri-axial transducers (less than 2.0 inches square or diameter) may be employed if installed properly. The disadvantages of using tri-axial sensors are the mounting requirements needed to support the large sensor mass on walls, and the large number of seismographs required to measure both whole structure and mid-wall responses. In addition, only three of the four seismograph channels are utilized with interior units

At a minimum the instrumentation system includes two blasting seismographs to measure whole structure response. If differential response between GV and S1 is possible, then three seismographs are necessary. If mid-wall motions are required, one to two additional blasting seismographs will be needed.

3.2 Time-Related Motions

All records should be time correlated. This can be accomplished by physically connecting the seismographs in series. A common time base is produced by physically connecting the units. The exterior (master) seismograph, once triggered, must in turn produce a signal to trigger the interior seismographs. A single cable can be used to connect the two (or more) seismographs in series via the serial download port using “Y” cable connectors.

In addition, the resolution (or gain) designed for the airblast and vibration channels must be checked for consistency. Otherwise, modifications may be made to the connector (interface) box to vary the gain for some or all of the channels.

3.3 Polarity Testing of Tri-axial Sensors

When comparing time correlated motions it is important to document the polarity of the sensors for any given direction of motion. Prior to deploying equipment in the field, polarity testing should be conducted on all sensors to ensure common transducer wiring for consistent voltage output (positive or negative). For example, the vertical component should show positive velocity amplitude for vertical ground motion in an upward direction. This is *normally* the case for the first arrival of ground vibration. The radial or longitudinal component should show positive amplitude when motion is in the direction of the arrow traditionally placed on the sensor housing to indicate the R (or L) direction. The transverse horizontal component can be mounted with any convention and it is difficult to predict polarity direction. However, each manufacturer will generally use a consistent convention for attaching wire leads to the transducer and transverse time history outputs among each sensor should be similar for any given motion direction.

Polarity becomes critical when measuring and comparing relative motions between the ground or lower structure corner with the upper corner of structures, particularly in the horizontal components. This directionality is important because differential displacements must be calculated in order to estimate gross structure strains and in-plane wall strains. If polarities are not matched, differential displacements may be over two times greater than displacements for a common polarity.

Polarity tests can be easily performed by taping all sensors mounted with identical orientations to a sturdy but movable object such as a cardboard box. Move the box in each of three directions parallel to each sensor component (for instance, against the radial or longitudinal “arrow” and in a vertical direction), recording each motion using the smallest record length practical. Using the seismograph software, plot and compare the time histories near the time history arrival to observe consistent first motion arrival pulses (positive or negative).

4.0 INSTRUMENT INSTALLATION

4.1 Instrument Locations

At a minimum the instrumentation system includes two blasting seismographs to measure whole structure response as shown in Figure 2, one inside the house (S2) and one outside (GV, not shown). If GV cannot approximate S1 a third seismograph is necessary. If mid-wall motions are required, one or two additional sensors can be mounted as shown in Figure 3.

For two story structures, it is important to place the upper and lower corner transducers on one floor at a time. This is particularly important when the construction materials are different among the different stories. Wall strains computed from differential displacements measured at

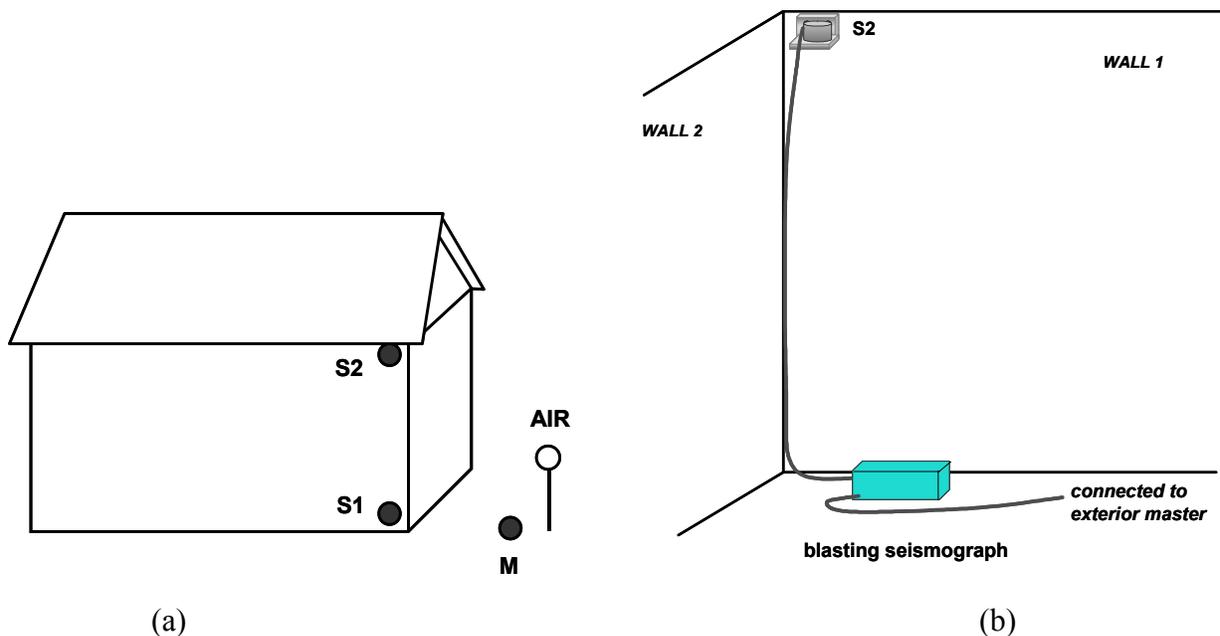


Figure 2 Positions of velocity transducers to measure whole structure response and excitations (a) showing the interior units relative to the exterior master (M) triggering unit and (b) showing a cut-away section of the interior corner sensor locations

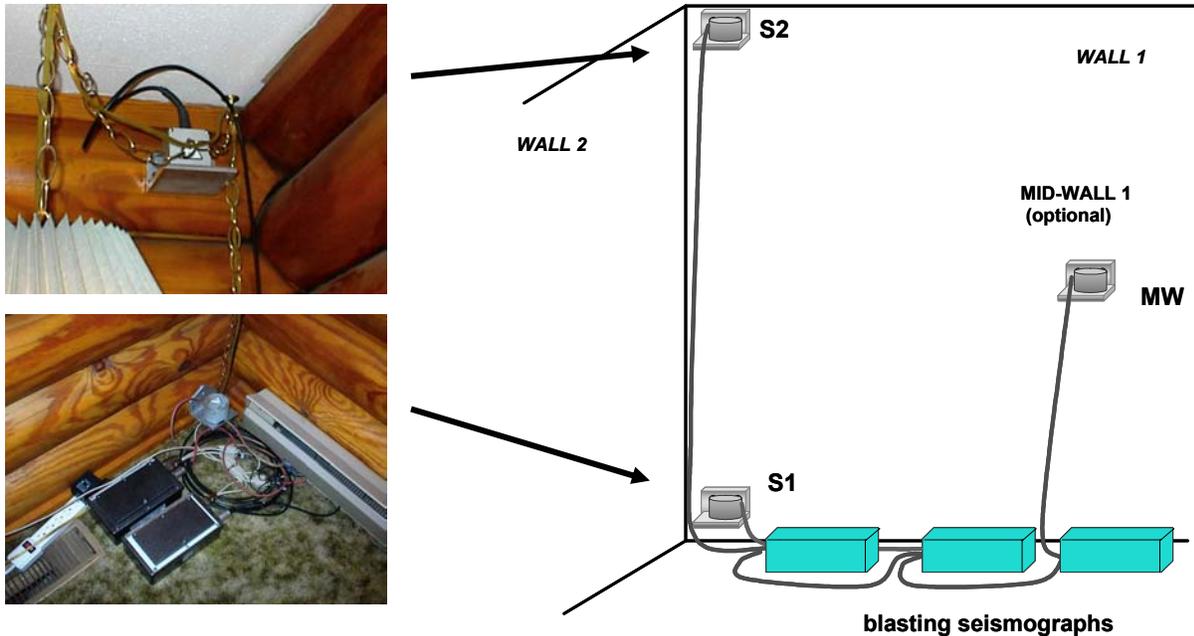


Figure 3 Transducer positions to measure whole structure and mid-wall (one wall only) bending response

wall corners can only represent one wall or material and should not be extended to two stories unless the structure can be assumed to respond uniformly as shown in Figure 1.

4.2 Ground Vibration Measurements

A blasting seismograph should be set outside the house at the same corner of the intended structure monitoring. Installation should be consistent with the “ISEE Field Practice Guidelines for Blasting Seismographs.” The only exception is the orientation of the ground tri-axial sensor where the radial channel must be aligned with the long axis of the house as shown in Figure 1.

4.3 Mounting Sensors Inside the Structure

Methods to attach single-axis sensors that disturb walls the least include drywall screws or gluing. Tri-axial sensors, because of their large weight in size, must be screwed to the wall. Care must be taken to minimize the damage to residential walls. Owners of structures may be willing to allow minor wall surface damage from screws or gluing. However, it is always prudent practice to explain to the owner that surface cosmetic wall damage may occur and be prepared to provide minor wall repairs.

While single axis sensors are preferred, tri-axial sensors may be employed if properly installed. Tri-axial sensors require “L”-shaped brackets for interior mounting as shown in Figures 2 and 3. Because of the weight of these sensors, drywall screws must be used to attach to walls. Standard aluminum 6061 T6 structural angles, 1/4 to 3/8 in. in thickness, are suitable

for bracket material. The horizontal base should be sufficiently wide to accommodate the sensor. A small amount of hot glue using a glue gun is adequate for affixing the transducer to the bracket. Holes may be machined on the back (vertical) plate for use with screws.

Some sensors may be mounted on the structure exterior. Depending on the component being evaluated, such as a brick veneer, outside mounting may be essential. It is important to ensure that the brick veneer is attached to the load bearing walls with masonry tabs.

4.3 Mounting at Other Structures

Other structures such as water towers, silos, electric towers, barns, etc., may be monitored by carefully mounting seismographs in the appropriate locations on a case-by-case basis.

5.0 RESPONSE MONITORING

5.1 Waveform record

Full waveform records of each event are necessary to evaluate the structure response characteristics. Be sure that the data is recorded digitally for subsequent analysis.

5.2 Seismograph Settings

All seismographs in series must have the same settings for sample rate and record length. Sample rates of 1024 or 512 per second are sufficient to measure structure response. Total record length should be long enough to ensure the recording of any free response well after the arrival of the airblast pulse and well after the ground motions have ceased. Depending on the structure distance from the blast, set the record time for a total time equal to at least five to seven seconds *plus* one second for every 1000 ft. of distance to the blast. For example, if the structure is 3000 ft. from the blast, a minimum record length of 8 to 10 seconds is required.

The exterior master unit must be set on the trigger mode while the remaining seismographs measuring structure response should be placed on manual or slave mode. Again, this setting depends on the manufacturer. Ground motion and airblast trigger levels of 0.03 inches per second (in/sec) and 125 decibels (dB) are recommended.

The upper range for the interior seismograph units should be set to at least 5.0 in/sec to capture the higher velocity motions that may be reached in the upper structure and mid-walls. However, if the distance to the blast is beyond 2000 to 3000 ft. and expected peak ground velocities are less than 0.3 in/sec, an upper range of 2.5 in/sec will allow good resolution of structure time history motions at low amplitudes. It is critical that time histories show amplitudes well above the lowest resolution of the seismograph in order to accurately integrate velocity time-based motions for analysis. It is best to check with the manufacturer to assist with setting the amplitude range to maximize the data quality.

6.0 STRUCTURE RESPONSE ANALYSIS

Structure response analyses are outlined in the U.S. Bureau of Mines and reported in RI 8507 and by Aimone-Martin, et al., (2003). Basic analyses should include estimation of natural frequency and amplification of the structure. Advanced analyses include strain estimation.

6.1 Time correlated waveforms

Velocity time histories of the ground, lower, and upper structure must be correlated in time in order to calculate strains. Two examples of time correlated motions are given in Figure 4. Other approximate approaches may involve errors in calculating displacements.

6.2 Estimating Whole Structure Natural Frequency

Whole structure natural frequency is estimated during free response of the structure, i.e. after the ground vibrations have stopped as shown in Figure 5 or after the airblast induced motion. The normal range of structure natural frequency is reported to be 4 to 12 Hz. Visual estimation, zero-crossing, or FFT methods can be used to determine the structure frequency.

If the waveform is uniform, visual estimation is possible by identifying a one second window during free response and counting the number of cycles. In Figure 5, between seconds 4 and 5 there are 4 cycles for a frequency of 4 Hz.

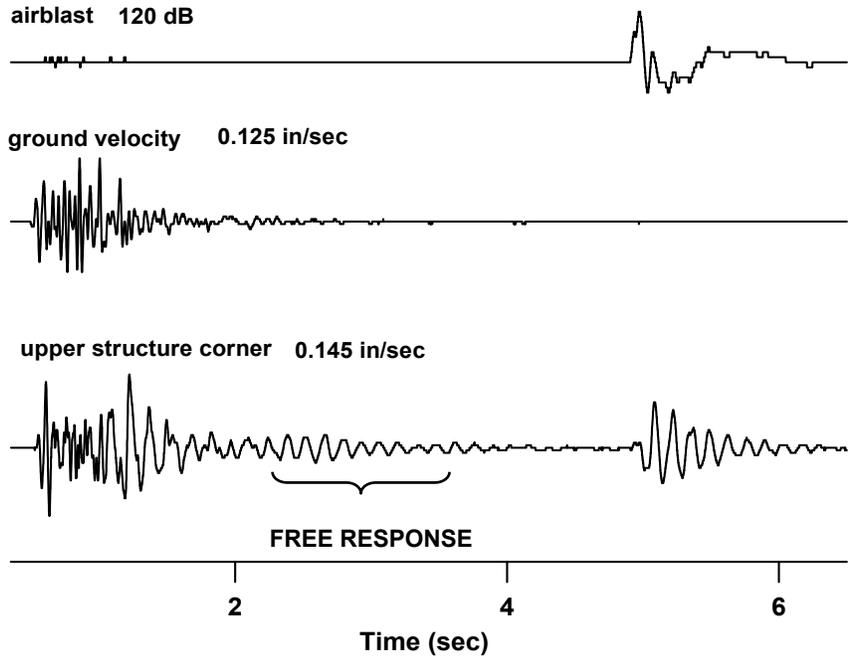
If only a few cycles of free response exist, the “zero” crossing method of calculating frequency can be employed. In Figure 5 near second 4, the wave crosses the zero line 0.125 second apart as determined from the software. Since that is a half of a wave, the period of a full wave is 0.25s. Taking the inverse of the period yields natural frequency of 4 Hz.

One alternative to calculating natural frequency from free response is to calculate the Fast Fourier Transform (FFT) of the response motion. While this may not always yield satisfactory results, it may be useful. Most seismograph software has an option to display the FFT graph to observe the distribution of frequency content.

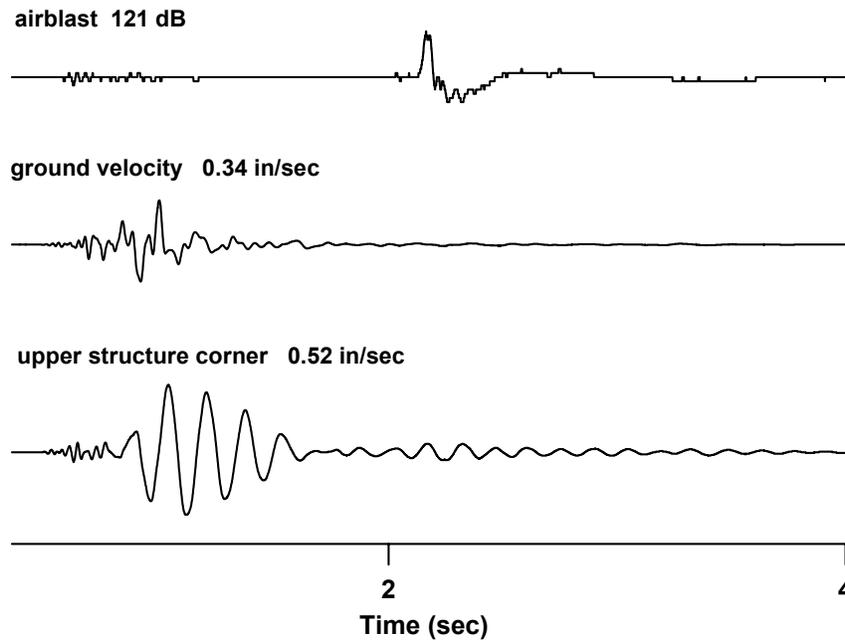
Figure 6 is an FFT plot of the S2 time history in Figure 5. The 3.9 Hz peak value compares well with the 4 Hz computed using the “zero” crossing method over that portion of the time history during free response. Hence, FFT analysis provides a good measure of structure free response. This agreement is good because the record involves free response. When there is no free response, the approach is more complicated as described in Dowding (1996).

6.3 Whole Structure Amplification of Ground Vibrations

Amplification is a comparative measure of the maximum structure response to ground vibration at the same point in time. Amplification occurs when motion at S2 becomes larger than that at S1.



(a)



(b)

Figure 4 Airblast, ground velocity, and upper structure corner response time histories showing strong structure response within (a) the airblast phase and (b) the ground motion phase

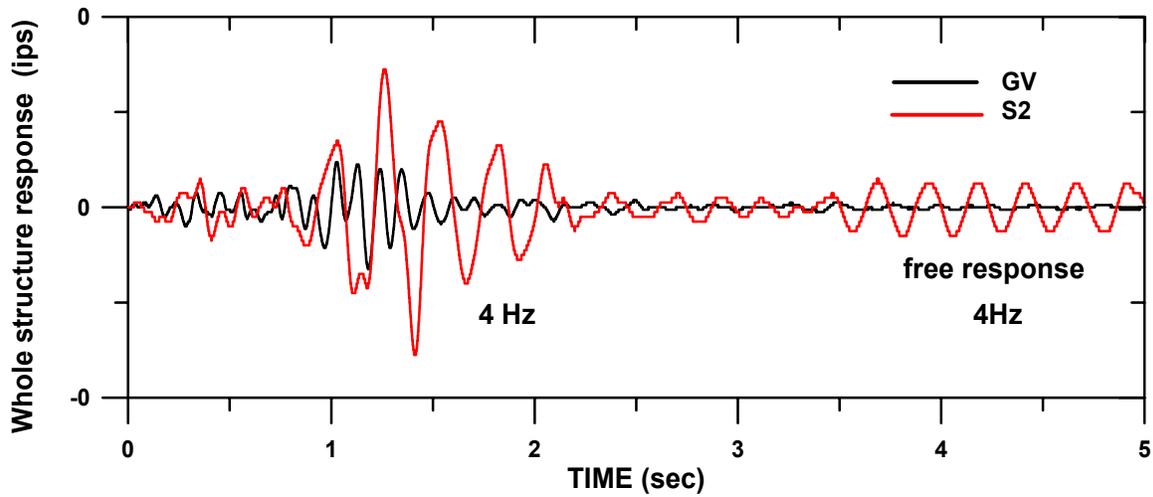


Figure 5 Whole structure response (S2) and ground vibrations (GV) showing structure free response for a single wide trailer at 4 Hz when ground motions have arrested

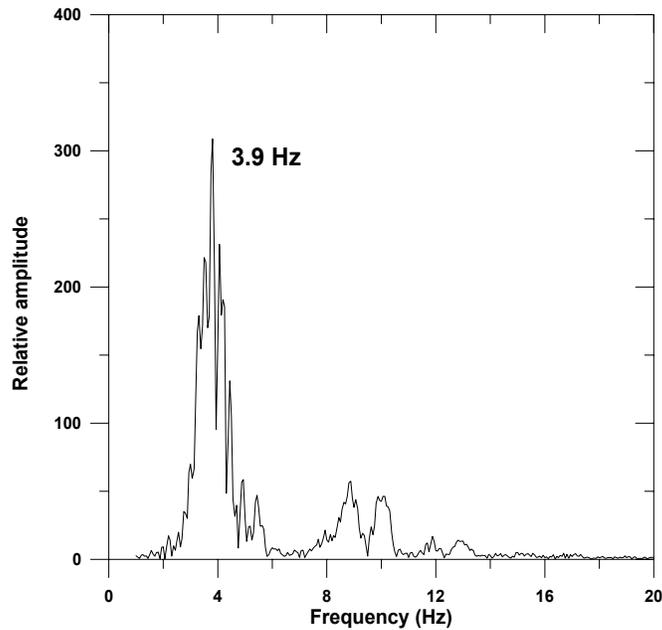


Figure 6 FFT plot of relative amplitude versus frequency for S2 given in Figure 5, showing a strong predominance at 3.9 Hz

In the 1980 study, Siskind, et al., describes amplification in terms of velocity. This method involved finding the peak upper structures velocity response and dividing this by the nearest preceding ground velocity that most likely drove the structure. This approximate approach is described as equation (1),

$$AF = \frac{S2_{peak}}{GV} \quad (1)$$

where $S2_{peak}$ is the peak velocity of the upper structure and GV is the velocity of the ground motion for the same direction component at the corresponding moment of time or immediately preceding the time of the peak $S2$ motion.

Waveform analysis necessary to calculate AF values can be carried out in one of two ways depending on available seismograph software. One way is to display the time histories of the vibration component of interest recorded in the ground (GV) and at the upper structure corner ($S2$) in the same display window shown in Figure 7. Find the peak velocity at $S2$ then establish the peak GV for the same phase (the negative pulse in this case) at a time just prior to $S2$ peak as indicated in Figure 7. The vertical line represents a common time mark to locate peaks. AF for this example is

$$AF = \frac{0.30}{0.135} = 2.22$$

If the seismograph software does not have this windowing feature comparing different recorded events, then the time histories can be converted to ASCII format and saved. Using a convenient plotting software package, the time histories can be plotted together with a common time base and the peaks selected in the same manner described above. A third way is to graphically print the waveforms on the same scale and overlay the waveforms to observe which ground vibration amplitude is driving the peak response.

The range of amplification factors for trailers reported by Aimone-Martin, et al. (2003) are shown in Figure 8. The data can be compared to Figure 39 in U.S. Bureau of Mines RI 8507 where the U.S. Bureau of Mines found the highest AF to be 4 and an average value for all structures of 1.5. The average AF in the 2003 study was 1.9 for all 25 atypical structures with a maximum of 5 at one structure.

To compare and project structure response to ground vibrations, the peak upper corner ($S2$) velocity value of either the R or T component (whichever is the larger), termed peak upper corner horizontal component, is plotted against the corresponding peak horizontal value of ground vibration (GV). An example of this plot is shown in Figure 9. The line shown envelopes of all U.S. Bureau of Mines data reported in RI 8507 Figure 35. In this example, all data in Figure 9 fall within the data range reported by the U.S. Bureau of Mines.

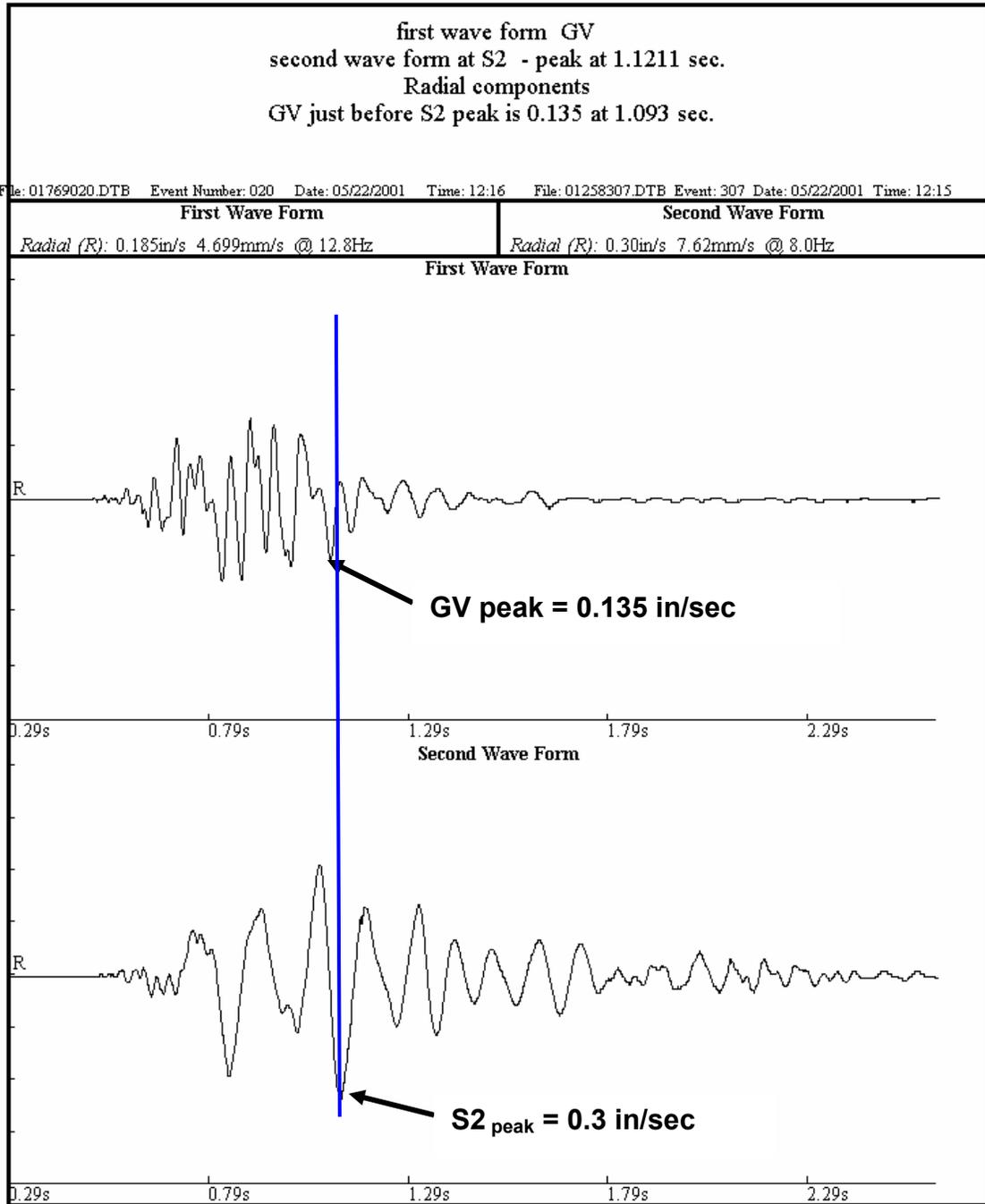


Figure 7 Comparing radial vibration time histories recorded at the upper structure (S2, top) and in the ground (GV, bottom)

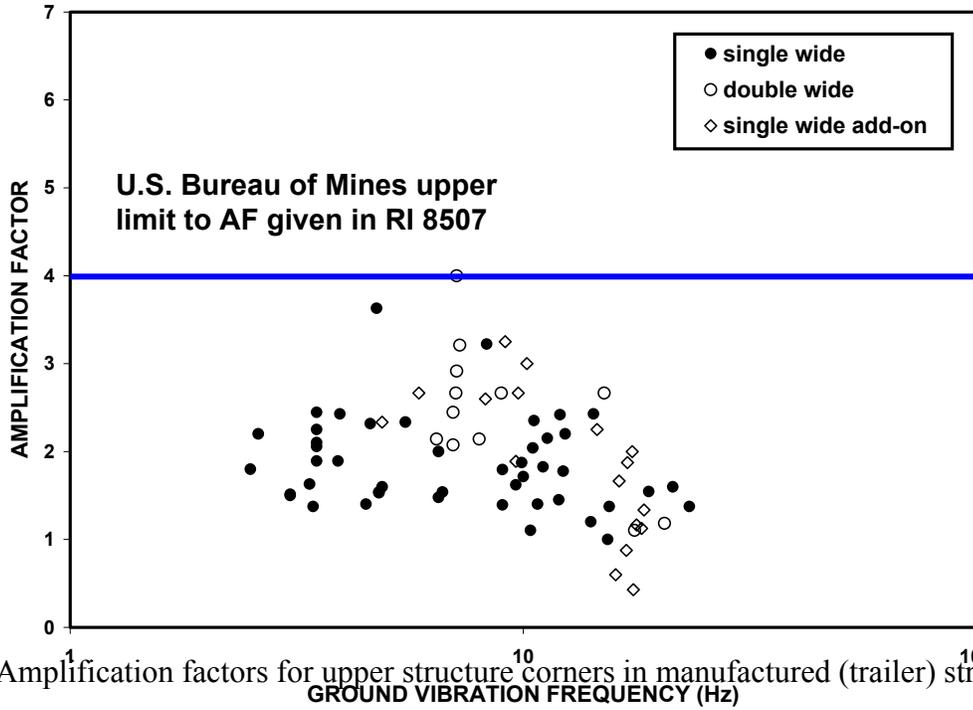


Figure 8 Amplification factors for upper structure corners in manufactured (trailer) structures

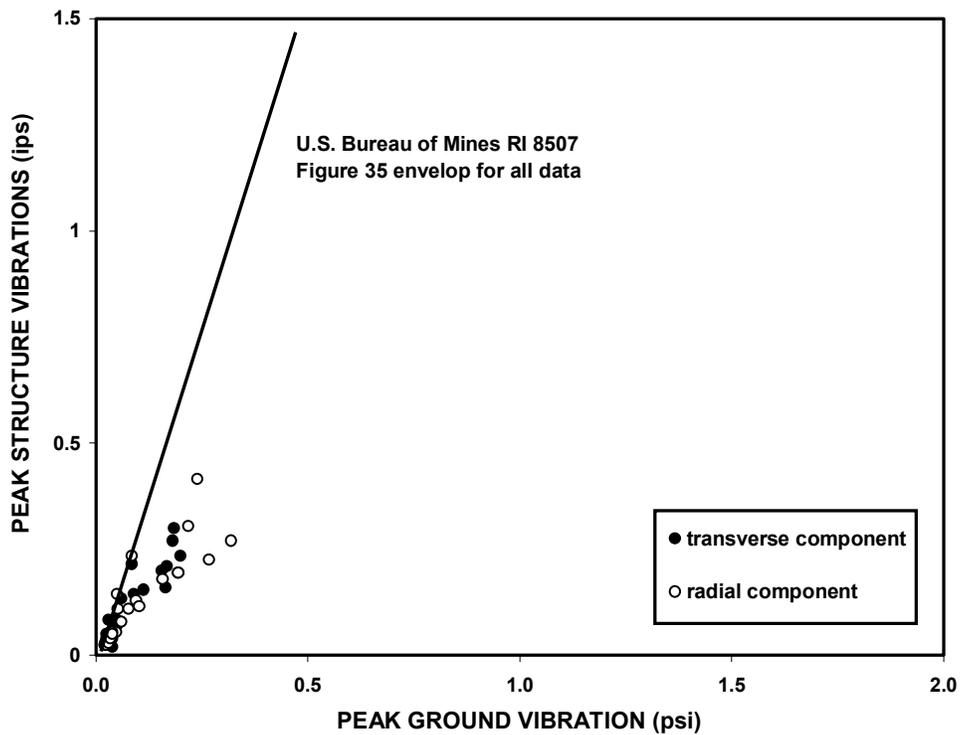


Figure 9 Structure upper corner response from peak ground vibrations for horizontal components, the slope of the line represents an amplification factor of 3

6.4 Airblast Induced Structure Response

Unlike ground vibrations, airblast impacts the house through the roof and walls of the structure. Airblast analyses include evaluating the upper corner response (S2) and mid-wall (MW) response to air over pressure (in terms of pounds per square inch, psi) as an indication of structure sensitivity to airblast. If the seismograph software does not report airblast in terms of pressure, the following relationship can be used to convert airblast pressure in psi to sound pressure level, SPL, in decibels (dB):

$$\text{SPL (dB)} = 20 \log \text{AP (psi)} + 170.8 \quad (2)$$

The natural frequency can be estimated during airblast excitations if free response of the upper structure is observed within the structure time history as discussed in Section 6.2. However, if strains are to be estimated from airblast, seismographs must be located at S1 and S2. Then in-plane tensile wall strains can be estimated from differential displacement time histories between the upper and lower structure corners.

The maximum horizontal structure response at S2 (the larger of wall 1 or wall 2) and the peak mid-wall responses (MW) are plotted against airblast overpressure, in pounds per square inch (psi) in Figures 10 and 11 taken from Aimone-Martin, et al., (2003) and compared with historical data provided by Siskind (2002). Whole structure response sensitivity to airblast in Figure 10 was found by Aimone-Martin to be 77 in/sec/psi, for well-confined blasts, and 155 in/sec/psi, for unusually high frequency airblasts. In comparison, historical U.S. Bureau of Mines structure response data and values provided by Siskind (2002) for equivalent type airblasts were 42 and 135 in/sec/psi, respectively.

In contrast to whole structure response, the envelope for trailer mid-wall responses to airblast shown in Figure 11 was 442 in/sec/psi. For other structure types, exclusive of trailers, the upper envelope was 266 in/sec/psi. Siskind (2002) reported an upper envelope of 319 in/sec/psi that did not include trailers.

6.5 Strain Analysis

Lower structure horizontal responses (S1) are generally equal to or lower in amplitude than the same component of ground motion for all structure design with the exception of trailers and other structures that are not well-coupled to the ground. By comparing the peak amplitude of the lower corner with the ground motion, a determination can be made whether or not a lower corner sensor is necessary for structure response measurements. However, if strains are to be estimated from airblast impacts, measurement at S1 is essential.

The time histories of wall 1 for three trailers taken from Aimone-Martin, et al., (2003) are shown in Figure 12 to illustrate this point. Trailer dimensions are long (wall 1) compared with the width (wall 2) and they tend to exhibit higher response in a direction perpendicular to the long axis. The lower corner response (S1) of the single wide is higher in amplitude than the ground motions, whereas the double wide and single wide add-on designs do not show this lower corner amplification of ground vibrations. Therefore, in the later two cases, a lower sensor may

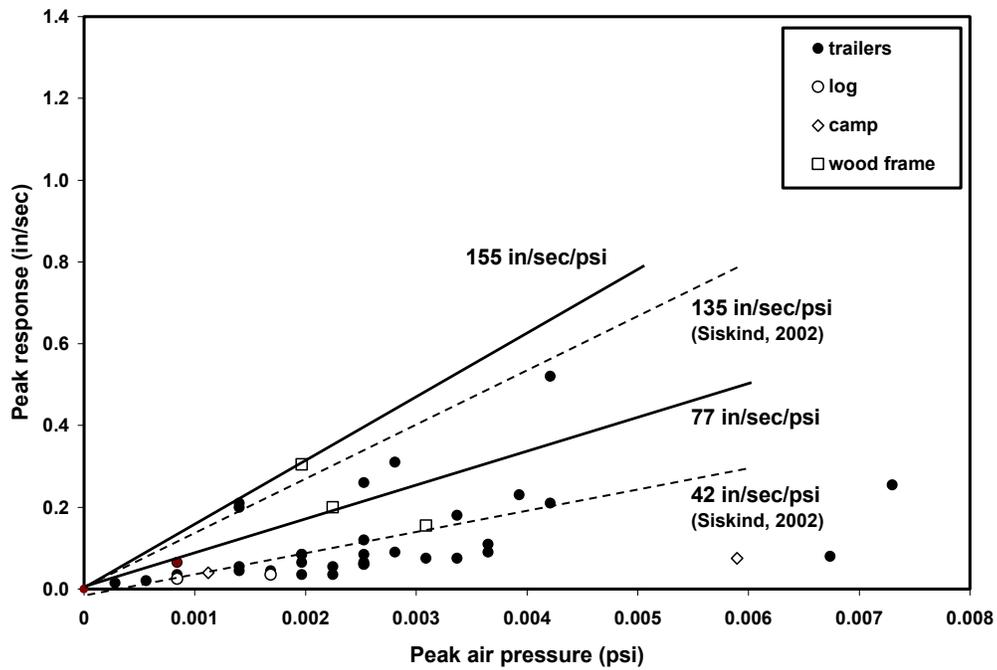


Figure 10 Airblast-induced whole structure response measured at S2

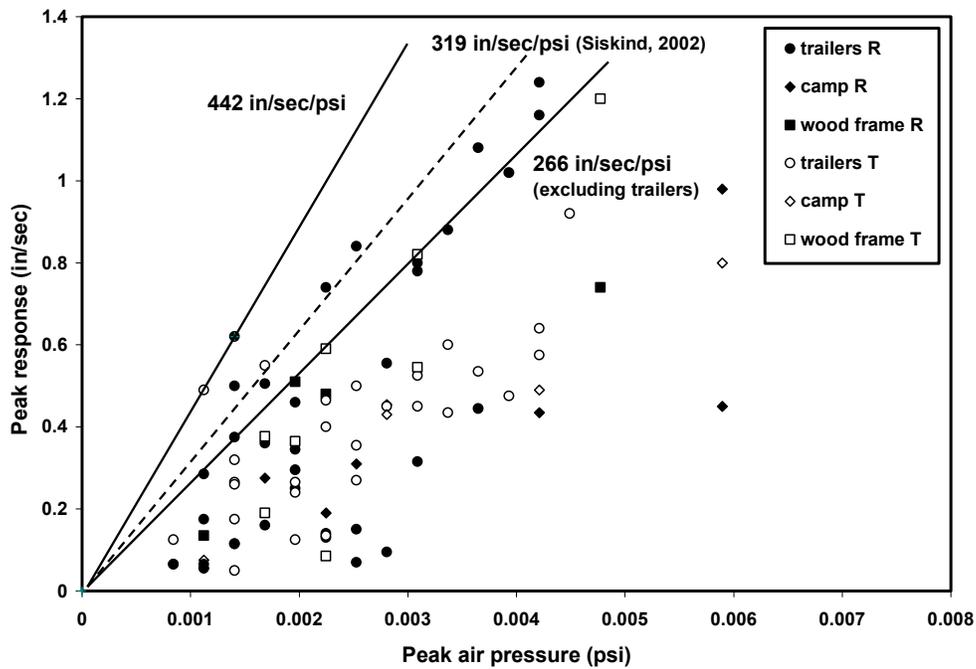


Figure 11 Airblast-induced mid-wall response (wall 1 = T and wall 2 = R in this case)

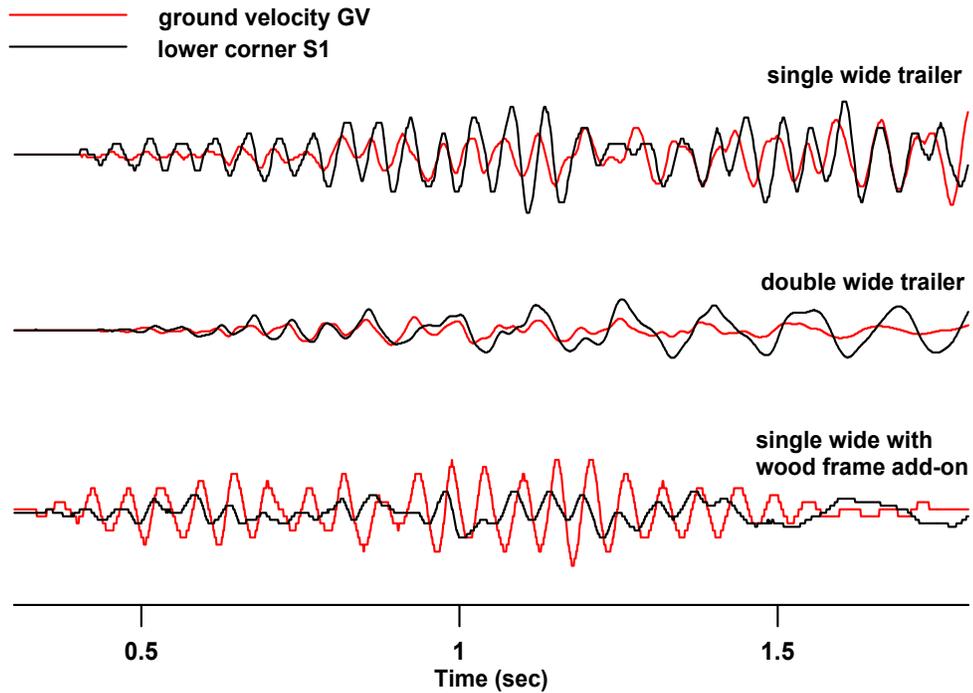


Figure 12 Comparison of velocity time histories for the ground (GV) and lower structure corner (S1)

not be necessary and lower corners may be estimated using the ground motions. However, for single wide trailers, a lower sensor is necessary for structure response analysis.

Estimating wall strains from calculated gross structure shear strains requires the calculation of differential structure displacement time histories, or S2 minus S1 over time. The ASCII format of velocity time histories for the upper (S2) and lower (S1) structure corners are first converted to displacement time histories by mathematical integration using seismograph software (if this feature is available) or using software such as NUVIB developed at Northwestern University, MATHCAD®, or similar software. By computing S2-S1 over time, the peak or maximum whole structure differential displacement, $\Delta\delta_{\max}$, between the upper and lower structure corner can be determined.

The maximum global structure shear strain of the wall, γ , is computed knowing the wall height (or measured distance between the transducers placed at S2 and S1) as follows:

$$\gamma = \frac{\Delta\delta_{\max}}{L} \quad (3)$$

where L is the wall height in inches and $\Delta\delta_{\max}$ is in inches. Therefore γ is given as $\mu\text{-in./in.}$ or $\mu\text{-strains}$.

The maximum in-plane tensile wall strain, ϵ_L , for a square wall is then computed as

$$\epsilon_{L(\max)} = (0.5) \gamma_{\max} \quad (4)$$

In-plane tensile strains are critical to threshold wall cracking potential. Refer to Siskind (2000) for crack threshold strain levels in various materials.

7.0 Structure Response Evaluation

The goal of structure response measurements is to recognize when responses exceed values that may indicate unusual vibration characteristics. The primary indicator of damage potential is strain, which is related to frequency matching, structure amplification and gross structure differential motions. If structure vibrations approach or exceed historical observations, blasting may need to be modified to keep damage probabilities within acceptable ranges.

Ground vibration limits are typically between 0.5 and 1.0 in/sec while airblast limits are typically 133 dB or 0.01288 psi. When ground vibration frequencies match the structure's natural frequency (4 – 12 Hz), structure response amplitude to either the ground velocity or airblast may be as high as 1.5 in/sec measured in the upper corner (Aimone-Martin, et al., 2003). If responses are measured in excess of 1.5 in/sec, strain estimates should be made and compared to the threshold tensile strains for the appropriate building material.

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