

RESPONSE OF UN-CRACKED DRYWALL JOINTS AND SHEETS TO BLAST VIBRATION AND WEATHER

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ABSTRACT

Cracking is one of the most common concerns cited by owners of structures adjacent to construction or mining blasting. While a large database of case studies documenting the relative insignificance of ground motion induced by responsible blasting compared to weather effects on cracks in nearby structures has been established, the perception of damage to structures, particularly residences, remains common. In allegations of blast damage, the utility of the database of crack response to weather and ground motion is downplayed by citing that the behavior of the structure is altered by the existence of cosmetic cracks. To shed light on the influence of existing cracks, this study will compare the response of cracked and un-cracked areas of gypsum board in two structures – one near a surface coal mine in Indiana, the other near a limestone quarry in Florida – to blast-induced ground motion and air overpressure as well as changes in temperature and humidity.

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INTRODUCTION

Change in Crack width is an index of possible crack extension

Autonomous Crack Measurement [ACM] produces an index of the potential for cracks to extend from measurement of micrometer changes in crack width. The logic of this index is similar to splitting wood with a wedge as shown in **Figure 1**. Hammering the wedge into the wood increases the width of the crack, extends the crack, and eventually splits the wood. If the wedge is backed out, the crack would be less stressed, but still respond to small movements of the wedge. Only when the wedge is advanced beyond its farthest penetration (or the split is opened beyond past movement) will the wood split advance. Thus comparing changes in crack width provides a comparison of the potential for crack extension.

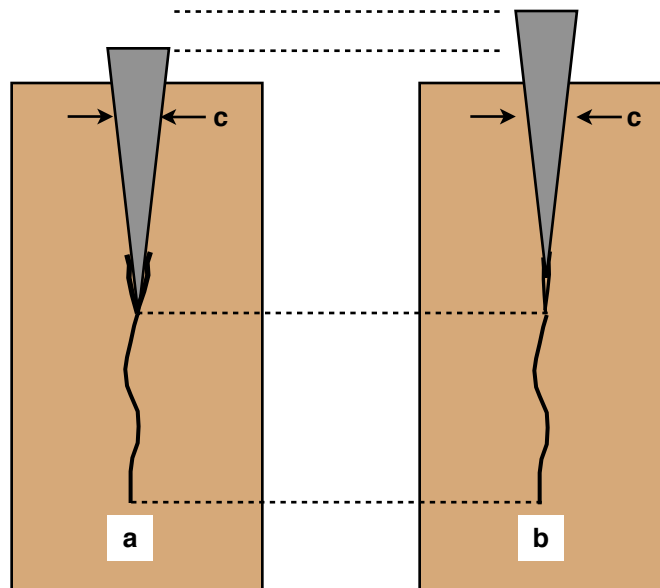


Figure 1 - Wedge splitting wood analogy with “a” the deepest penetration (widest opening at c) and “b” with wedge backed out (narrower opening at c).

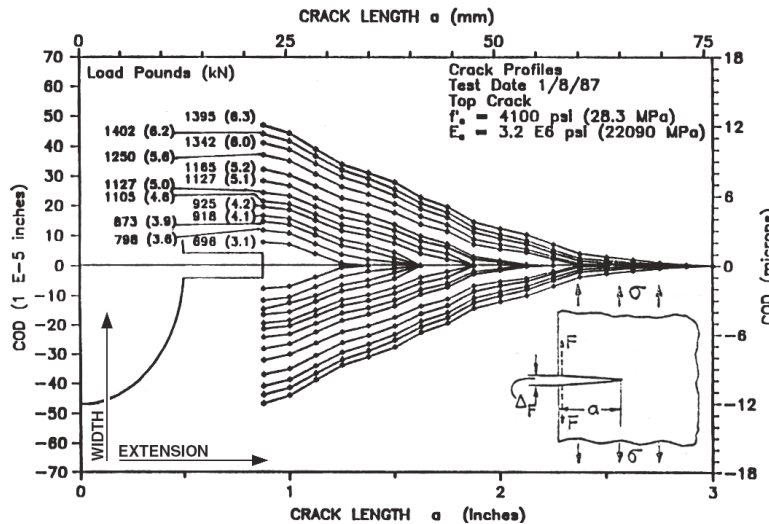


Figure 2 -Experimental observation that cracks extend as their width increases forms the foundation of fracture mechanics as well as the ACM measurement approach. Special visualization techniques were employed to measure the extension of a crack (marked by the rightward extension of the “>”) as its width (COD or “crack opening displacement) increases (marked by the increasing width of the mouth of the “>”) on the left. (Miller, 1989).

The wood splitting analogy is experimentally confirmed for fracture of cement paste as shown in **Figure 2**. Crack mouth opening (COD) on the vertical axis (similar to the action of the wedge to widen the penetration) is compared to fracture extension (length of the crack tip) on the horizontal axis. As the wedge width, COD, increases from 90 to 270 micro inches (2.25 to 6.75 micrometers), the crack extends from 1.4 to 2.1 inches. The graph itself displays both the opening and the extension as they increase in concert. Fracture extension by increasing crack mouth opening – crack width-- is the fracture mechanics foundation for the ACM approach.

Just as splitting wood requires the “V” from the wedge to be progressively widened by the wedge, it stands to reason that crack width must increase beyond its previous maximum for the crack to extend. Since it is unlikely that measurement would begin at the previous maximum width, the question then becomes, “what outside effects produce the largest change in crack width?” Those changes are the most likely to extend cracks. It also stands to reason that cyclic response at widths smaller than the maximum will not extend the crack. As has been measured in the more than 30 crack studies reported in Dowding (2008) climatologically induced changes in crack width (described as response) are far larger than those produced by typical vibratory induced response. Thus at present vibratory limits the most likely causes of crack extension are climatological effects.

Alternate Hypotheses

Concern has been expressed about the conclusion that crack measurements show that there is a floor below which vibrations have no cracking potential. These concerns involve at least the following assertions 1) cracks are not locations of current maximum strain and un-cracked locations may be more strained by vibration, 2) there are critical excitation motions that can maximize response that are not included in the data, and 3) there are maximum initial strain conditions in structures that render them vulnerable. These concerns have arisen because of several of coalescing points of view. First, there is the need to ensure that all critical factors have been included. Second there is the sensory difficulty of believing that environmental effects, which are silent, can be more influential than those that are noisy and disturbing. Finally there is the age-old issue of proximate cause: the assertion that even a small vibration can cause cracking if it occurs at the moment all of the other effects combine to maximize the strain in the wall. The three concerns will be addressed briefly first and then the first will be explored more thoroughly with data. Exploration of the second and third will be addressed in other papers as this paper is already too long after exploring the first two concerns.

Consider first the concern of most sensitive location. It has been hypothesized that once a crack is formed, the strain concentration is relieved and the large local deformations leading to cracking are reduced. Thus cracks are now positions of low strain or deformation and thus low potential for cracking. What may then be important is response of un-cracked locations. This paper will explore two case histories that involve measurement of the response of multiple, weak but un-cracked locations in gypsum drywall. These weak locations are the joints between drywall sheets. Dry wall joints are comprised of a thin, paper tape covered with 2 to 3 mm (1/16 to 1/8 inch) of plaster. The sheets themselves are composed of 12 mm of gypsum encapsulated by 2 to 3 mm of cardboard. All things being equal, the paper thin joints are weaker than the half inch thick sheets themselves. Response of the joints to long term, environmental effects will be compared to the response to vibratory effects. The long term and vibratory response of un-jointed locations on drywall sheets (basically the null response) will also be compared. Both or these responses will be compared to that of a cracked section where the crack was not fully extended.

Second, consider critical excitation. Critical is most often defined as high amplitude (particle velocity) excitation at the natural frequency of the structure or its components. It has been hypothesized that not enough cases of low frequency, high amplitude motions have been observed. If these low frequency events had been observed, higher amplification would have occurred which would have lead to higher dynamic crack response. Low frequency excitation would be that which would be equal to the natural frequency of the walls

and the super structure, 10 to 20 Hz and 5 to 10 Hz respectively. High amplitude would be near or exceeding 12 to 25 mm/s (0.5 to 1.0 inches per second).

The second section will briefly focus on response of un-cracked sections of wall to low frequency, 5 to 7 Hz, motions. The house instrumented with Kaman gauges in 1986 at the Universal mine in Indiana was subjected to such low frequency excitation and high amplitude motions. In several instances the amplitudes exceeded 12 mm/s at low excitation frequencies. Response of this house can be linked to cracked and un-cracked drywall joint response to explore the effect of excitation motions whose frequency matches that of the super structure. Excitation motions with dominant frequencies that match those of the walls, 10 to 20 Hz, are involved in almost all cracking studies and require no special investigation.

The third concern for proximate cause or "the straw that breaks the camel's back" will be addressed only briefly as there is not enough room for a suitable presentation with data. Proximate cause is one "without which the crack could not have occurred." Thus it will be instructive to consider the probabilities of effects other than blasting causing cracking and their relationship to the "natural and continuous sequence of events" in relation to all events that can occur. For the small vibration crack response to be the straw, the crack would have to be precariously on the brink of extension at the moment the ground motion reaches the house, and there can be no other straws in the air to land on the camel's back. For this brink to occur, the crack would have to be subjected simultaneously to the peak widths caused by the 1) historically largest extreme weather event (e.g. a drought that occurs in seven to ten year cycles), 2) largest seasonal response (e.g. high seasonal heating, cooling, or groundwater induced response), 3) largest weather front response (e.g. long period of high humidity), 4) largest daily temperature response (e.g. a few hours in the afternoon sun), and 5) high ground motion. Given the daily swings in crack response, this condition would exist only at a brief moment during one hour of the worst weather front week in the worst heating/cooling season during an extreme (drought, flood, etc) climatological condition. Another paper will address the probability of such an occurrence and other related exogenous events.

1. RESPONSE OF UN-CRACKED, WEAK SECTIONS OF WALLS

House and Crack Descriptions and Vibration Environment

Measurements described herein were obtained in two houses whose photographs and floor plans are shown in **Figure 3**; one in Blanford, Indiana and the other near Naples, Florida. The Indiana house contains two, instrumented, un-cracked drywall joints and a cracked drywall joint for comparison. Multiple sections of the house shown in the photograph were built over a period of 10's of years, with the middle the oldest and the right most, two-story section the newest. Each section is built on a basement, with a full basement under the two-story section, a shallow basement beneath the middle, and a crawl space beneath the left (Dowding, 1996). The walls, interior and exterior, are constructed of standard wood studs and were covered in drywall for the observations. The Florida house contains an instrumented drywall joint in the garage ceiling. It is a slab on grade structure, whose exterior covered walls are built with concrete masonry units (CMU), and interior walls and ceilings were constructed of wood studs and gypsum drywall (Kosnik, 2009). Context (top) and details (bottom) of the instrument installations are shown in **Figure 4** with those for the Indiana house on the left, Florida house on the right. The living room walls in the Indiana house contain the instrumented dry wall joints as shown in the drawing and center photograph. Horizontal and vertical un-cracked dry wall joints are C9 and C10. Un-cracked locations near the centers of the drywall sheets are C2 and C6. Drywall joint crack, C7, shown in the bottom right most photograph, is at the doorway (adjacent to C6) between the living room and the kitchen. This crack is not fully extended, and did not extend during the observation period. Out-of-plane, midwall motions were measured with velocity transducers as shown in the bottom left photograph.

Similar information for the instrumented garage ceiling drywall joint is shown on the right of **Figure 4**. Sensor D1 spans the joint and D2 is nearby on the full section drywall. They are installed on the attic, upper, or uninhabited side of the garage ceiling as can be seen in the center photograph. As with the wall measurements, out-of-plane ceiling responses were sheetmeasured with a velocity gauge as shown in the middle photograph.

Both structures are located near surface mines (Indiana: coal and Florida: limestone), which require blasting. A typical blast, 2000 feet from the Indiana house, involved 54, 100 ft deep holes arranged in six rows (in a direction radial to the house). Each hole was loaded with 675 lbs of explosive with four decks and thus ~170 lbs of explosive per delay. Such a shot would produce ground motions with a peak particle velocity of 0.14 ips and a dominant frequency of 9 Hz. The Florida house is located some 3000 to 5000 ft from 30 to 50 hole shots loaded with 50 to 60 lbs of explosive. These detonations produce ground motions with peak particle velocities of some 0.05 to 0.18 ips with dominant frequencies between 5 and 33 Hz.

Comparison of Climatological and Vibratory Responses

Figure 5 compares four months of responses of the 3 un-cracked (C9,C10 & D1) and one cracked (C7) drywall joints, and 3 un-cracked drywall sheets (C2,C6 & D2) to temperature and humidity-induced, climatological effects. Indiana information [C] is on the left and Florida information [D] is on the right. Variation in temperature and humidity inside and out is presented on the bottom. Joint, crack and sheet responses are plotted to the same scale at the top for comparison.

Responses of the drywall sheets (C2,C6) are small, and positions such as these are regularly used as the null response. The null response describes the response of the sensor metal and un-cracked mounting material to changes in temperature and humidity. Comparison to the crack response (C7) shows that dry wall sheet response is so small as to be inconsequential compared to the crack response.

Responses to long-term climatological effects of the un-cracked, paper-thin (and thus weak) drywall joints (C9, C10) at the Indiana house are less than $1/10^{\text{th}}$ that of the cracked drywall joint (C7). The vertical and horizontal un-cracked joints are equally as responsive. The drywall joint in the Florida garage (D1) is some

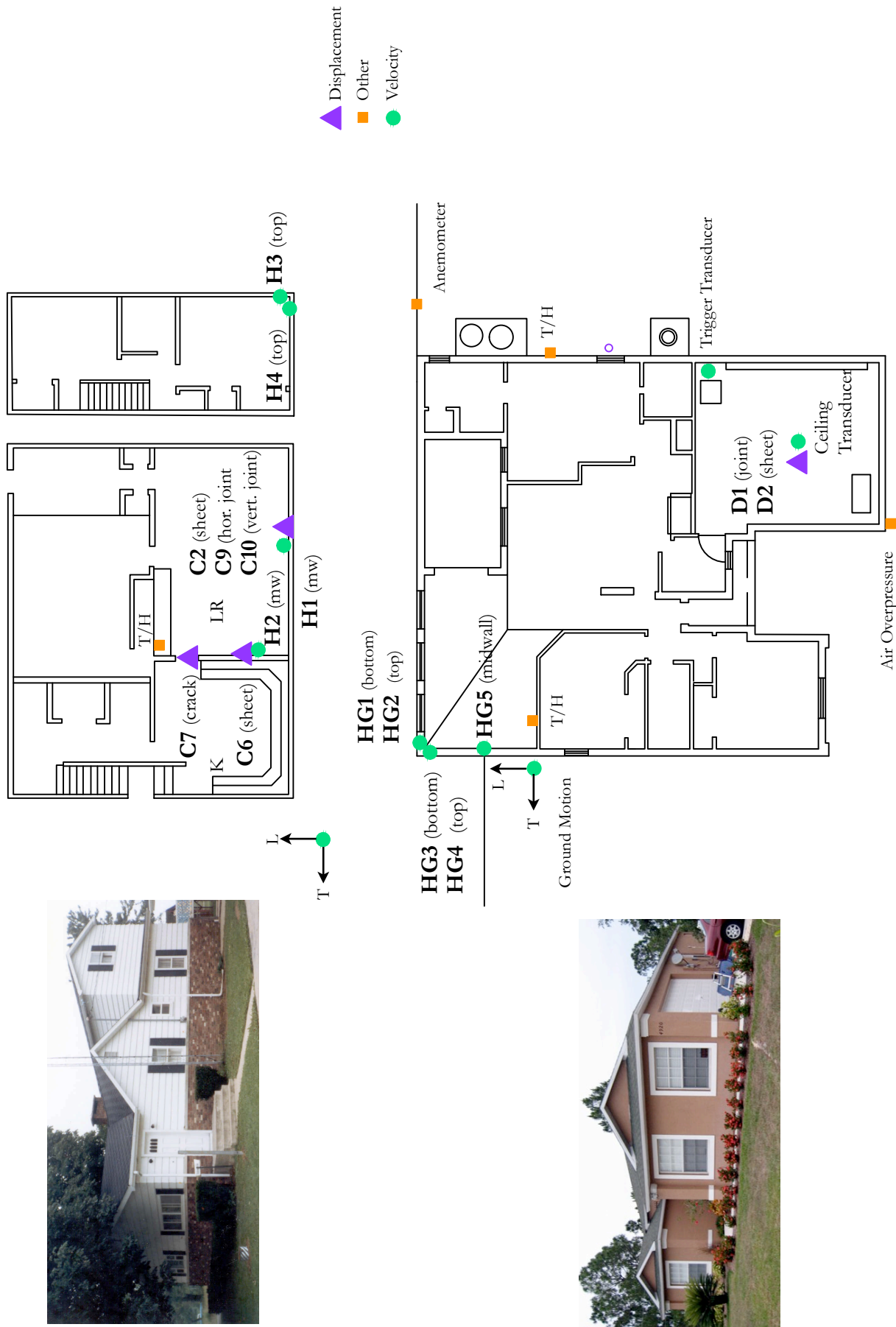


Figure 3 - Photographs and plan views of test structures. Top: Blanford, IN Bottom: Naples, FL

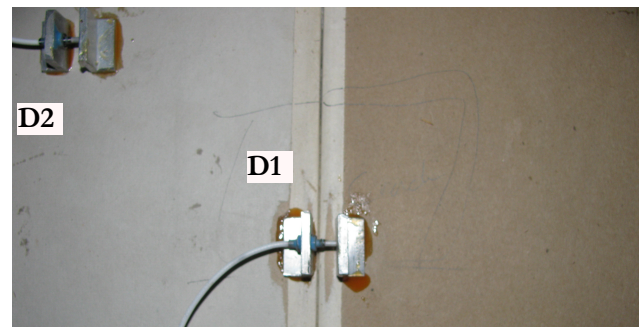
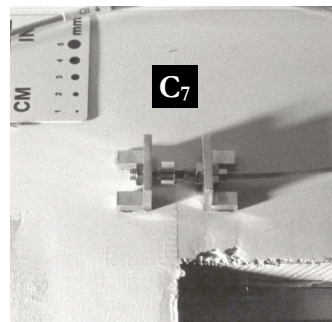
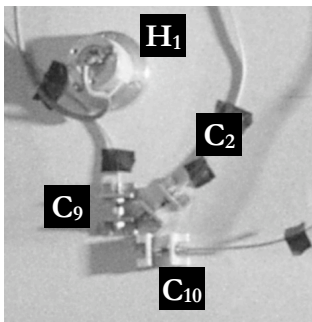
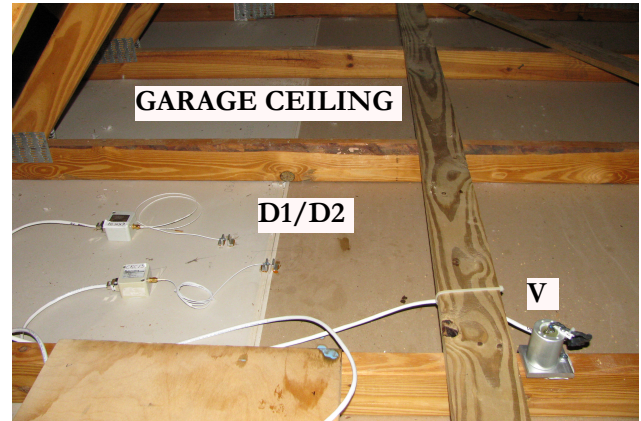
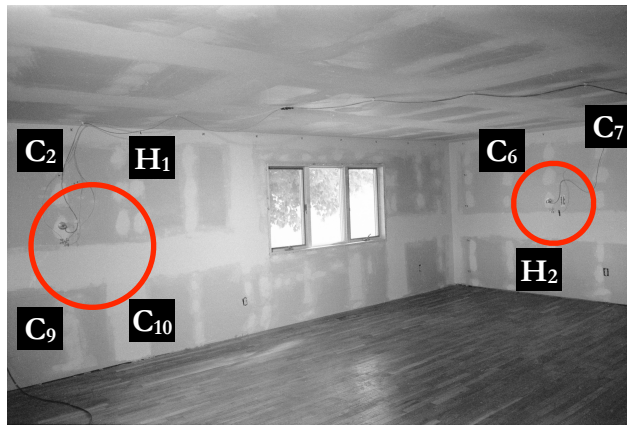
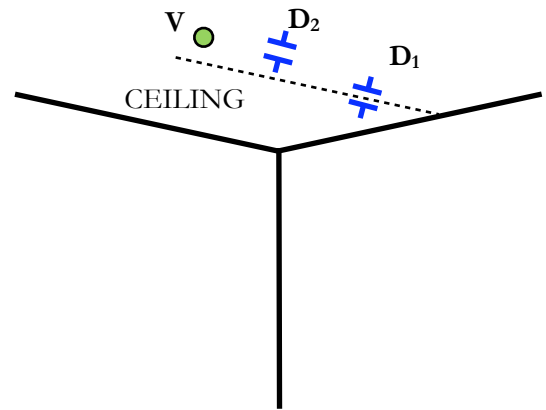
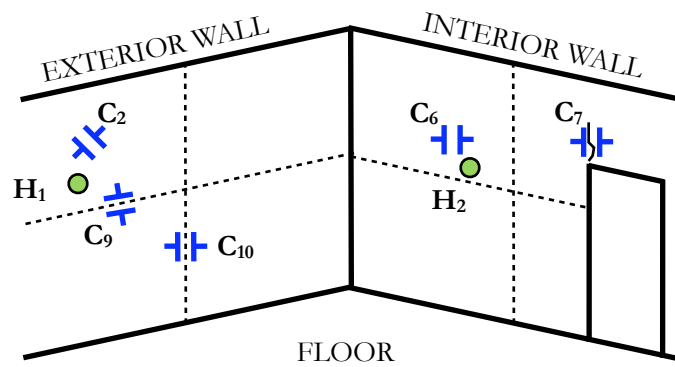


Figure 4 - Installation details for the Indiana (left) and Florida houses (right). Wall, joint and sensor orientation are illustrated on the top row. Photographs showing context are in the middle row and with detail on the bottom. C9&10&D1 cross un-cracked drywall joints; C7 crosses a cracked drywall joint; and C2&6&D2 are located on drywall sheets.

five times more responsive to climatological effects than are the Indiana joints. This large response is not totally unexpected as the joint is in the ceiling of an un-moderated garage during the summer in Florida. Indiana joints were on an interior of a house heated at a constant temperature during the late winter and early spring. Even though larger than that in Indiana, the Florida joint response was small compared to crack response in the garage. A crack in the garage wall at the interface between the doorframe and the CMU wall

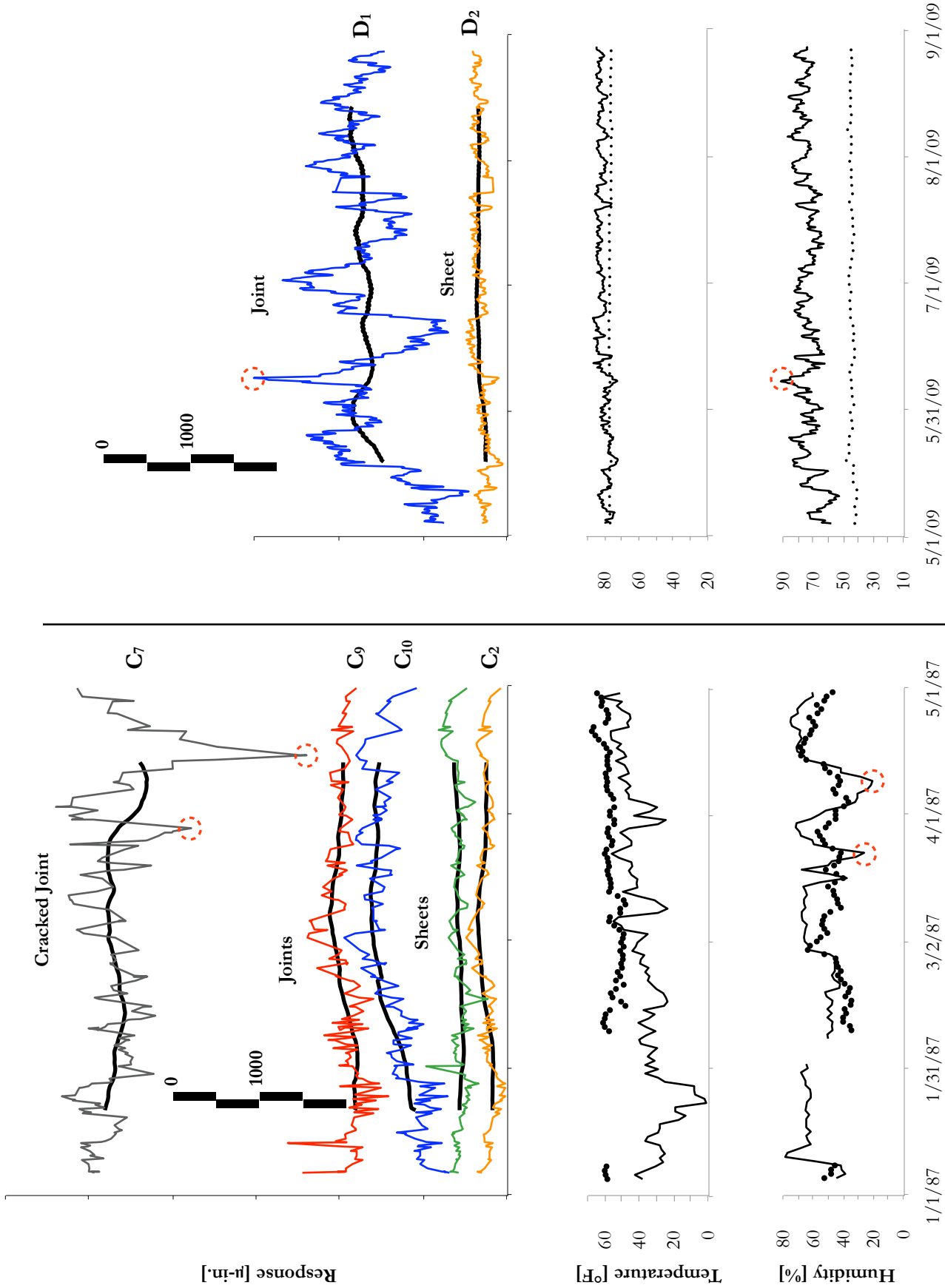


Figure 5 - Comparison of four months of climatologically induced responses of Indiana (left) and Florida (right) joints. 30-day central moving average shown with the thick line. Temperature and Humidity are plotted on the bottom (dotted=inside, solid=outside), and joint responses are plotted on the top with common time and response scales for comparison.

was five times more responsive than the un-cracked Indiana drywall joints (C9&C10) (Meissner et al, 2010). In both cases, significant changes in exterior humidity, marked with circles, seem to drive the largest long-term crack response. It is reasonable for changes in humidity to produce crack and joint response because of the response to changes in humidity of wooden wall frames to which the sheets are attached.

These long-term measurements, spanning some four months, show that un-cracked weaknesses in wall covering are less responsive to long term, climatological effects than other cracked locations. The same is true for vibratory response as shown next.

Vibratory response time histories of un-cracked and cracked dry wall joints for these two houses are shown in **Figure 7**. As before Indiana responses are on the left and Florida's are on the right. Particle velocity time histories of the ground motions that induce the responses are shown at the top and the joint responses are shown at the bottom. The vertical Indiana drywall joint (C10) responds the most – of all uncracked dry wall joints -- and is far more responsive than the horizontal joint. However, its response is still smaller than that for the cracked joint (C7). Response of the Florida drywall joint (D1) to ground motions is small and barely out of the noise level (see **Appendix A** for thunder response). The low frequency ground motions at the Indiana house are evident. Their significance will be discussed in the next section.

The relationship between vibratory and climatological response for un-cracked wall weakness (dry wall joints) is the same as for cracks as shown by the bar chart comparisons in **Figure 6**. Where climatological response is small, so is vibratory response for both cracked and un-cracked joints. Cracking of a joint does not appear to diminish its dynamic response; at least not relative to other un-cracked weaknesses such as the joints. Cracked joints are seen to respond more than un-cracked joints to both vibratory and climatological drivers.

Large response of cracks is not unexpected. The cracking of wall covering provided by the drywall and its weakest element, the paper thin joints, can often be a function of the structural deformation beneath “the wall cover.” Deformation of the underlying structural interface or element is unlikely to be affected significantly by a thin covering. Comparison of the vibration response of C7 to that of H3 and H4, structural velocity response at the second story (shown in **Figure 8**), shows an almost harmonic congruence of the crack response and structural motion. The mass and stiffness of the lower story walls responding to the second story motion will be affected little by the appearance of a hairline crack in a piece of paper spanning a drywall joint.

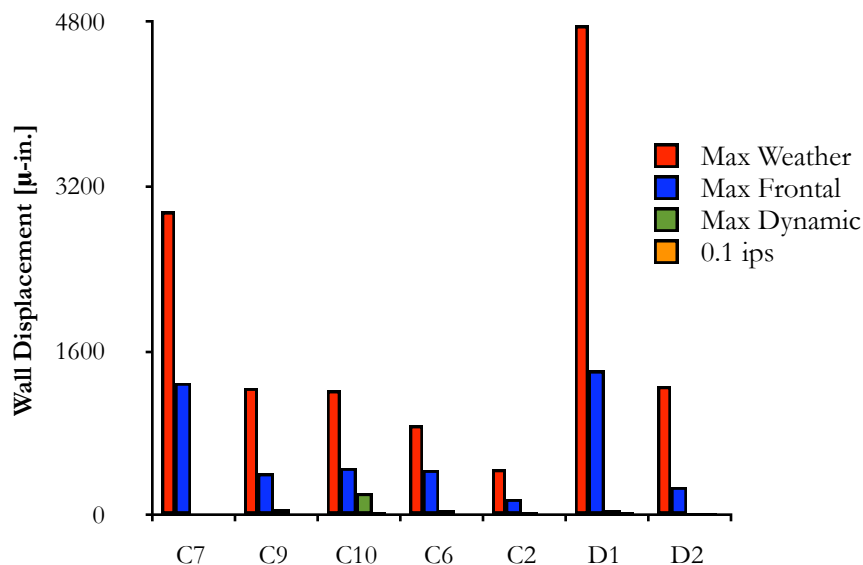


Figure 6 - Bar chart comparison of crack/joint/sheet response induced by weather and blast events. Weather response is at least an order of magnitude greater than dynamic response.

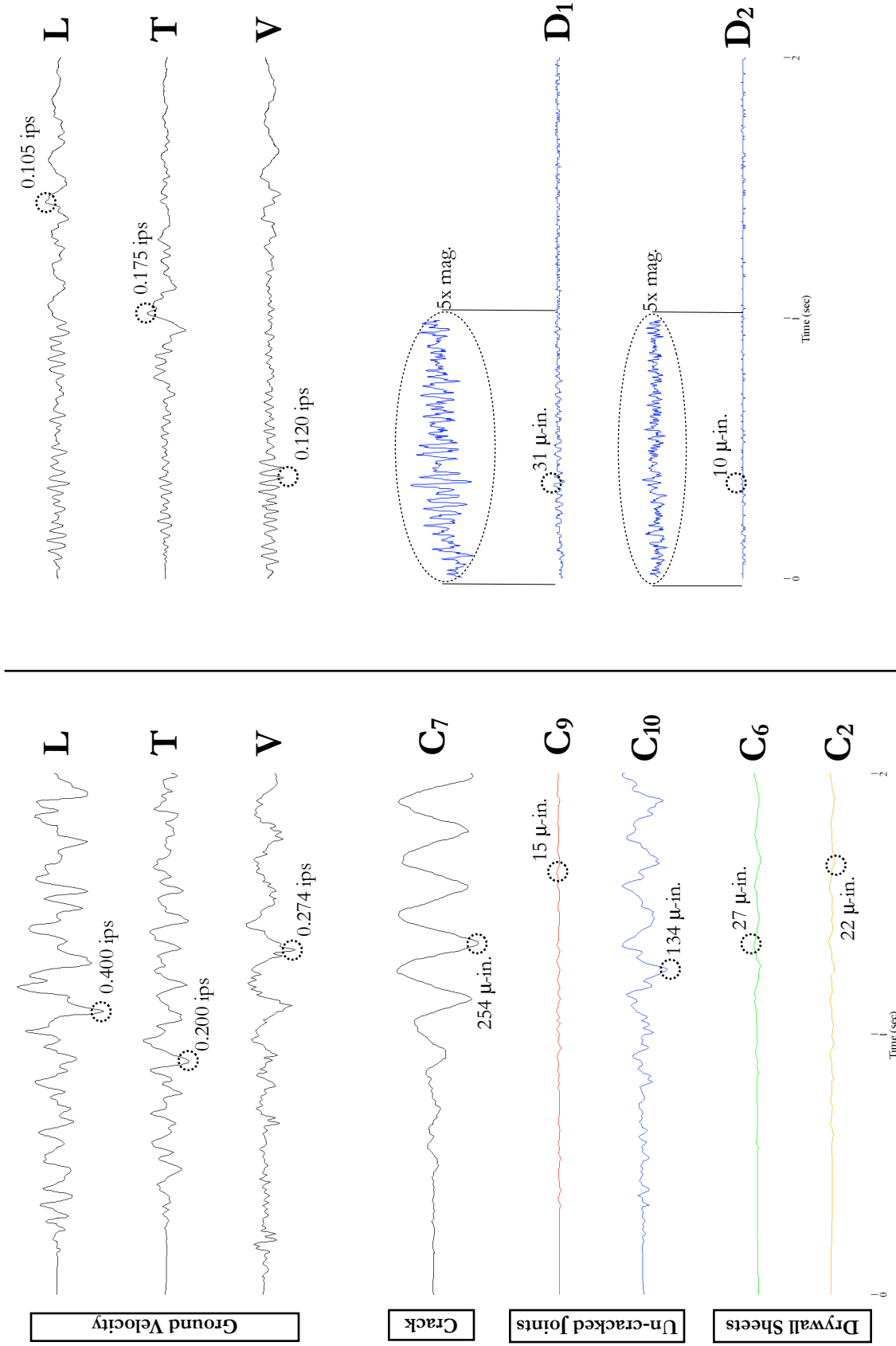


Figure 7 - Comparison of ground motions (top) with joint responses (bottom) showing unusually low excitation frequency of the Indiana ground motions (left) compared to Florida (right). Cracked joint responds more than most sensitive un-cracked joint.

2. LOW FREQUENCY, HIGH AMPLITUDE EXCITATION

As shown in **Table 1**, a number of the blast events produced low frequency, high amplitude ground motions at the Indiana house. **Table 1** compares ground motions, structural response and cracked (C7) and un-cracked (C9,C10) responses for some of the highest amplitude events. As seen in the table, six of the shots produced ground motions in the 5 to 7 Hz range that either coincide with or nearly match the 5 Hz natural frequency of the superstructure demonstrated by the 5 Hz responses of H3 and H4 velocity transducers in the second story as shown in **Figures 7&8**. These data are unique because they combine measurements of both structural and crack response for a case with unusually high amplitude, low frequency ground motions. These low frequency motions normally arrive later in the wave train and are thus likely to be surface waves. The earlier arriving waves are the higher frequency body waves as described in earlier presentation of these data (Dowding, 1996).

No new cracks or extensions were observed as described in the original project report. Information for the Indiana house has been exhumed from 25 year old project files for this paper. In addition to the extensive instrumentation, the house was thoroughly inspected for cracking before and after each blast. The house was divided into inspection grids, which were visually inspected by the same person in the same fashion in each instance. The project report has been scanned for archival purposes and is available for public inspection (Dowding and Lucole, 1988).

Table 1 allows confirmation of several important issues regarding frequency, amplitude and amplification. Amplification values in **Table 1** were calculated in two ways: **1)** the OSM Method (Aimone-Martin, et al 2002): as the ratio of the maximum structural velocity divided by the amplitude of the immediately preceding largest particle velocity excitation pulse preceding and **2)** by the response spectrum or Structural Dynamics method, which employs the entire wave train of the excitation pulse (see Appendix B for a detailed explanation).

Figure 8 presents time histories of ground motion, first-story wall out-of-plane (H1 & H2) and top second-story superstructure (H3 & H4) velocity responses. These wall and superstructure motions are compared with un-cracked (C10) and cracked (C7) joint responses for shots 10 and 12 that demonstrate some of the following observations. First, amplification values from low peak particle velocity motions (PPV's) cannot be assumed to be applicable for high PPV's. Second, both of the horizontal components must be considered.

Figure 9 graphically compares responses of the 5 drywall joint locations with the maximum PPV in the direction parallel to the wall of interest. These plots have more data points than **Table 1**, because only 16 events had recorded time histories from which the table was developed. The other responses are tabulated in the 1988 Dowding & Lucole report. They are remarkably consistent and show the same trends that were measured in previous crack-structural response studies that summarized Office of Surface Mining work (Aimone-Martin et al, 2002). Cracks continue to respond more than do un-cracked weaknesses as can be seen by the comparison of C7 and C10's sensitivity to PPV (greater slope) as also tabulated in **Table 1**. Here the cracked joint sensitivity is approximately 3 times greater than that for the un-cracked joint even for low dominant frequency ground motions.

These comparisons show in **Figure 9** that even for high PPV (10 to 23 mm/s or 0.4 to 0.9 ips) and a mix of low (4 to 8 Hz) and higher frequency (9 to 28 Hz) excitation motions, response of the cracked tape joint (C7) is the same as observed for other vibratory environments. Response of C7 follows a relatively linear trend and the sensitivity is similar to that reported by McKenna & Dowding (2004) where they reported slopes of 50 to 1900 compared with approximately 380 and 630 for the two slopes corresponding to C7. When the lowest frequency (4 to 8 Hz) motions were separated for analysis, the sensitivity of the cracked joint increased slightly. There was no discernible difference in sensitivity of the un-cracked joints between low and higher frequency excitation. The ratio of vibratory response to climatological effects is still small even for low frequency excitation. This ratio is 0.18 for typical weather events and even less for the extreme event in April as shown in the bar charts in **Figure 6**.

The largest crack (C7) response did not occur with the lowest frequency excitation, because the low frequencies were associated with particle velocities below 0.5 ips (12 mm/s). In order to generate higher PPV's, the shots had to be detonated closer to the test house; smaller absolute distances are generally associated with higher PPV's and higher excitation frequencies.

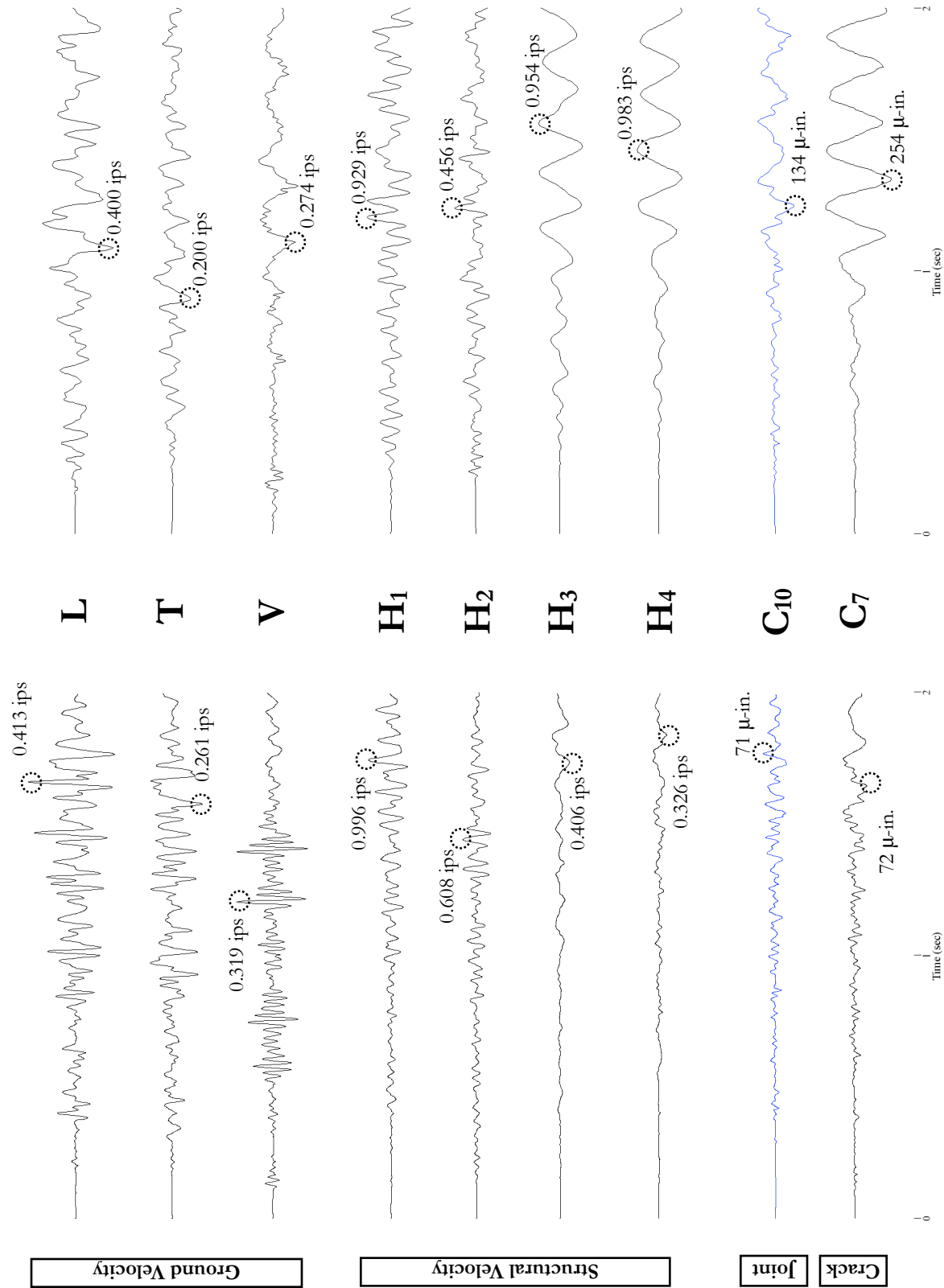


Figure 8 - Time histories of ground motion, structural response, and cracked (C7) and un-cracked drywall joint response (C10). Low frequency excitation show joint response follows the motion of the upper story. 2/23/87 on the left and 4/2/87 on the right.

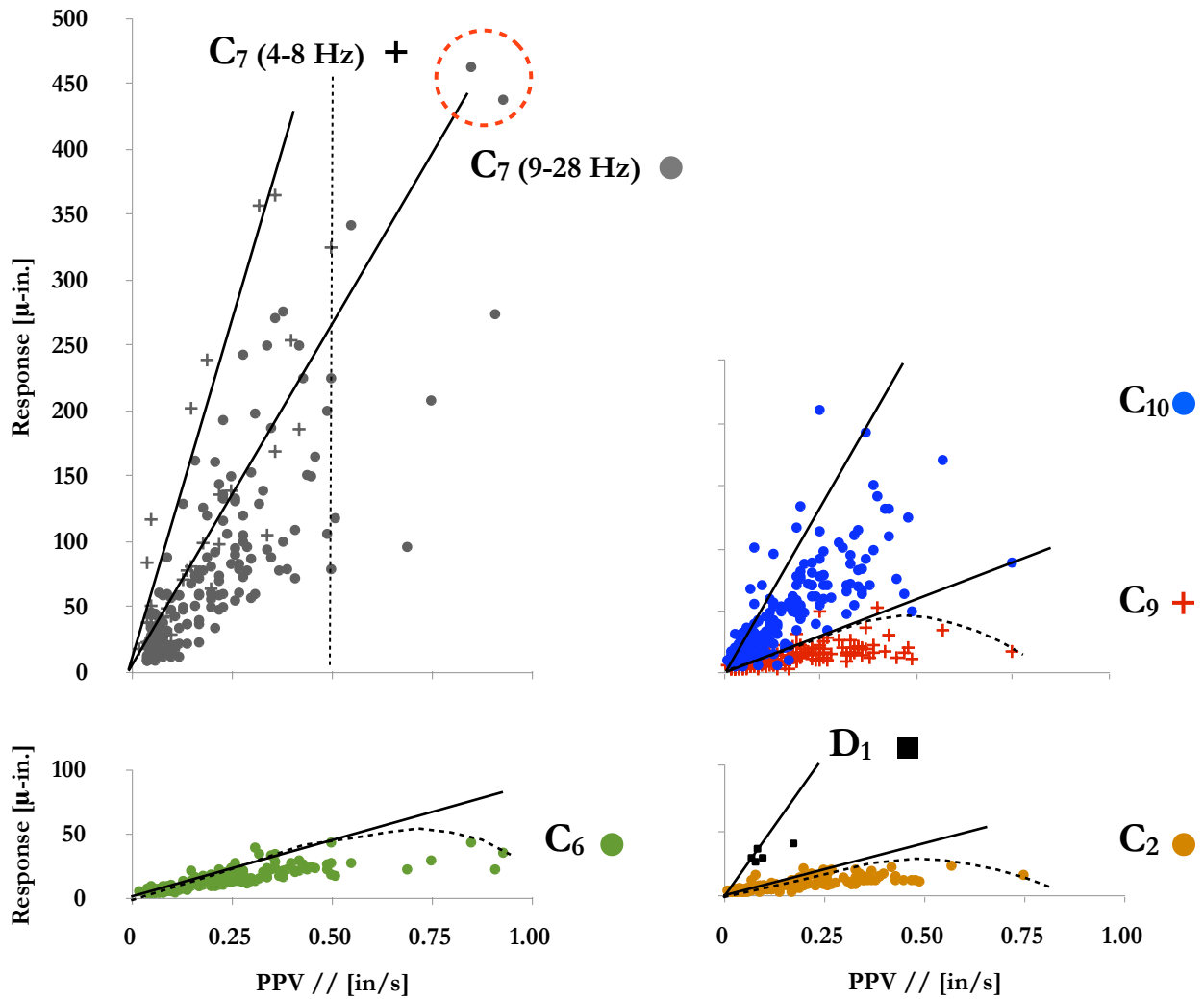


Figure 9 - Comparison of uncracked joint (C10,C9) on the right with crack (C7) response on the left to increasing peak particle velocity in the direction of the wall containing the joint/crack. Crack C7 is the most responsive or sensitive (has the steepest slope) of those instrumented. Sensitivity of drywall sheet (C2,C6) is the smallest as expected. Sensitivity of the Florida uncracked joint (D1) is similar to that of C10 for Indiana.

CONCLUSIONS

Measurements have been made in two structures to investigate several concerns regarding the usefulness of the observation that cracks respond more climatological than vibratory effects. Concerns addressed are: 1) cracking relieves strains and strains concentrate elsewhere, reducing the sensitivity of cracks to excitation relative to un-cracked locations and 2) there are not enough observations of crack response in low excitation frequency - high particle velocity environments that may cause greater amplification. Measurements presented herein show that:

A cracked joint does not respond less than other un-cracked weaknesses in the wall covering to either climatological or vibratory effects.

Even in high particle velocity (10 to 23 mm/s or 0.4 to 0.9 ips) and low excitation frequency (5 to 7 Hz) environments, cracks continue to respond more than do un-cracked weaknesses.

Responses of the weakest of wall components, the paper-thin joints between drywall sheets were measured and shown to be less than that of cracked joints.

ACKNOWLEDGEMENTS

The authors are indebted to all those who have contributed to this synthesis of two separate projects, one which occurred several decades ago. The structures were/are owned by two resource companies, Peabody Coal Co. and Jones Mining without whose cooperation this project would not have been possible. Instruments were installed by Digital Vibration Incorporated and field engineers of the Infrastructure Technology Institute [ITI] of Northwestern University: Dave Kosnik, Mat Kotowsky, and Dan Marron. We are also grateful for the financial support of ITI for this project through its block grant from the U.S. Department of Transportation to develop and deploy new instrumentation to construct and maintain the transportation infrastructure.

REFERENCES

Aimone-Martin, C, Martell, M.-A., McKenna, L.M., Dowding, C.H., Siskind, D.E., and Dowding, C.H. (2002) Comparative study of structure response to coal mine blasting, Prepared for the Office of Surface Mining Reclamation and Enforcement, Appalachian Regional Coordinating Center, Pittsburgh, PA (<http://hpa.osmre.gov/arblast/downloads/StructureResponse.pdf>)

Dowding, C.H. (1996) Construction Vibrations, Prentice Hall. Now available through ISEE and Amazon.com

Dowding, C.H. (2008) Micrometer Crack Response to Vibration and Weather, International Society of Explosives Engineers, Cleveland, OH, USA, 409 pgs.

Dowding, C.H. and Lucole, S.W. (1988) Comparison of Environmental and Blast Response: Test House, Blanford Indiana. Reproduced on www.iti.northwestern.edu/acm.

Kosnik, D. (2009) "Autonomous Crack Displacement Monitoring of a Residence Near a Quarry. Proceedings of the 35rd Conference on Explosives and Blasting Technique, International Society of Explosive Engineers, Cleveland, OH, February.

Meissner, J.E, Waldron, M. J. and Dowding, C.H. (2010) "Comparison of In-Plane and Out of Plane Response of Cracks to Blast Vibration and Weather," Proceedings of the 36th Conference on Explosives and Blasting Technique, International Society of Explosive Engineers, Cleveland, OH, February.

McKenna, L.M.(2002) Comparison of Measured Crack Response in Diverse Structures to Dynamic Events and Weather Phenomena, M.S Thesis, Department of Civil & Environmental Engineering, Northwestern University, Evanston, IL.

Miller, R. (1991) Analysis of Fracture Process in Mortar using Laser Holographic Interferometry, Ph.D. Thesis, Department of Civil and Environmental Engineering, Northwestern University, Evanston, IL

APPENDIX A - Joint Response to Thunder

The un-cracked joint and sheet also respond to events like lightning strikes that produce an air overpressure pulse that is normally experienced as window rattle. Nearby events, say within 1 km, produce the largest air overpressures. In Figure A-1 below, the drywall joint responds 282 μ -in, much more than during any blast (31 μ -in). Also, note that the air overpressure from the thunder clap (0.01 psi) is ten times greater than any blast in the study period.

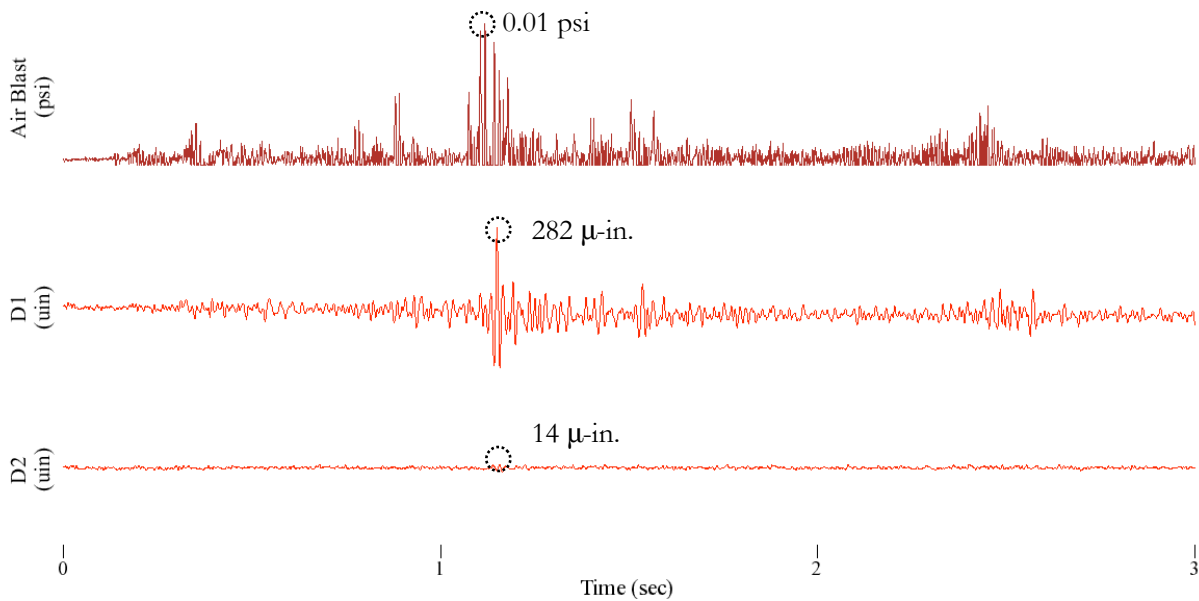
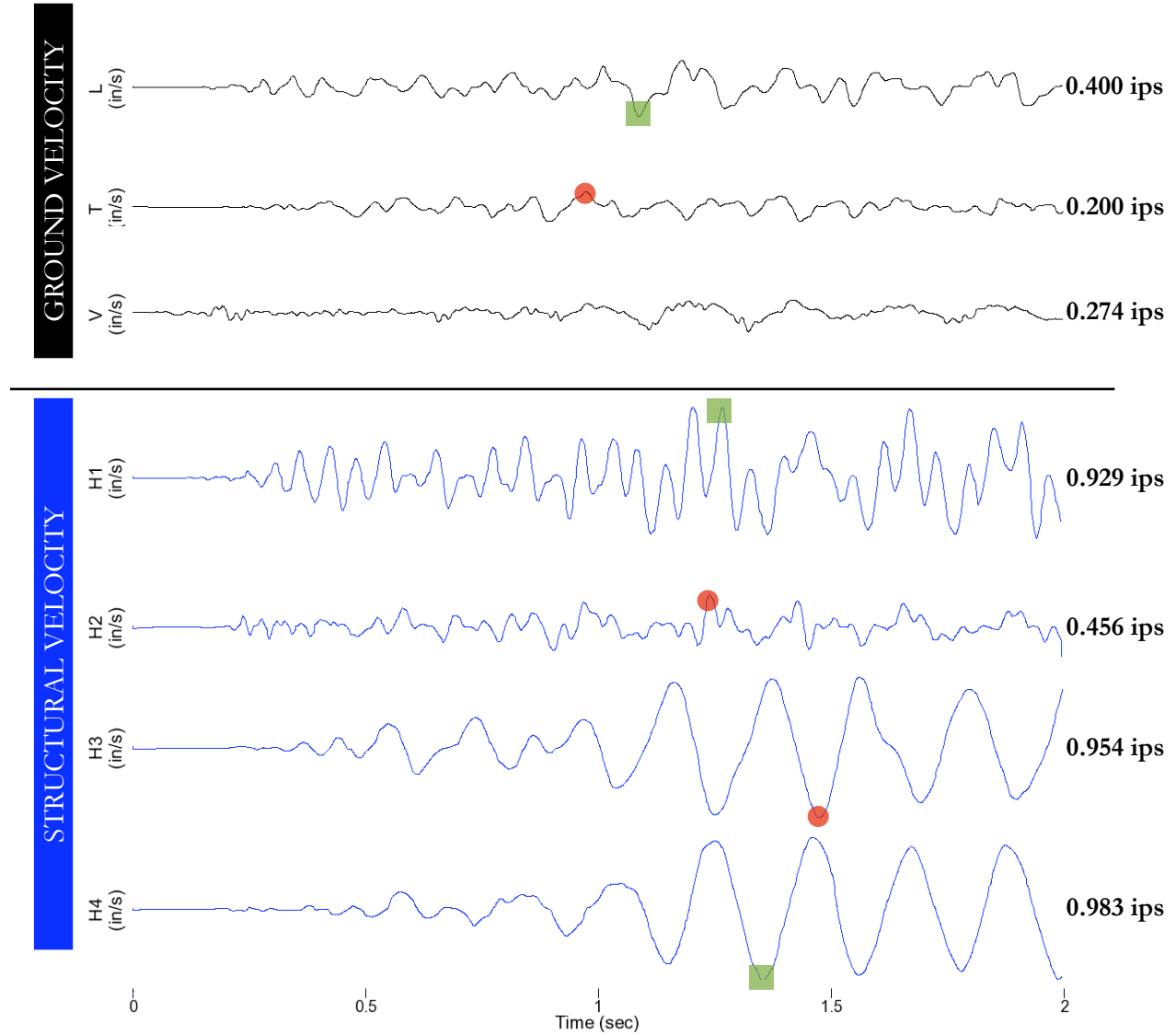


Figure A-1 - Response time histories during a thunder event on September 5th, 2009. The cracks respond directly to the very large air overpressure.

APPENDIX B - Amplification Calculation

1) OSM Tradition Method



$$Amplification_{H1} = \frac{0.929ips}{0.400ips} = 2.32$$

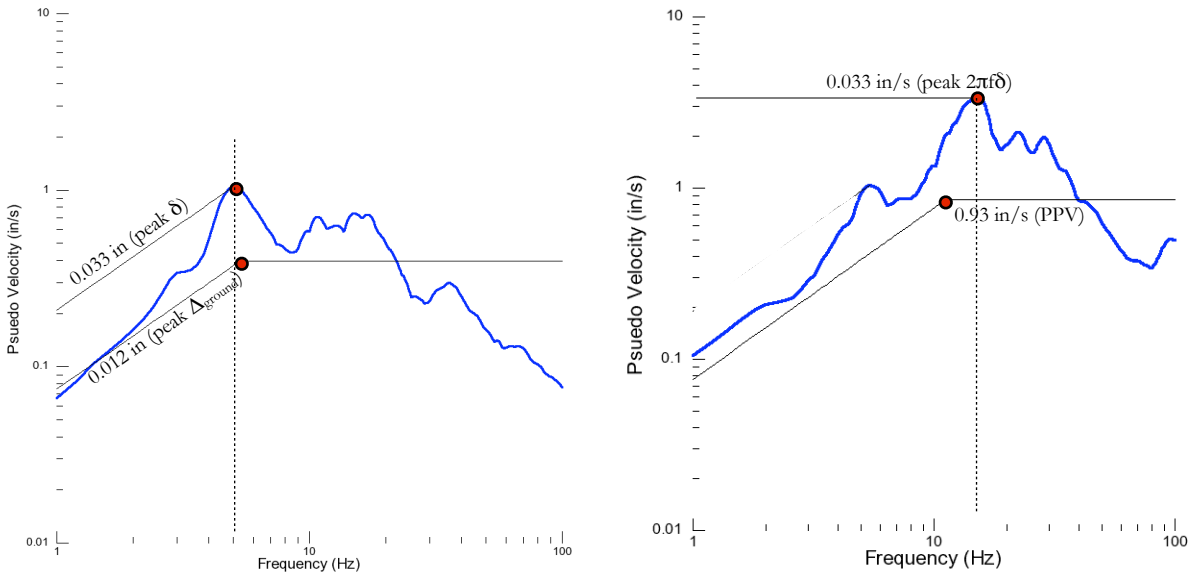
Figure B-1 - Traditional method of calculating Amplification. Transducer velocity is divided by the largest preceding excitation pulse in the same direction. Square peaks correspond to square peaks and circle to circle.

2) Structural Dynamics Method

In structural dynamics, the structure is modeled as single degree-of-freedom (SDOF) system to find the relative displacement, δ . The ratio of the relative displacement (5 Hz peak) in the response spectrum to the peak ground displacement is taken as the amplification factor.

Relative displacement is the measure of importance because it causes strain in the wall, which in turn produces cracking. Amplification is then described by the ratio of the maximum relative displacement divided by the maximum ground displacement if the peak falls on the displacement bound as in case (a) for the 4/2/87 ground motion event. In case (b), the response spectrum peak occurs at a higher frequency above the velocity bound. In this case, the amplification factor is calculated as the ratio of the pseudo velocity ($2\pi f\delta$) and the maximum ground velocity. In **Table 1**, all amplification ratios were calculated as δ/Δ for H3 & H4 because they occurred on the displacement bound or $2\pi f\delta/PPV$ when on the velocity bound (H1 & H2).

The response spectrum approach also incorporates the full ground motion wave train to calculate/estimate amplification (Dowding 1996). Use of the full wave train incorporates the entire energy spectrum of the excitation event. It also eliminates the need to determine which ground motion peak coincides with the maximum structural response in the traditional method.



$$Amp_{H4} = \frac{\delta}{\Delta} = \frac{peak_rel_disp}{peak_gnd_disp} = \frac{0.033in}{0.012in} = 2.81$$

Figure B-2 - Structural Dynamics method of calculating Amplification for the 5Hz natural frequency of the superstructure and 16 Hz natural frequency of the walls. SDOF response spectra are those of the longitudinal direction for 4/2/87 (a) lower frequency event and 4/20/87 (2) (b) higher frequency event.

APPENDIX C - Digitized Time Histories

The following appendix shows digitized time histories for 5 blast events from 1987 in Blanford, Indiana. Each event has 3 pages:

Page 1: Ground Velocities, Structural Velocities

Page 2: Ground Velocities, Crack/Joint/Sheet Displacements

Page 3: Structural Displacements, SDOF Abs. Displacements, Response Spectra, Upper Story Corner Path

The SDOF Response Spectrum model requires two inputs: ground displacement time history and damping ratio. Furthermore, the SDOF Absolute Displacement time history can be calculated with the additional input of natural frequency. The superstructure of a house generally has a lower natural frequency (5-10 Hz) than the walls (10-20 Hz) (McKenna 2002).

Fourier Analysis was used to determine the frequency content of the ground and structural velocity time histories for the five events. Power spectral density functions of absolute structural velocity ($|G(f)|^2$), ground velocity ($|F(f)|^2$), and a transfer function ($|H(f)|^2$) were calculated for both the superstructure and the walls in the longitudinal and transverse directions. The peak of the transfer function indicates the natural frequency of the structure. In this particular structure, the superstructure natural frequencies were determined to be the 4 to 6 Hz range, while the wall natural frequencies were in the 15-17 Hz range.

$|G(f)|$ = output of Fourier Transform Integral for structural velocity

$|F(f)|$ = output of Fourier Transform Integral for ground velocity

$|H(f)|^2$ = Fourier Transfer Function = $|G(f)|^2 / |F(f)|^2$

The structure's critical damping fraction (β) was calculated from the decay of free oscillation for some events (structural oscillation after excitation motion has passed). The formula for the calculation is shown below in equation **C-X** where u_n is the series of successive amplitudes in free vibration. Damping ratios for this superstructure and walls were in the 6-8% range.

$$\beta = \frac{1}{2\pi} \left(-\ln \frac{\dot{u}_{n+1}}{\dot{u}_n} \right)$$

Table C-1 describes the parameters used in the NUVIB2 software for the Single Degree-of-Freedom models. Though the values used below are precise to the 0.1 [%/Hz], the accuracy of the methods above is probably only to the 1 [%/Hz].

Response	Damping Ratio (%)	Natural Frequency (Hz)
Longitudinal (H1) - Wall	6.8	15.5
Transverse (H2) - Wall	6.8	17.0
Transverse (H3) - Structure	6.6	5.0
Longitudinal (H4) - Structure	7.8	5.0

Table C-1 - Damping Ratio (β) and Natural Frequency used in SDOF model in NUVIB2 for 4 responses

It's also important to note that the structural velocities and displacements shown in this appendix are absolute. Because the instrumentation lacked a transducer at the bottom of the structure, only the absolute velocity (displacement) is known (relative to the earth rather than the bottom of the structure). While relative displacement is a good indication of strain in the walls, absolute displacement says nothing because the structure could very well be translating without distorting. Nonetheless, the SDOF Response Spectra yields pseudo-velocity and relative displacements. Therefore, the structural dynamics method of calculating Amplification is perhaps more appropriate because it incorporates a strain-related displacement that is an index for cracking potential.

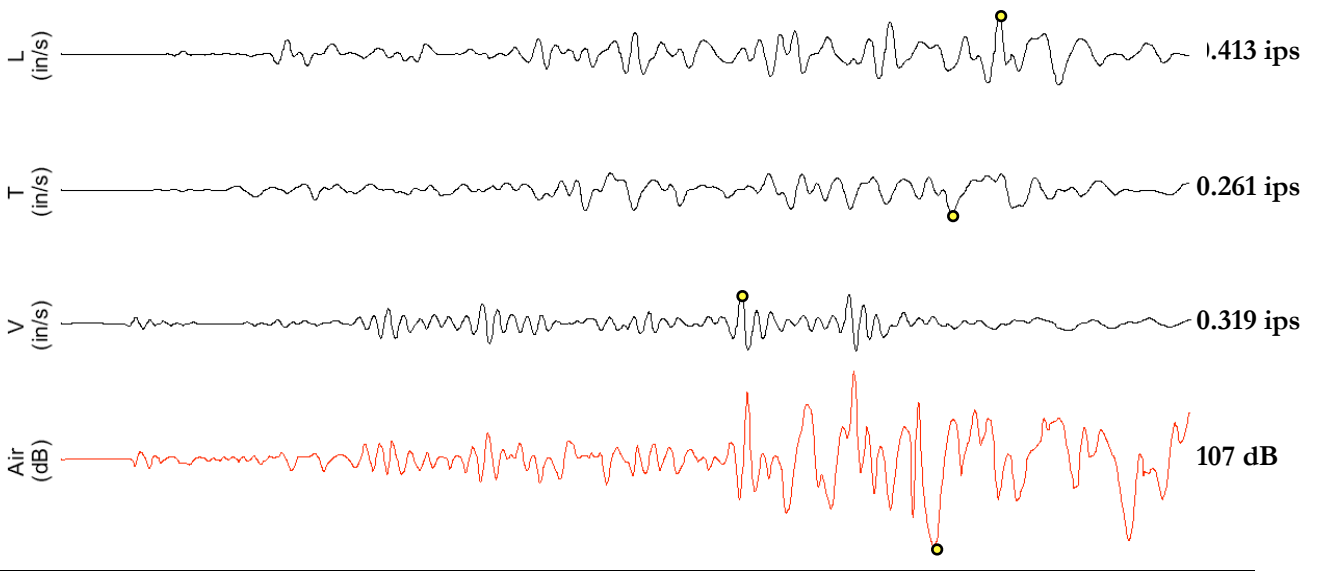
While pages 1 & 2 of the events represent data collected directly from the transducers, page 3 plots are calculated from the raw data:

- a) The top four plots are the absolute structural displacements:
 - (i) blue-solid is the integration of the velocity over time
 - (ii) black-dashed is the absolute displacement calculated from the SDOF modelH1&H4 correspond to the longitudinal ground motion (parallel), while H2&H3 correspond to transverse

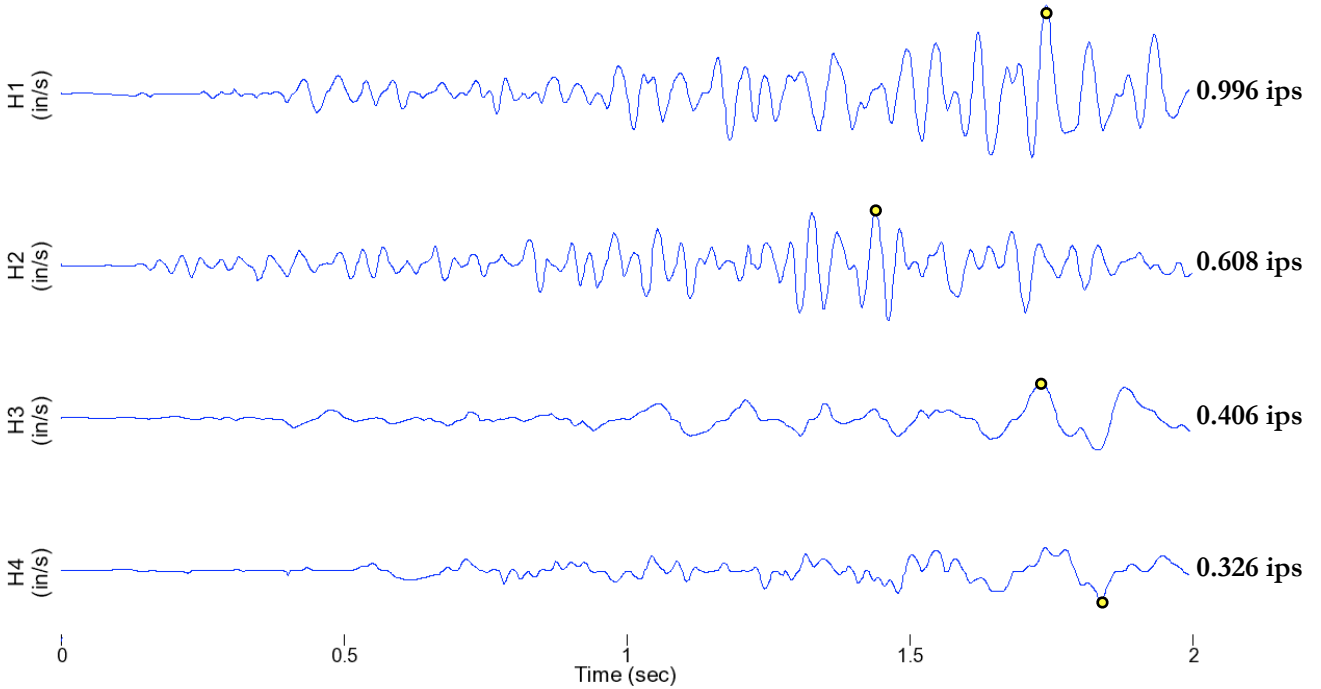
- b) The middle charts are SDOF Response Spectra of Pseudo-Velocity vs. Frequency
 - (i) left is Longitudinal direction ($\beta=6.6\%$)
 - (ii) right is Transverse direction ($\beta=7.8\%$)The thin black inclined line represents the peak ground displacement and the horizontal one represents the PPV

- c) The bottom most plot is the Path of the upper story Corner in plan view (transducers H3 & H4)
The chaotic response is from the beginning of the time history, while the more outer global motion represents the end of the time history.

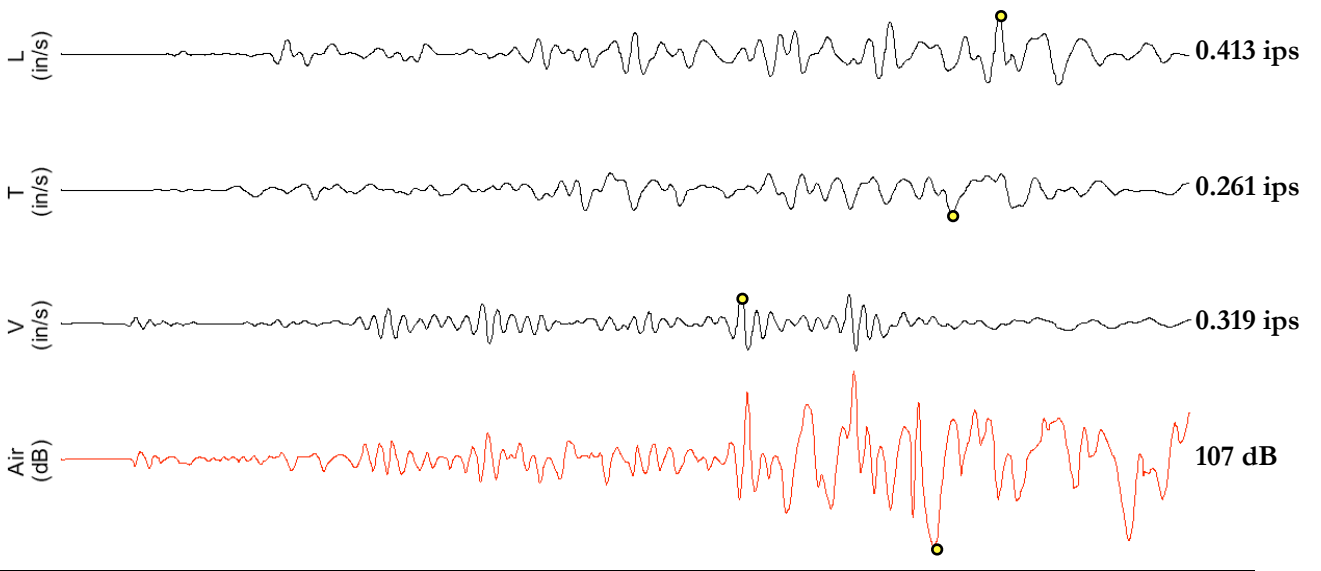
GROUND VELOCITY



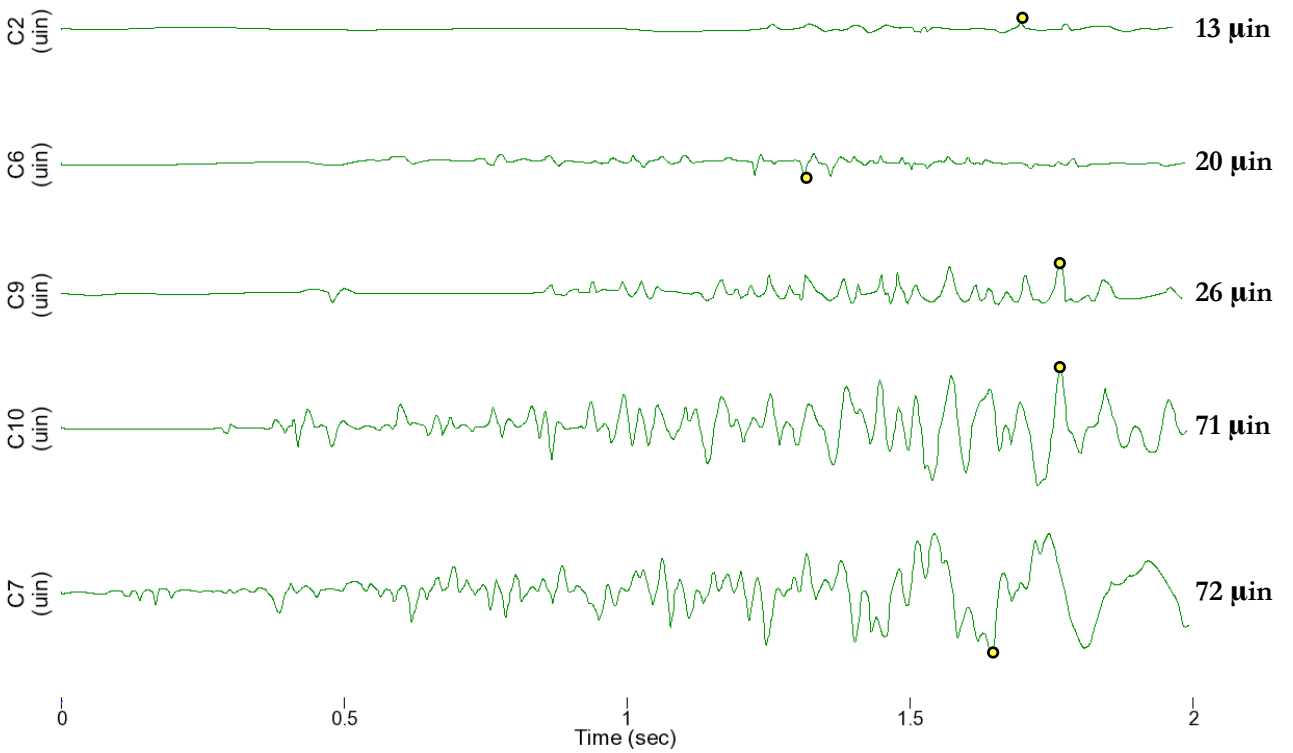
STRUCTURAL VELOCITY



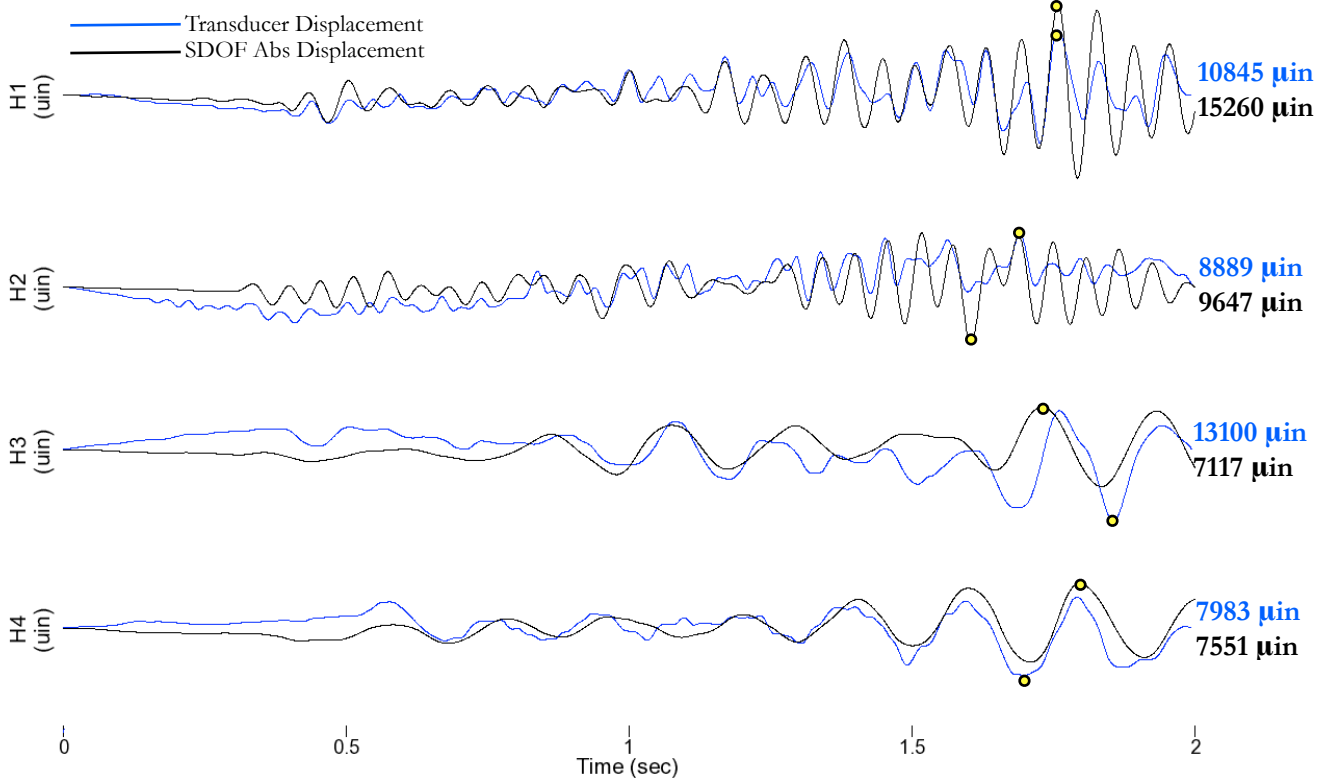
GROUND VELOCITY



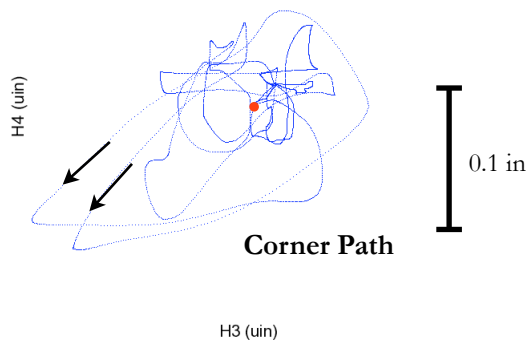
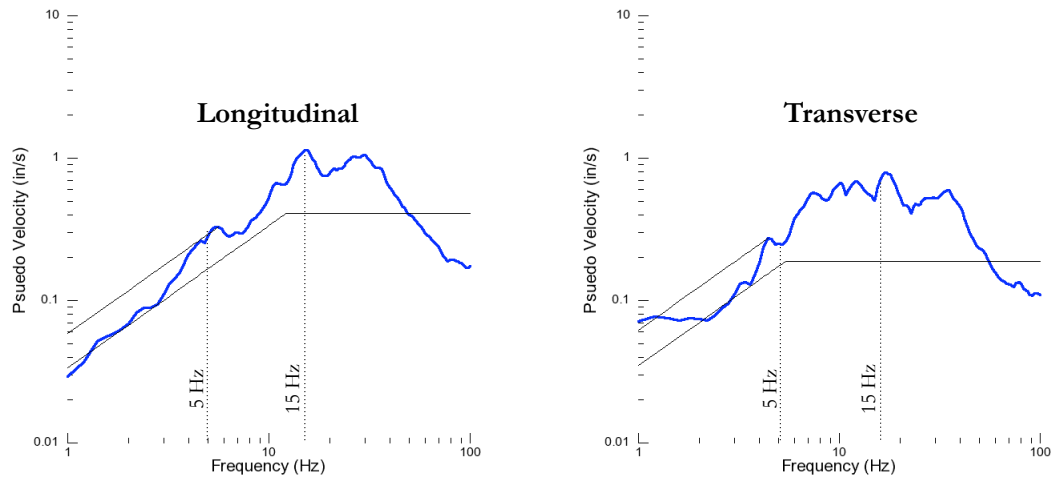
CRACK RESPONSE



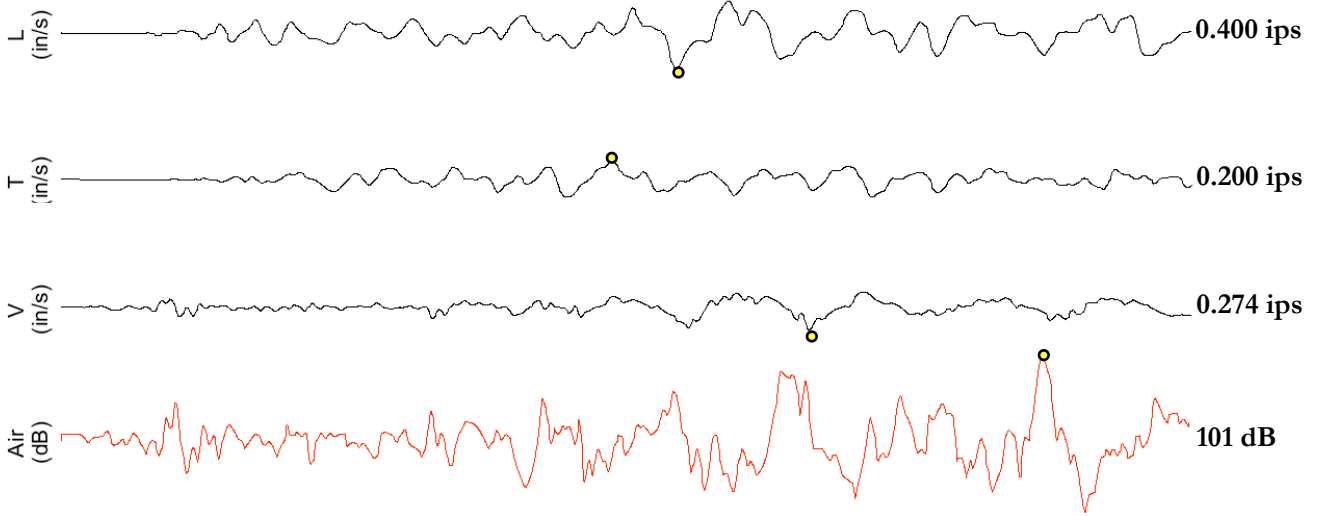
STRUCTURAL DISPLACEMENT (ABSOLUTE)



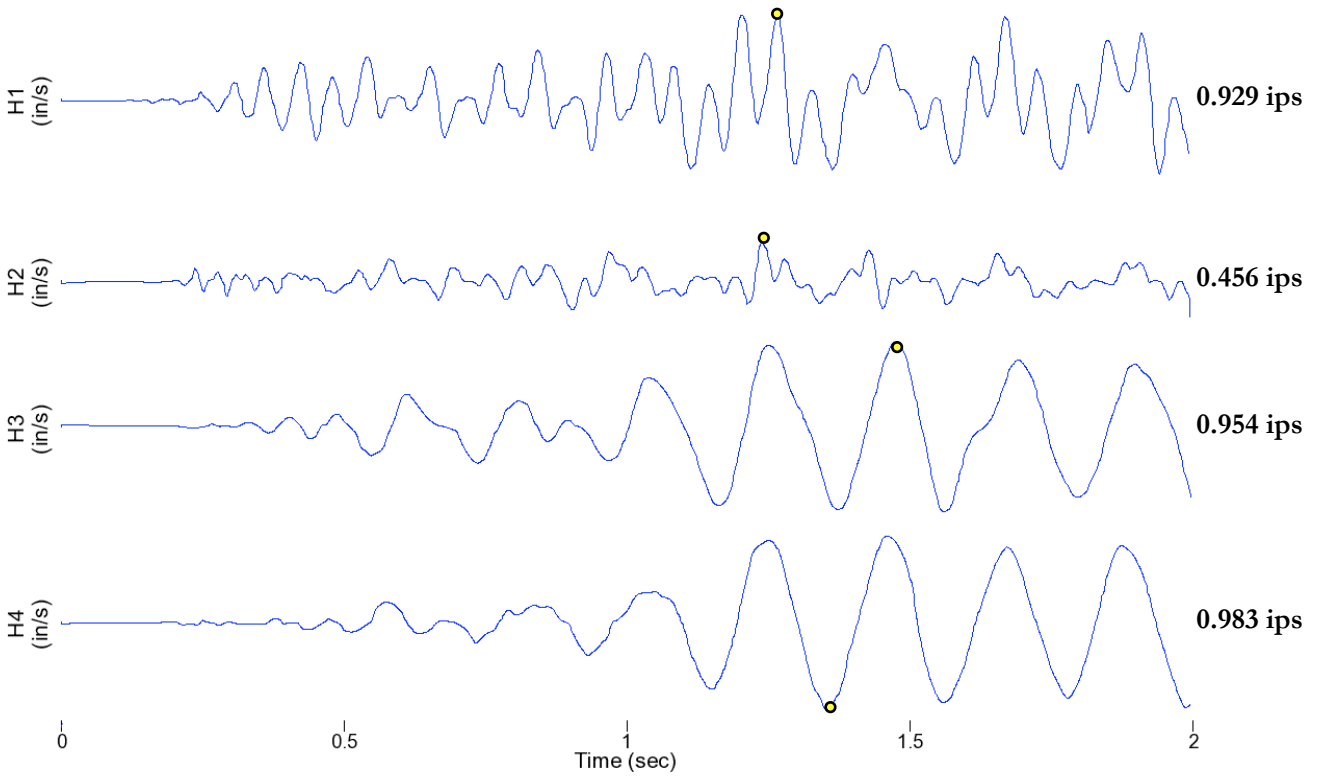
SDOF RESPONSE SPECTRA



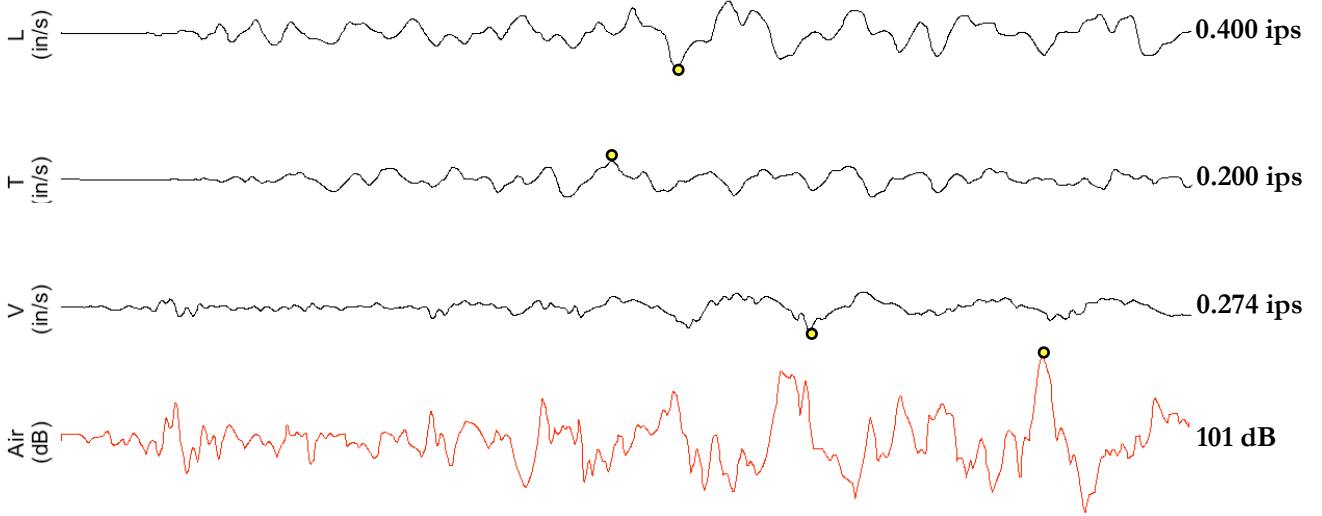
GROUND VELOCITY



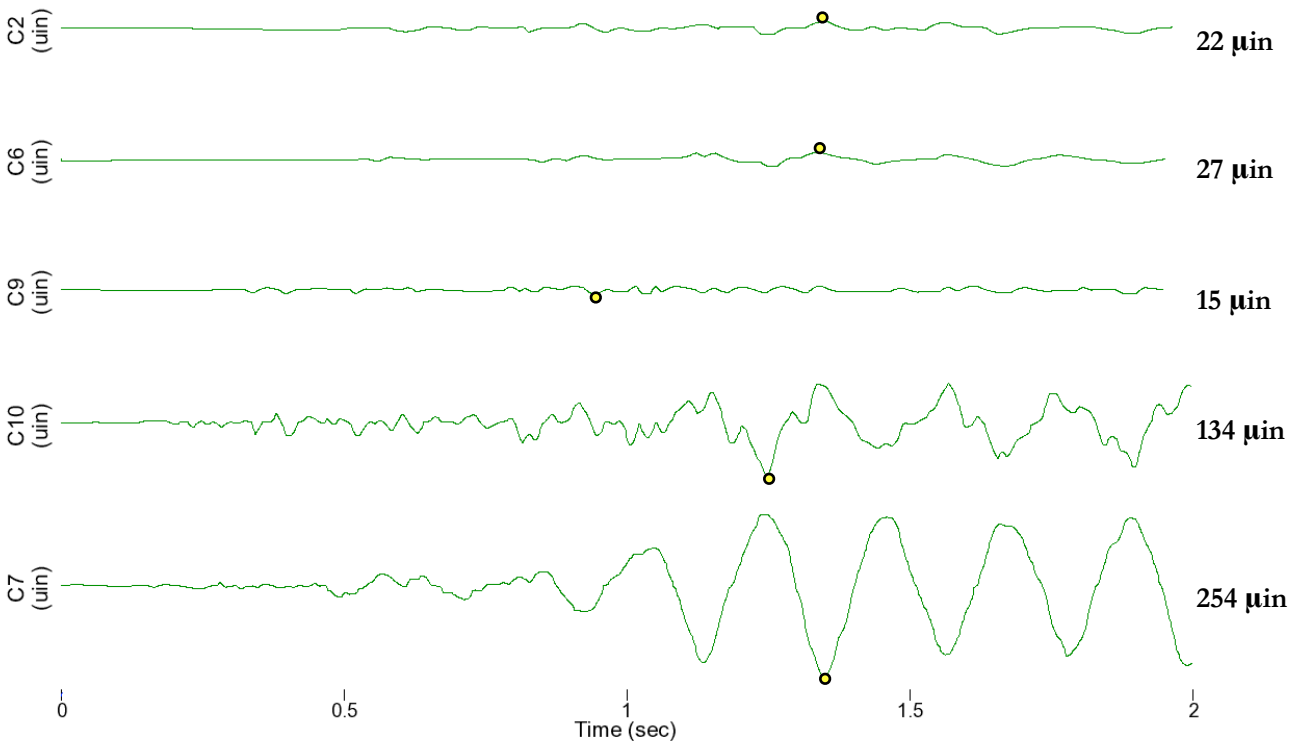
STRUCTURAL VELOCITY



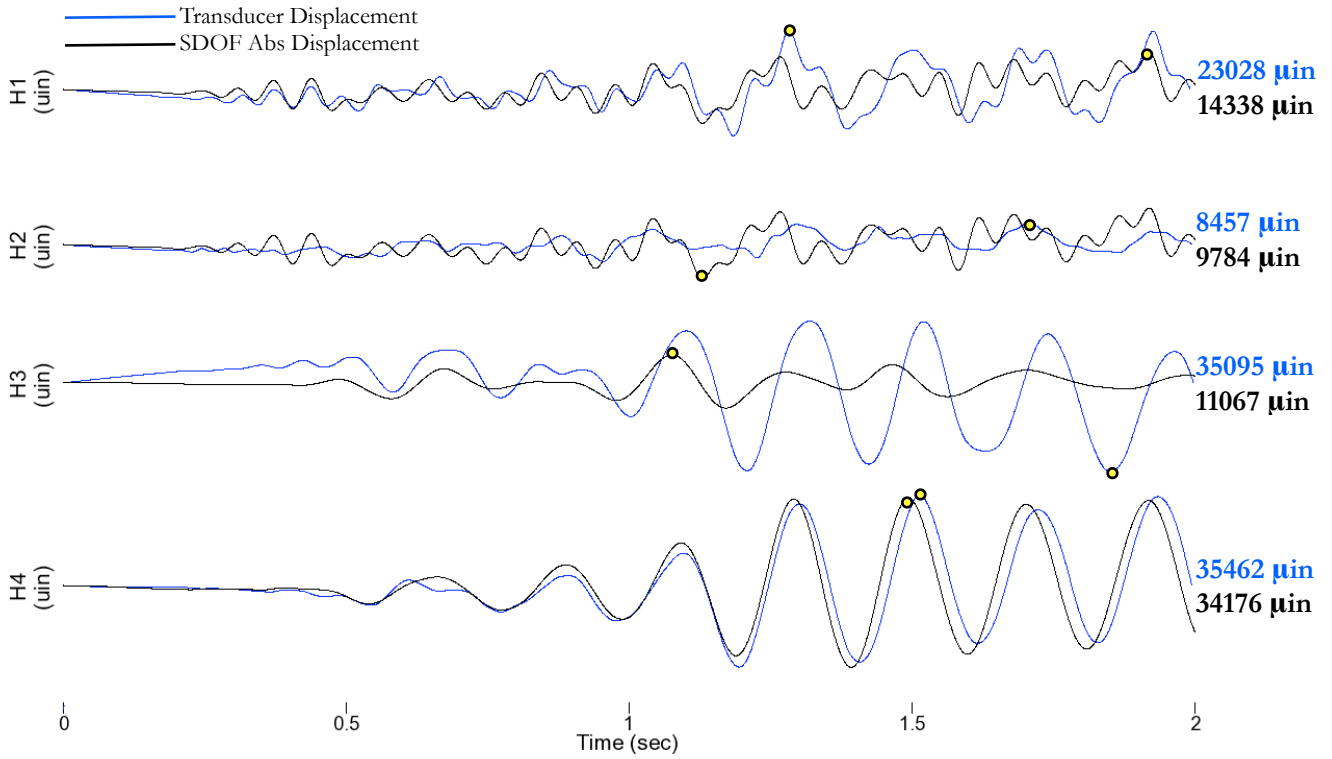
GROUND VELOCITY



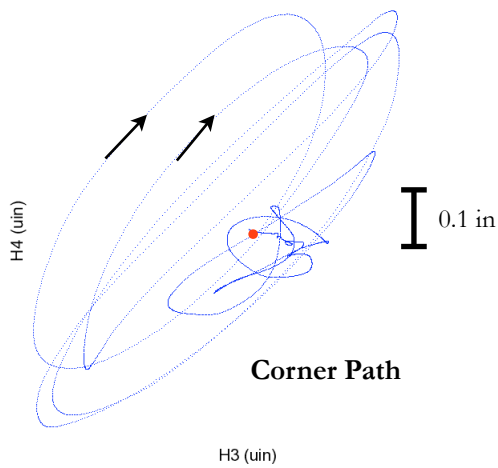
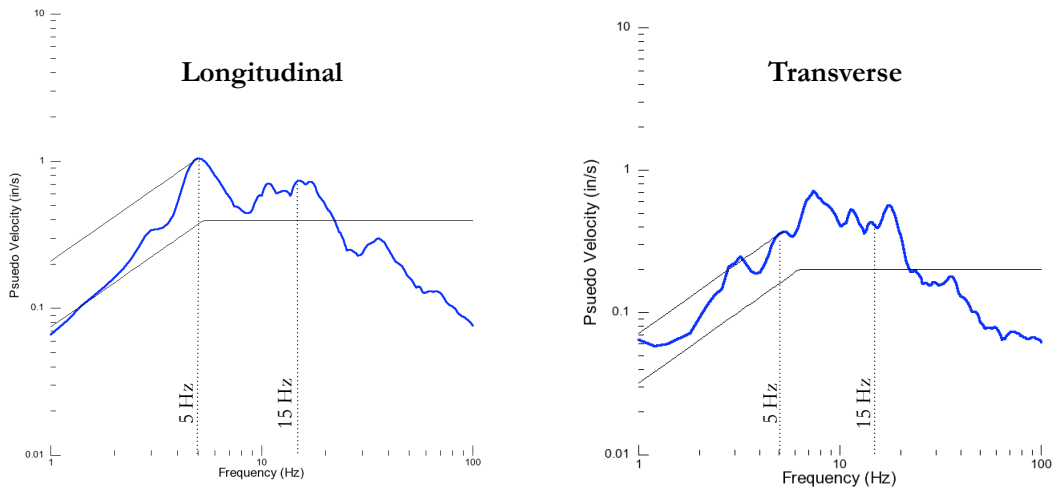
CRACK RESPONSE



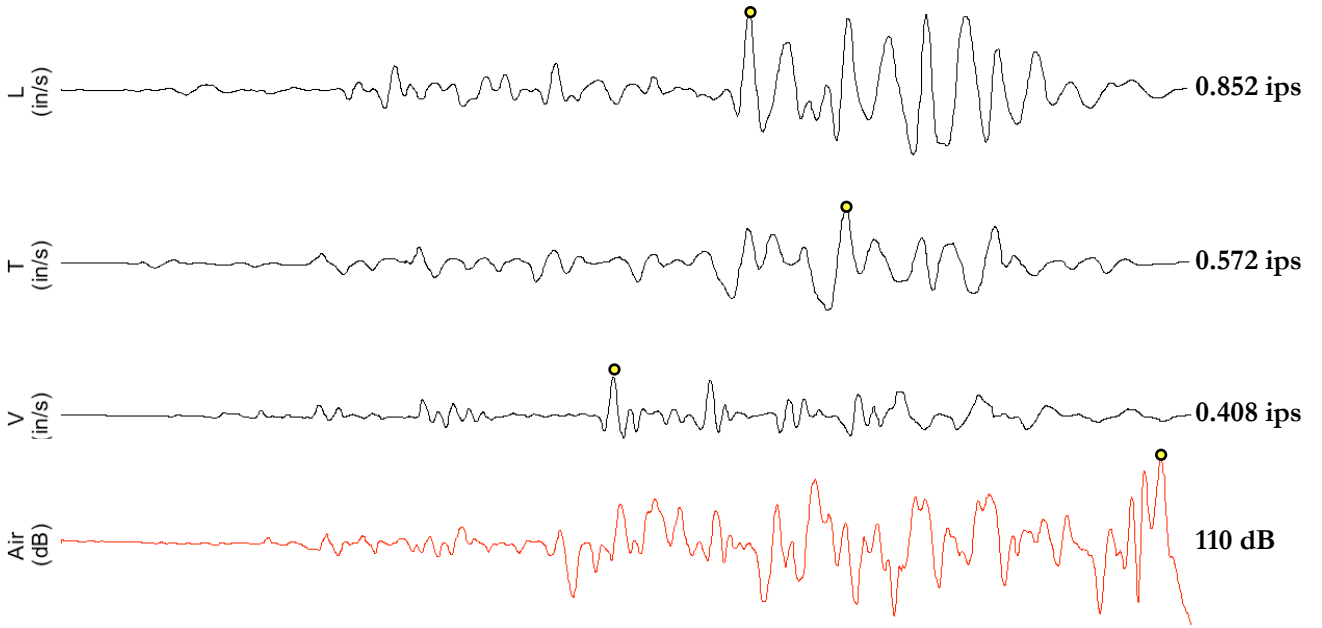
STRUCTURAL DISPLACEMENT (ABSOLUTE)



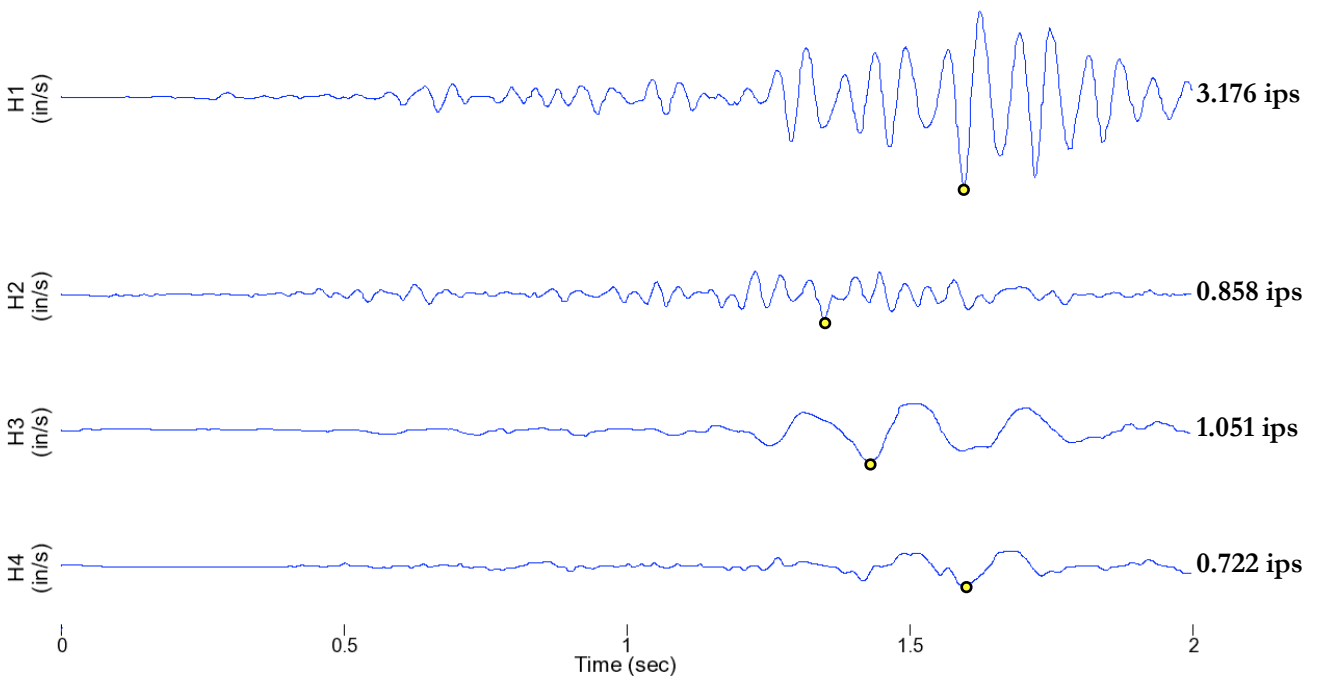
SDOF RESPONSE SPECTRA



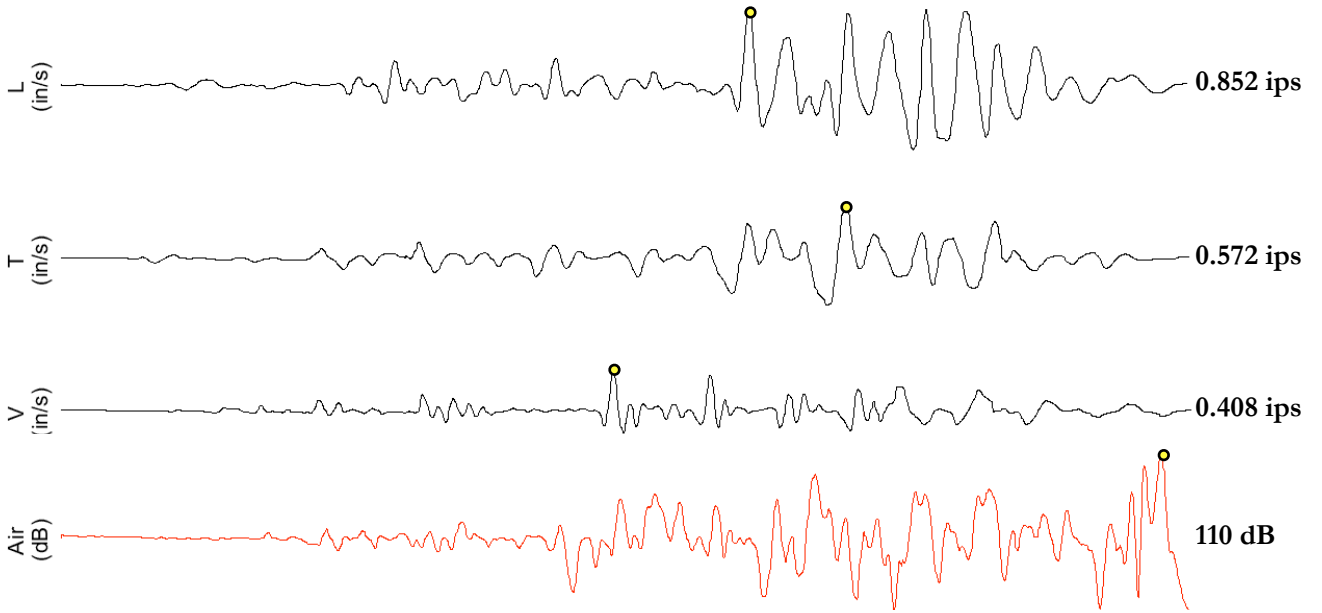
GROUND VELOCITY



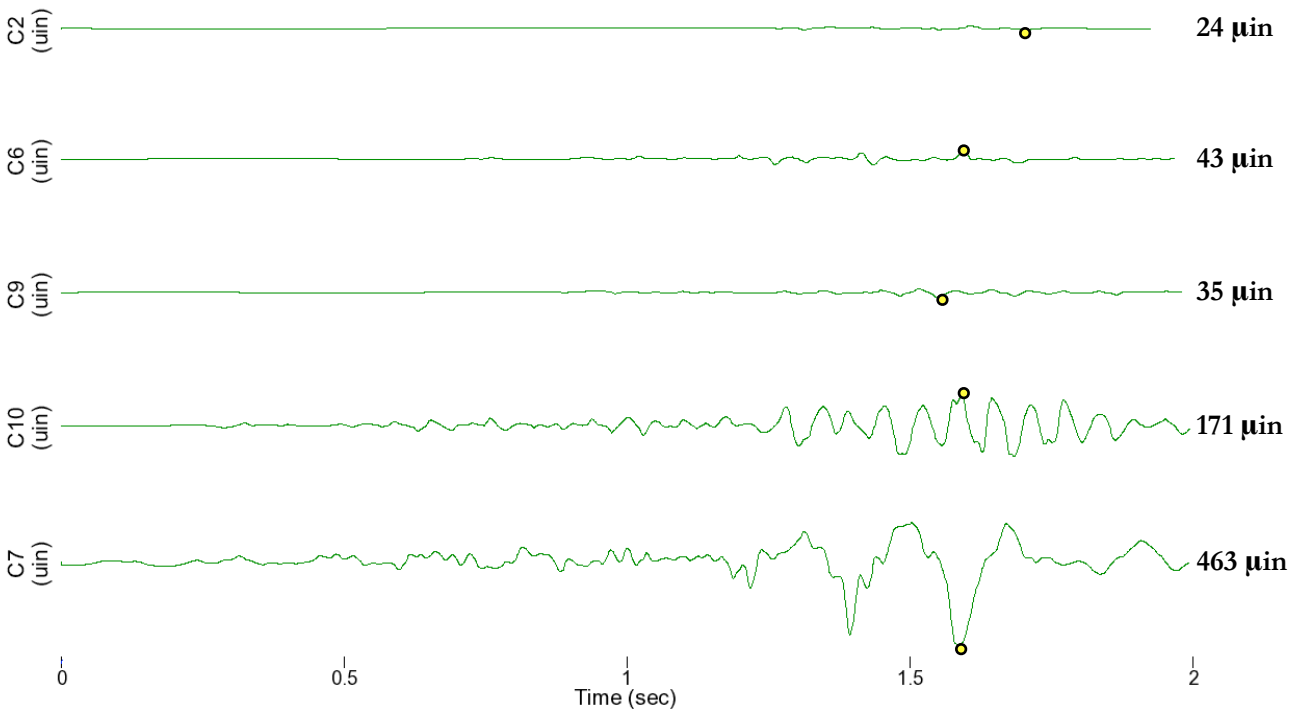
STRUCTURAL VELOCITY



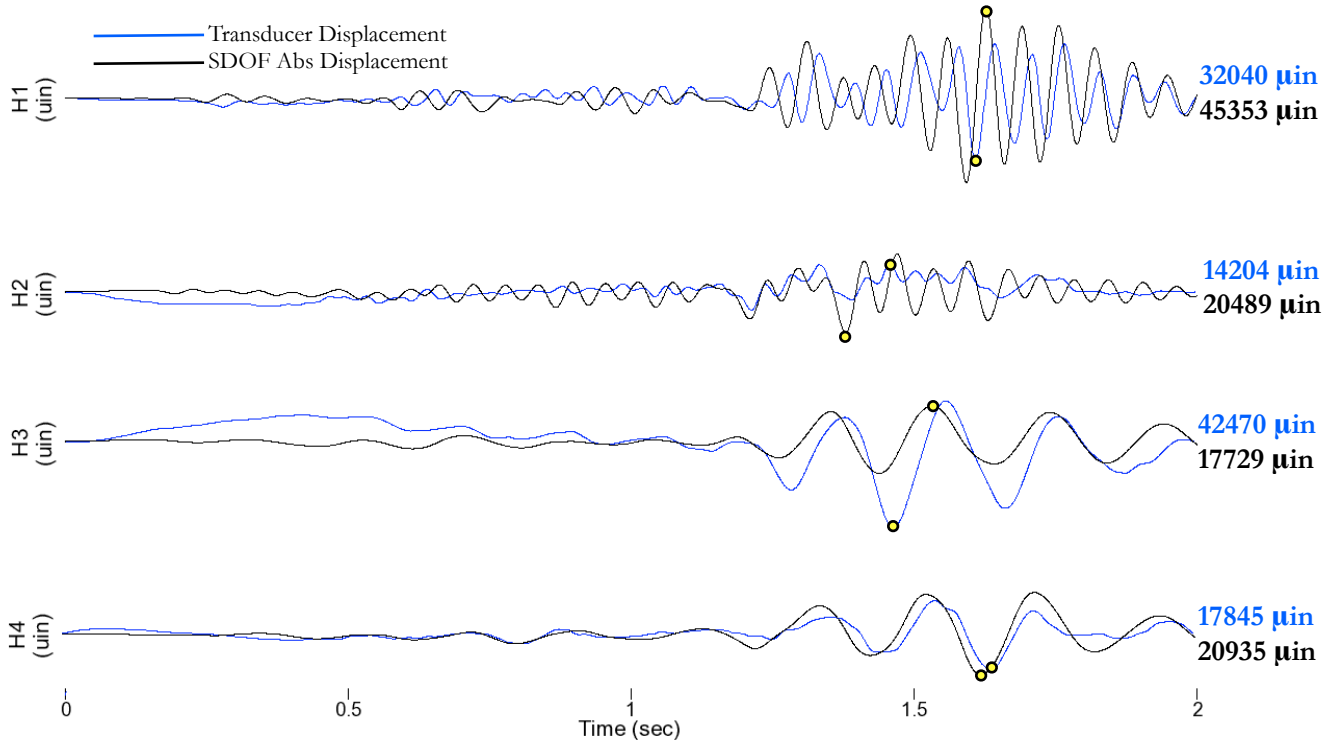
GROUND VELOCITY



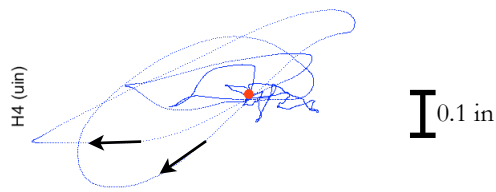
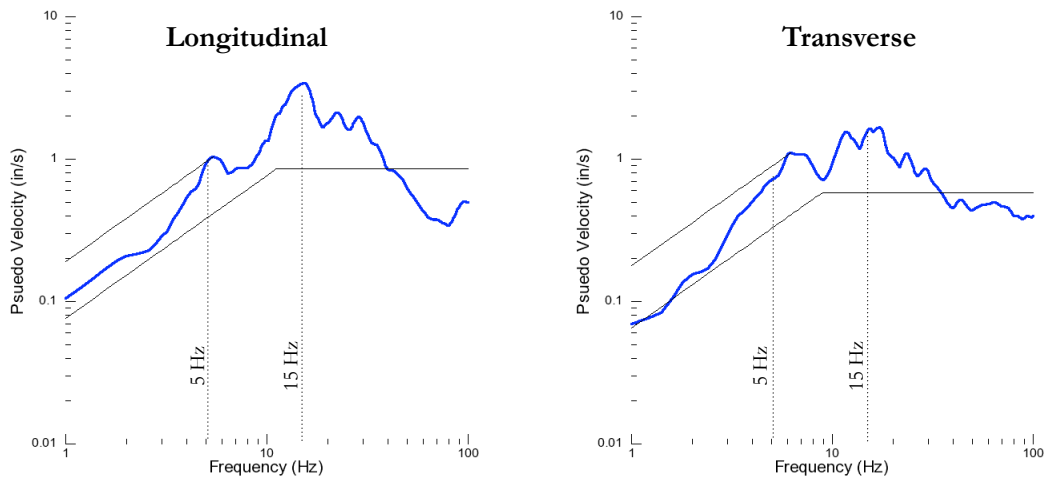
CRACK RESPONSE



STRUCTURAL DISPLACEMENT (ABSOLUTE)



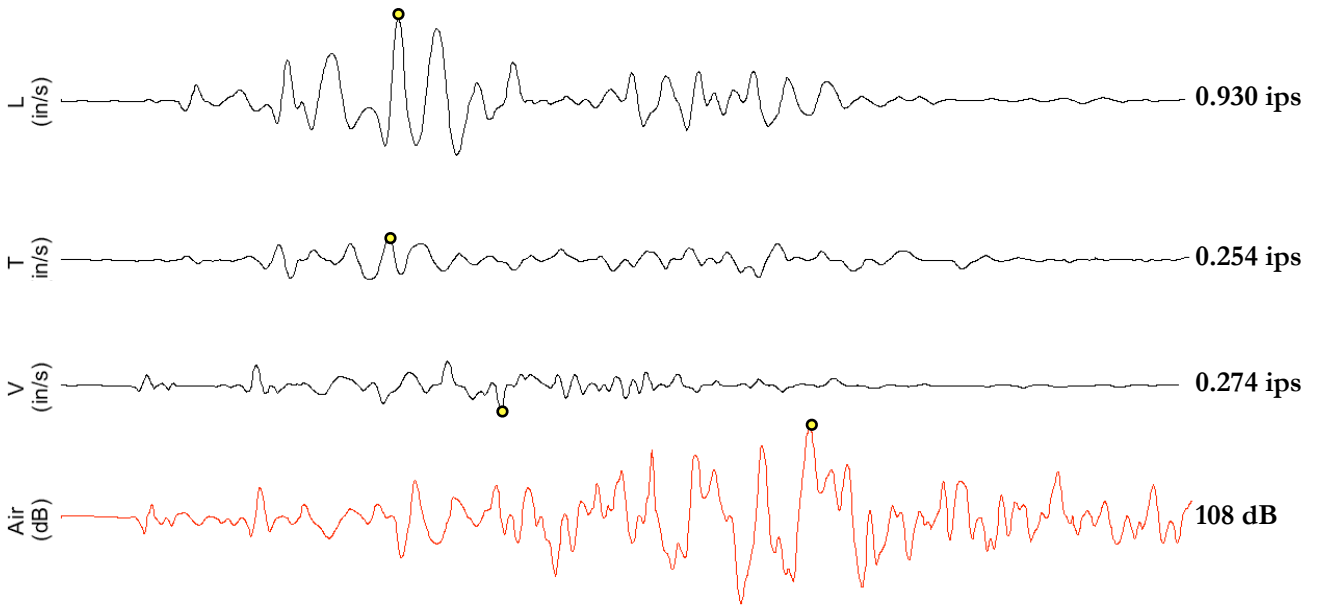
SDOF RESPONSE SPECTRA



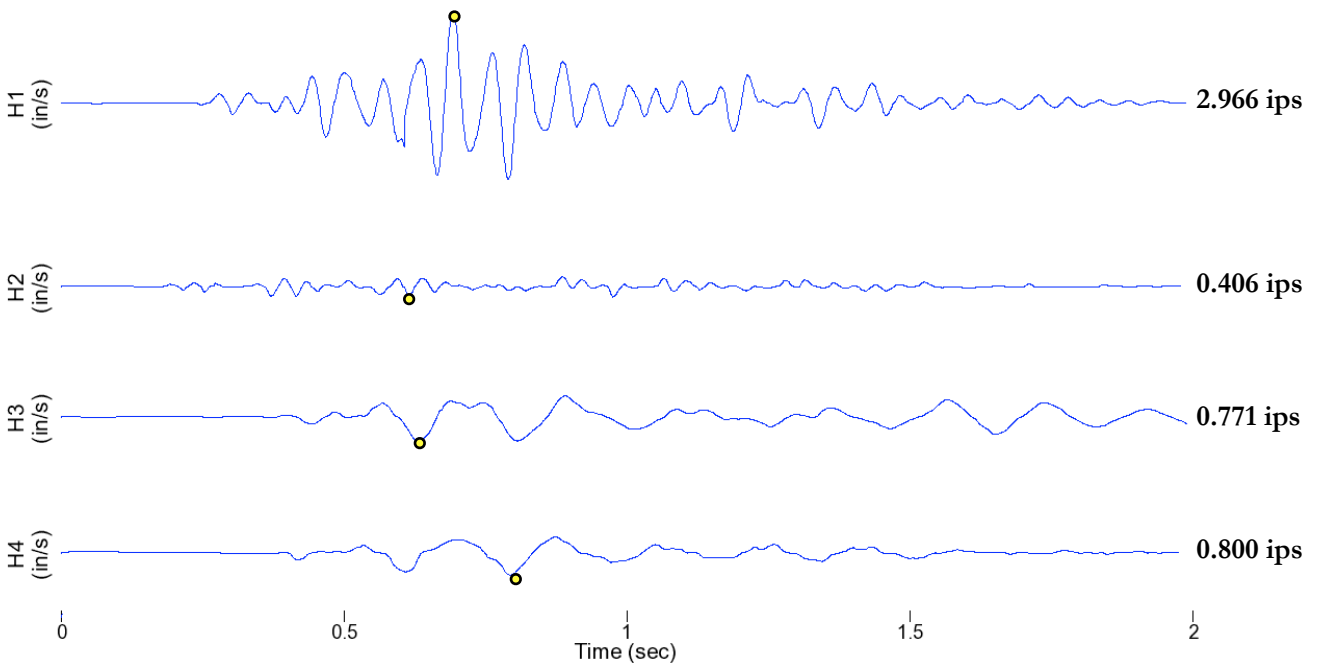
Corner Path

H3 (μin)

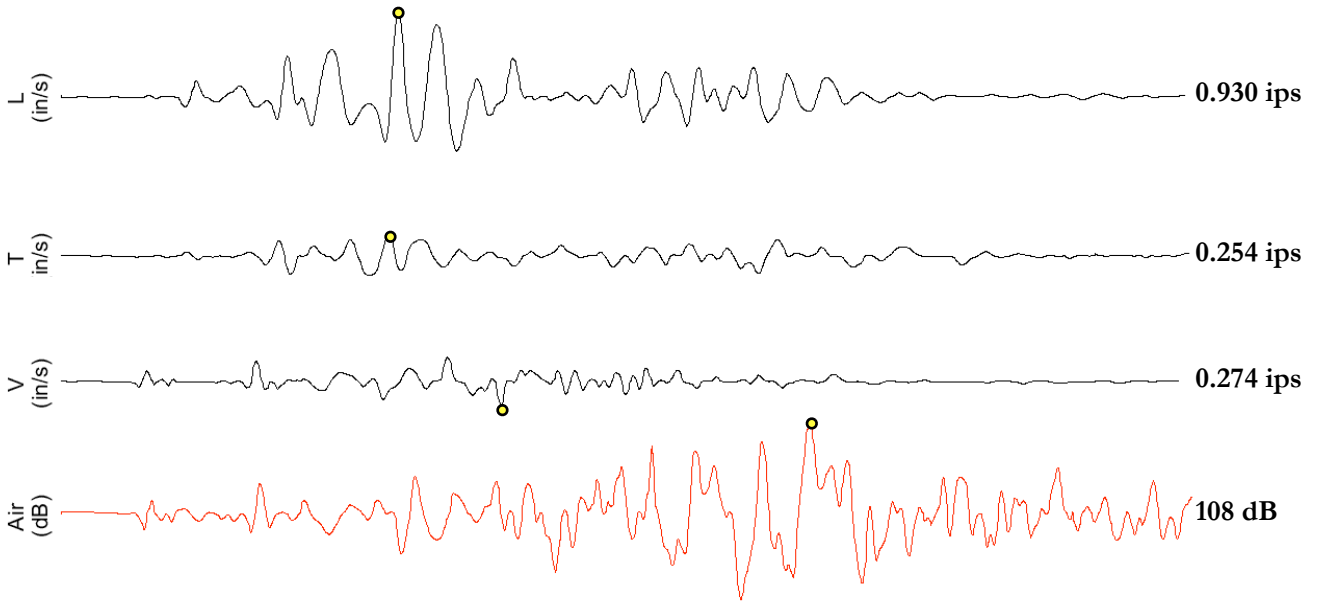
GROUND VELOCITY



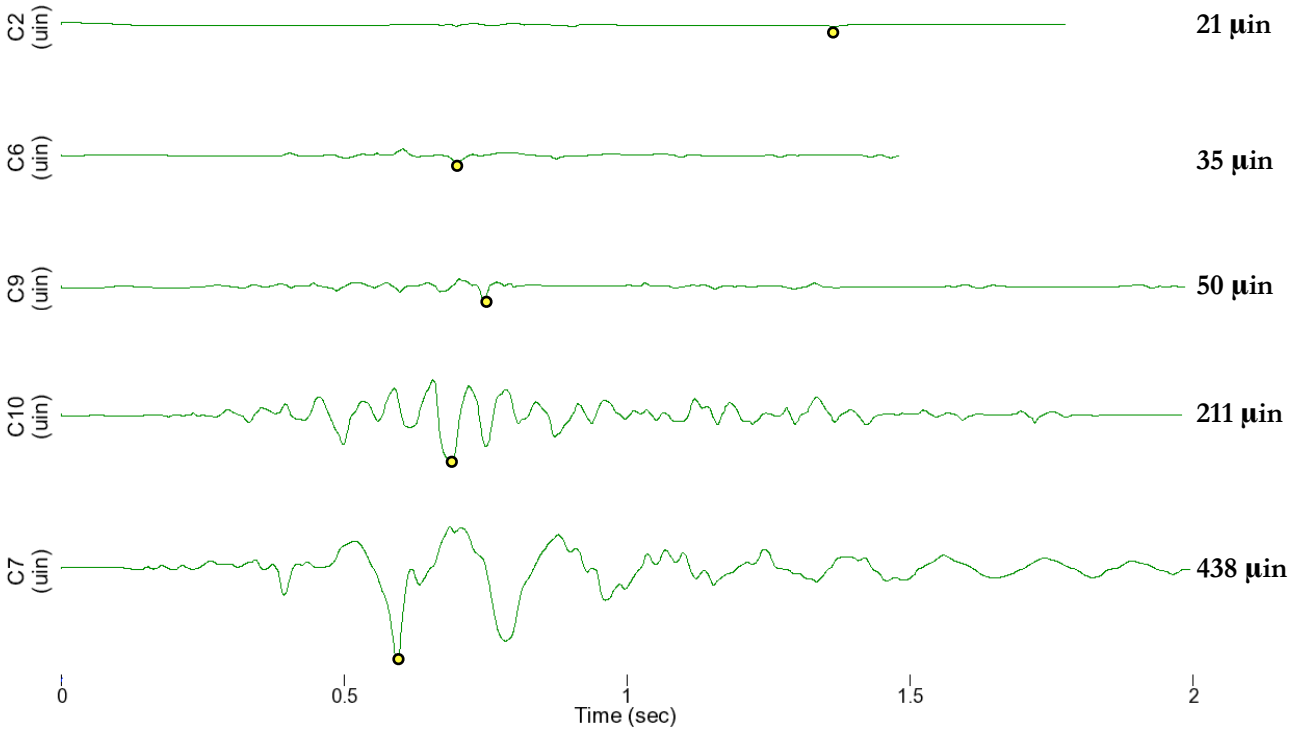
STRUCTURAL VELOCITY



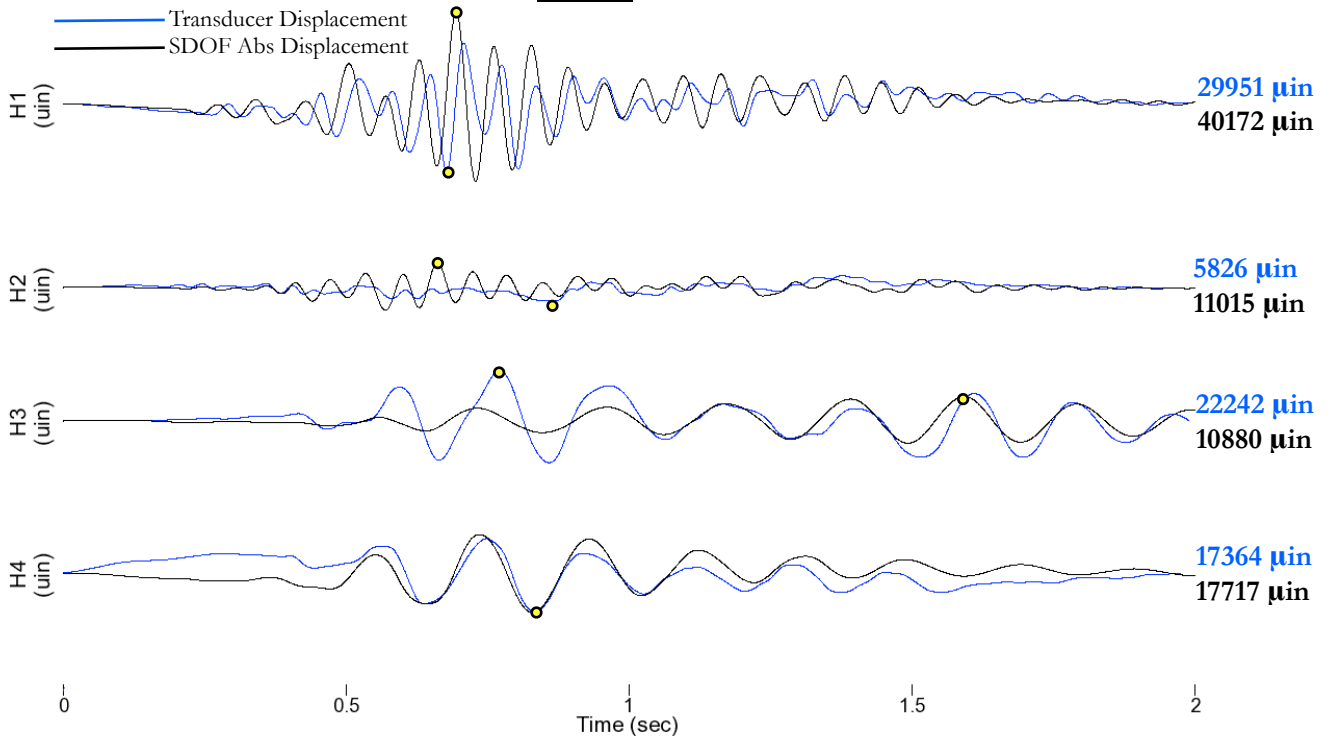
GROUND VELOCITY



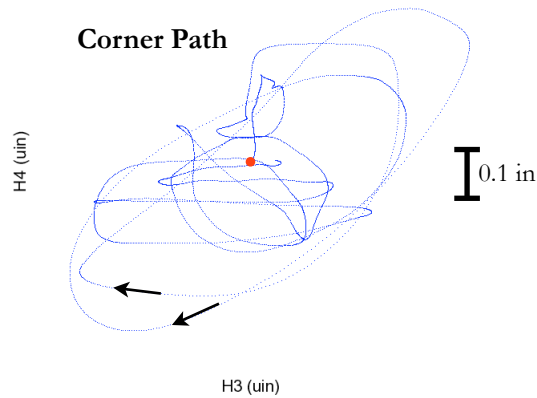
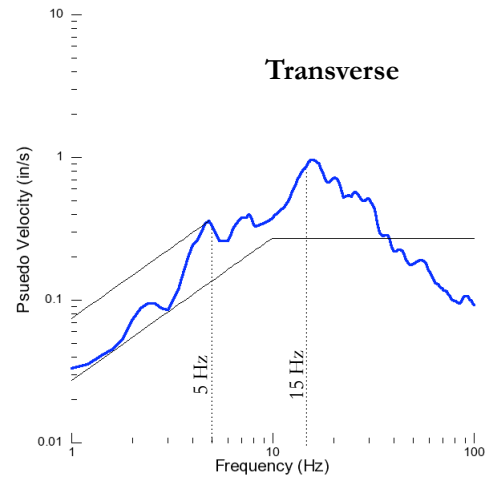
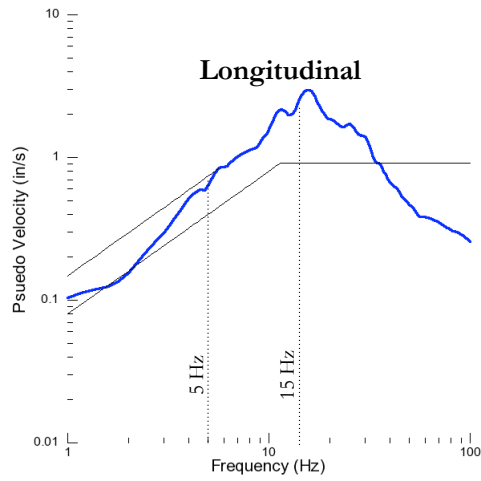
CRACK RESPONSE



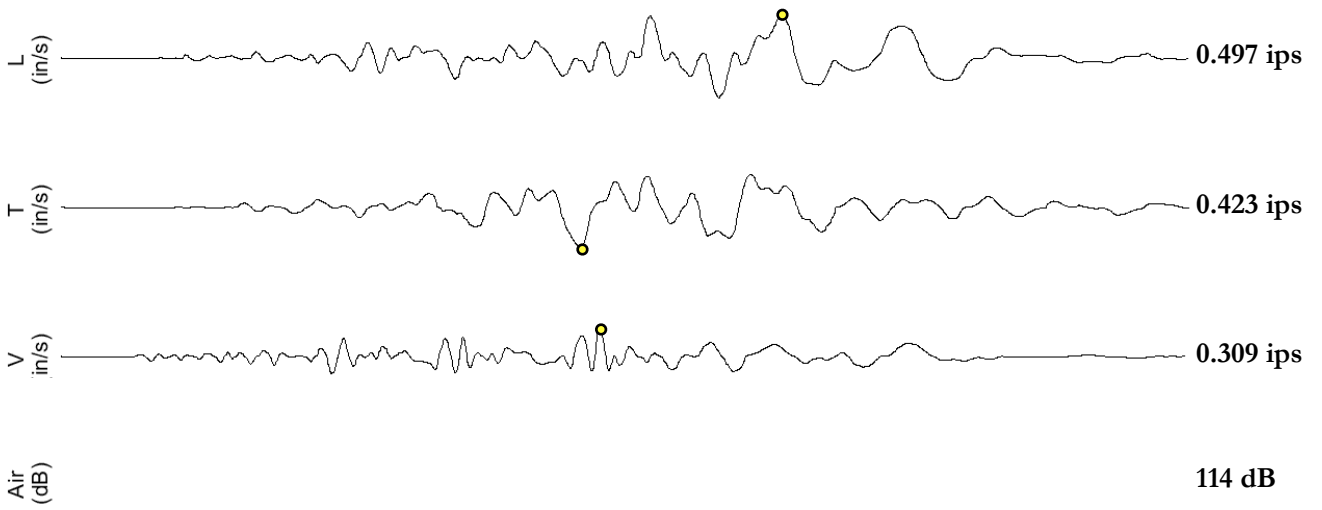
STRUCTURAL DISPLACEMENT (ABSOLUTE)



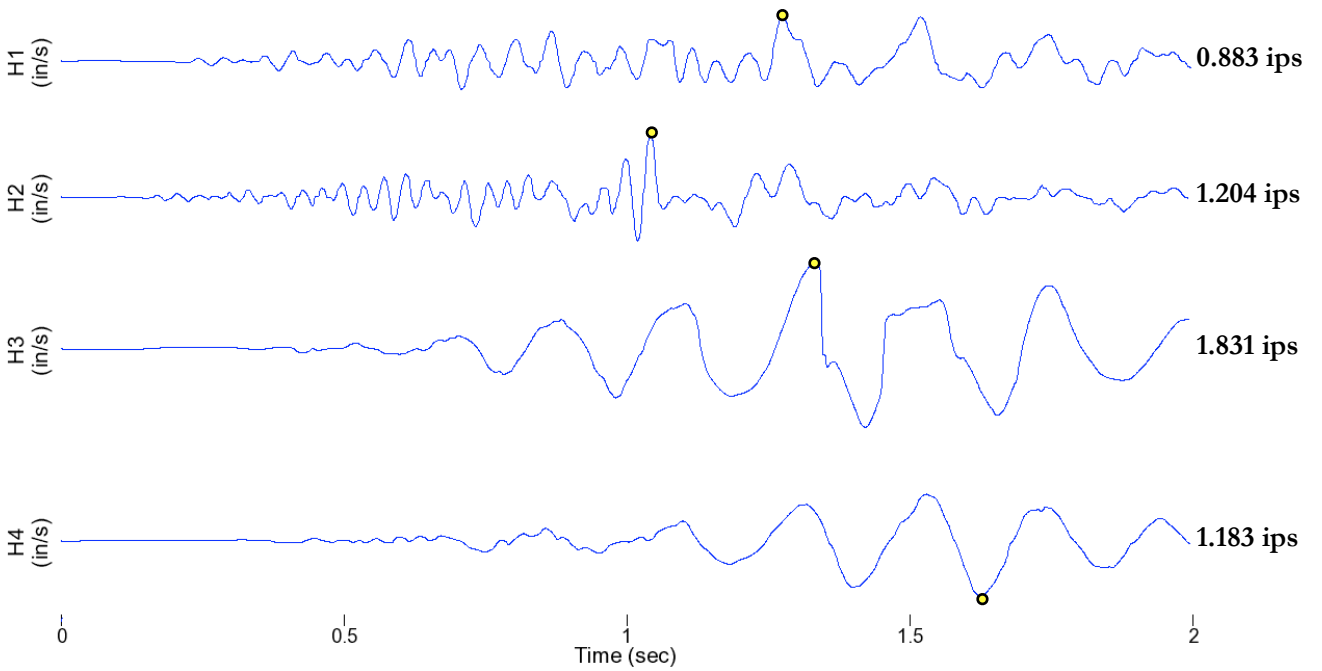
SDOF RESPONSE SPECTRA



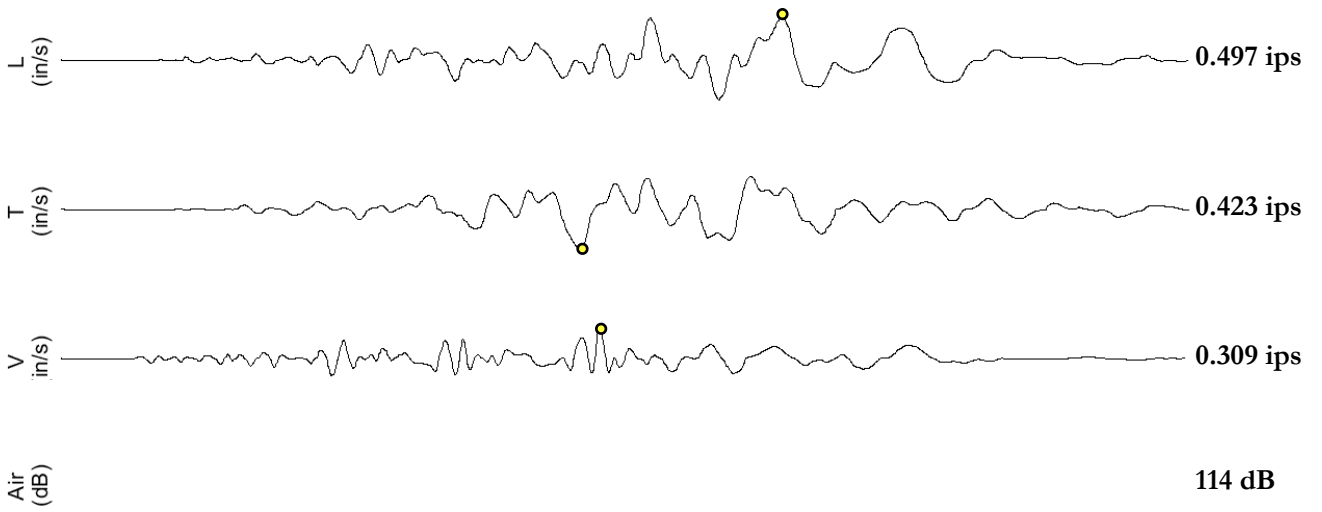
GROUND VELOCITY



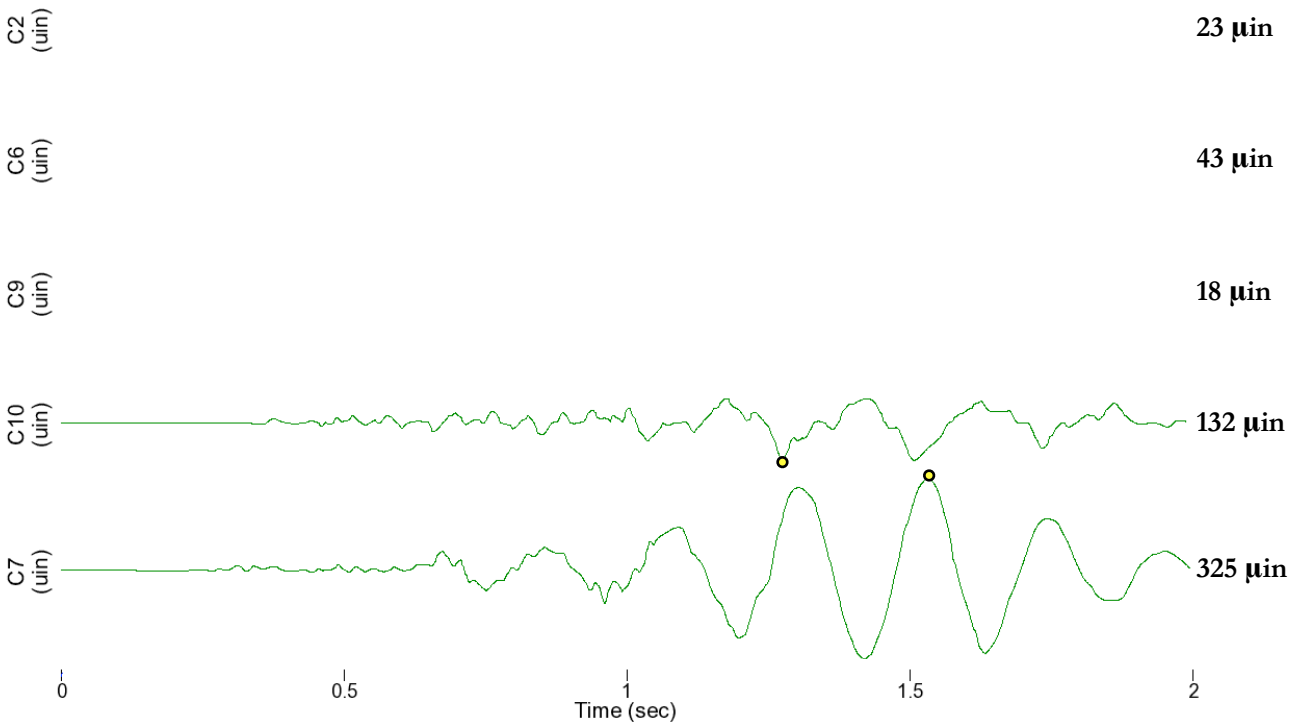
STRUCTURAL VELOCITY



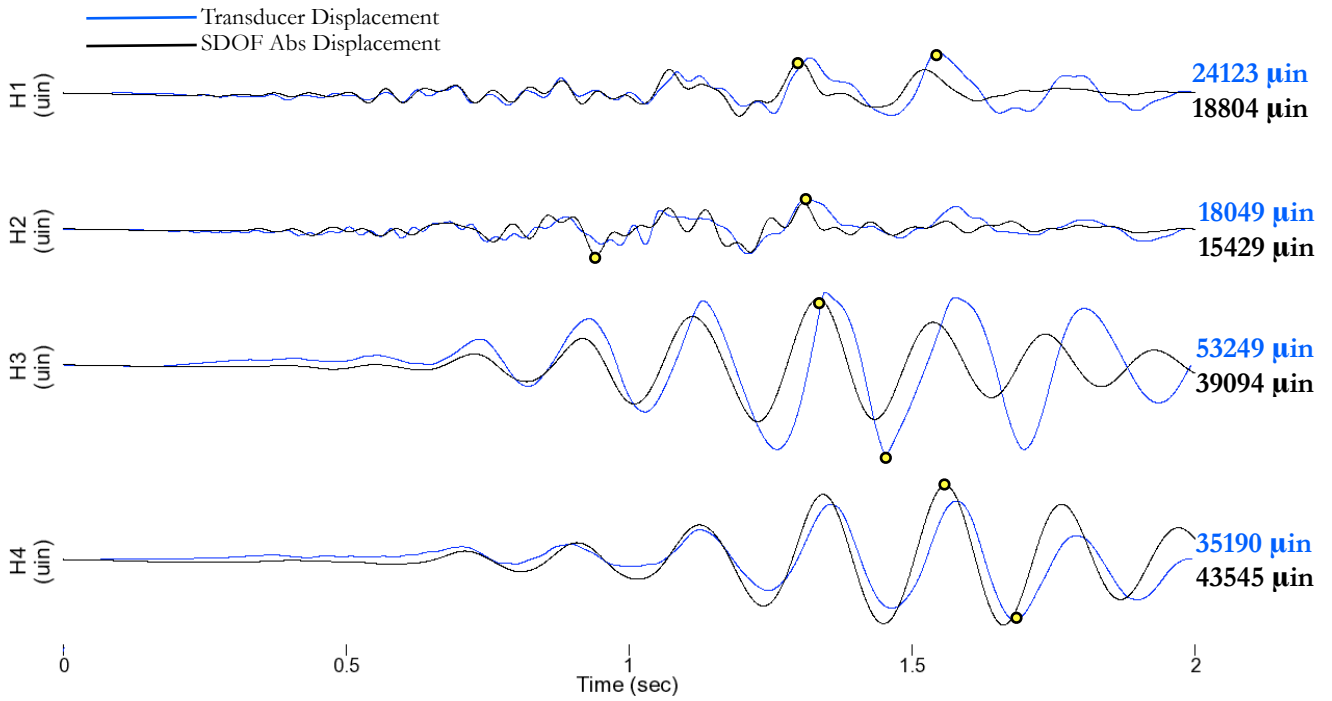
GROUND VELOCITY



CRACK RESPONSE



STRUCTURAL DISPLACEMENT (ABSOLUTE)



SDOF RESPONSE SPECTRA

