

NORTHWESTERN UNIVERSITY

**Crack Response to Weather Effects, Blasting, and Construction  
Vibrations**

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By

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## *Abstract*

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Autonomous Crack Measurement (ACM) facilitates simultaneous measurement of crack response to environmental changes and vibrations produced by various construction activities. Dual-purpose crack displacement sensors measure crack response, while the vibration environment is defined by standard seismological transducers and the weather environment is defined as changes in temperature and humidity.

This investigation involved an ACM study to examine the effects of rock blasting at the Stiles Road Quarry site in Southbury, Connecticut and vibrations produced by heavy construction equipment at the West Ann Road site in Las Vegas, Nevada. The study also allowed for the examination of blast design effects on crack displacement and a comparison of crack response from typical blasting and construction activities with that produced by weather changes.

Measurements and analysis show that (i) long-term weather-induced crack displacement is 30 to 150 times greater than the crack displacement produced by the largest blasting event ( $\approx 0.35$  ips) at the Connecticut site and the largest construction event ( $\approx 0.45$  ips) at the Las Vegas site, (ii) ground vibration frequency and stemming length have the largest effect on crack displacement of the four blast design controls studied, (iii) appropriate triggering mechanisms and on-site observation greatly facilitate vibration monitoring, and (iv) rock blasting at distances of approximately 2500 feet produced homogenous crack response, while localized construction activity in soils at distances less than 50 feet produced time varying, localized crack response.

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## *Chapter 1*

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### *Introduction*

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This thesis summarizes micro-inch crack response to blast-induced ground motions from the Stiles Road Quarry in Southbury, Connecticut, construction equipment-induced ground motions along West Ann Road in North Las Vegas, Nevada, and environmental phenomena at both sites. These structures were instrumented, and their response studied as part of the development of an Autonomous Crack Measurement (ACM) system sponsored by the Infrastructure Technology Institute at Northwestern University through a block grant from the United States Department of Transportation. The objective of the ongoing Autonomous Crack Monitoring study is to record and compare micro-inch crack displacements produced by long-term temperature and humidity changes to those produced by short-term blasting or construction vibrations in a concise and understandable fashion.

Responses of the Connecticut structure were measured with velocity transducers instrumented in the traditional manner of those in the study sponsored by the United States Department of the Interior Office of Surface Mining (OSM) (McKenna, 2002). Ground motions were measured in three orthogonal axes in front of the house. Structural responses were measured with three upper structure velocity transducers, three lower structure velocity transducers, two mid-wall velocity transducers, an air pressure transducer, and for the final month of study, one mid-ceiling velocity transducer. One wall and two ceiling cosmetic drywall cracks were fit with eddie current "Kaman"

sensors to measure micro-inch displacement response to environmental and blast-induced ground vibrations produced by the aggregate quarry approximately 2500 feet away.

Ground motions at the Las Vegas structure were measured via an in-ground tri-axial geophone customary to all ACM structures. Micro-inch displacements were measured across two interior drywall cracks and two exterior stucco cracks with LVDT displacement gages. No velocity response was measured in this structure. Construction adjacent to the house (within 50 feet) involved excavation for the installation of a 10x12 ft. reinforced concrete box culvert by trackhoe, excavation of an 8-inch water service line trench by a chain trencher, and vibratory compaction of trench backfill and granular sub-grade for the reconstruction of West Ann Road.

This thesis is divided into six chapters. Chapter 2 presents traditional OSM velocity and crack instrumentation, and the monitored response in the Connecticut house. The chapter includes the following:

- Description of the structure and location of instruments
- Summary of measurements recorded for each of 24 blast events occurring between May 20<sup>th</sup>, 2002 and September 20<sup>th</sup>, 2002.
- Determination of dominant/natural frequency of the structure
- Crack response to long-term and environmental phenomena
- Comparison and correlation of measured crack displacements to traditionally measured and calculated velocity and displacement parameters

Chapter 3 involves a discussion of the effect of blast design on crack displacements. Four time history comparisons are chosen to analyze the effects of differences in:

- Face geometry
- Stemming depth
- Frequency effect
- Total shot time and number of boreholes

Chapter 4 is a discussion of the construction processes and instrumentation for monitoring of construction vibrations at the Las Vegas, Nevada site. The chapter presents:

- Layout and structural details of house and proximity to monitored construction events
- Instruments and locations employed for monitoring purposes
- Soil profile of West Ann Road
- Introduction of construction equipment: trackhoe, trencher, and two vibratory rollers
- Ground attenuation study of trackhoe, trencher and rollers through cemented desert colluvium

Chapter 5 contains the measurement and analysis of heavy construction equipment-induced vibration response at the Las Vegas site. The chapter includes the following:

- Data acquisition, triggering mechanisms, and the challenges presented by construction monitoring
- Long-term response to environmental effects in Las Vegas
- An introduction to "non-vibratory response" monitoring and the proposed stick-slip phenomenon
- Event triggering, ground motions and crack displacements resulting from construction activities of a trackhoe, trencher, and two vibratory rollers
- Determination of dominant/natural frequencies of the Las Vegas structure
- Single degree of freedom response spectra for the trackhoe and vibratory rollers
- Comparison of traditional motion controls to measured crack displacements

Chapter 6 compares blasting response from the Connecticut site to construction equipment response at the Las Vegas site via the following:

- Comparison of time histories from Connecticut blasting and Las Vegas equipment and the number of principal pulses involved

- Introduction to "Normalization Factor" and the effect of 5, 10, 15, and 20 Hz signals on crack displacement
- Effect of construction vibration wavelength on homogenous structural motion versus individual wall component displacement
- Planar wave motion in Connecticut compared to radial wave motion in Las Vegas
- Determination of individual crack sensitivities to changes in humidity



## *Chapter 2*

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### *Blasting Vibration Response, Southbury, Connecticut*

---

#### *Introduction*

The Connecticut structure, shown in Figure 2.1, is a one-story house with a walk-out basement and an apartment and garage addition located approximately 2000 to 2500 feet from an aggregate quarry in Southbury, Connecticut. Ground motion and crack data collected on-site from May 20<sup>th</sup>, 2002 to September 20<sup>th</sup>, 2002, is summarized in Table 2.1. Table 2.2 summarizes essential descriptors of the ground motion such as number of significant pulses, total shot time, scaled distance factors, shot geometry specifics, etc. These blasts produced peak particle velocities between 0.03 and 0.345 inches per second, peak airblast from less than 100 to 132 db, maximum upper structure responses of 0.030 to 0.480 inches per second, and maximum crack responses between 0 and 90 micro-inches. Weather conditions varied daily with indoor temperatures ranging between 88.7° and 65.9° F and indoor humidity ranging from 76.2 to 32.6 %.



**Figure 2.1 Single-Story Connecticut House with Apartment Addition**

Shot Date	shot number	GROUND MOTION AND AIRBLAST								CRACK DISPLACEMENT			
		Longitud. PPV	Transverse PPV	Vertical PPV	SDOF Frequency (Long.)	FFT Frequency (Long.)	SDOF Frequency (Vertical)	FFT Frequency (Vertical)	Airblast	Crack 1 (Vertical Plane)	Crack 3 (Long. Plane)	Normalized CD/PPV Crack 1/V	Normalized CD/PPV Crack 3/L
		(in/sec)	(in/sec)	(in/sec)	(Hz)	(Hz)	(Hz)	(Hz)	(dB)	( $\mu$ in)	( $\mu$ in)		
5/20/2002	1	0.18	0.165	0.055	21	21.5	24.0	22.0	106	41	N/A	745.5	N/A
5/22/2002	2	0.03	0.05	0.025	20	23.4	24.0	23.6	110	14	N/A	560.0	N/A
5/22/2002	3	0.065	0.05	0.02	19	18.1	40.0	37.6	100	19	N/A	950.0	N/A
5/23/2002	4	0.105	0.15	0.075	18	18.0	25.0	23.4	106	39	N/A	520.0	N/A
5/23/2002	5	0.125	0.085	0.065	21	21.9	24.0	23.0	122	44	N/A	676.9	N/A
5/23/2002	6	0.1	0.085	0.06	18	17.9	25.0	23.5	123	38	N/A	633.3	N/A
6/3/2002	7	0.08	0.08	0.04	19	22.4	27.0	22.8	106	30	N/A	750.0	N/A
6/10/2002	8	0.08	0.07	0.03	22	22.1	23.0	22.8	132	17	N/A	566.7	N/A
6/17/2002	9	0.145	0.125	0.055	22	18.5	25.0	24.9	110	29	25	527.3	172.4
6/18/2002	10	0.08	0.11	0.03	19	19.1	36.0	22.9	112	26	19	866.7	237.5
6/26/2002	11	0.08	0.065	0.05	24	19.0	25.0	23.3	110	25	15	500.0	187.5
7/9/2002	12	0.125	0.16	0.16	23	22.6	25.0	25.0	110	41	31	256.3	248.0
7/12/2002	13	0.08	0.065	0.045	25	24.5	25.0	25.0	122	20	13	444.4	162.5
7/16/2002	14	0.19	0.18	0.095	19	19.0	25.0	25.1	106	60	59	631.6	310.5
7/22/2002	15	0.345	0.255	0.095	23	22.8	25.0	23.6	117	90	50	947.4	144.9
7/26/2002	16	0.14	0.14	0.07	23	20.1	25.0	25.1	122	50	28	714.3	200.0
7/30/2002	17	0.075	0.055	0.035	25	23.5	25.0	25.9	117	23	12	657.1	160.0
8/2/2002	18	0.195	0.155	0.115	25	25.5	25.0	25.3	126	80	37	695.7	189.7
8/15/2002	19	0.215	0.13	0.085	22	23.0	25.0	25.9	119	38	33	447.1	153.5
8/23/2002	20	0.285	0.14	0.095	26	19.9	26.0	25.6	110	47	38	494.7	133.3
9/3/2002	21	0.28	0.21	0.1	23	21.4	25.0	25.8	110	70	60	700.0	214.3
9/10/2002	22	0.27	0.15	0.135	27	26.1	25.0	25.9	112	60	34	444.4	125.9
9/17/2002	23	0.28	0.175	0.095	21	21.5	25.0	31.0	110	57	48	600.0	171.4
9/20/2002	24	0.155	0.06	0.055	24	21.5	25.0	25.4	116	27	28	490.9	180.6

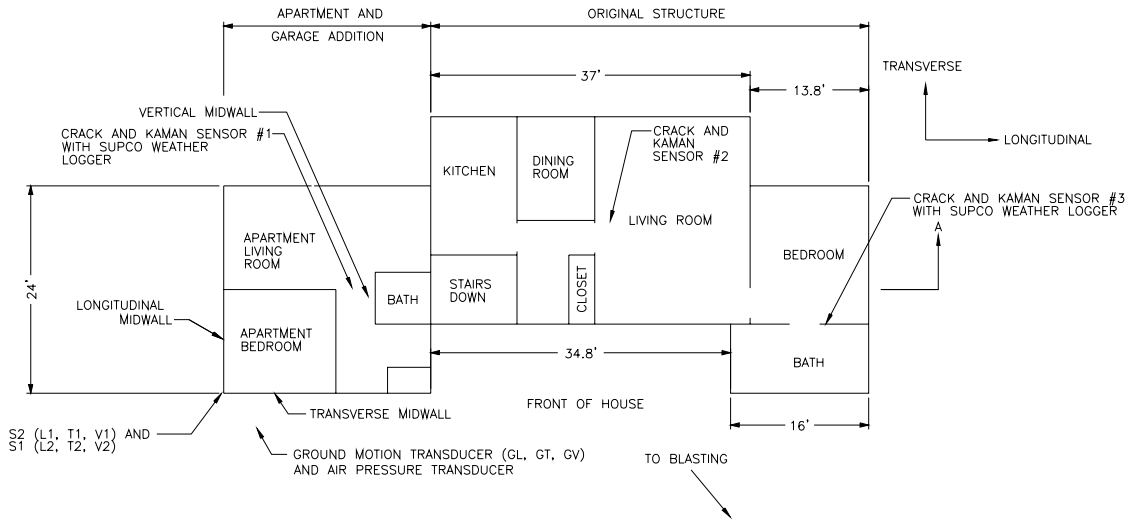
Table 2.1 Ground motion, frequency, airblast and crack displacement information for all blast events in Connecticut

Shot Date	shot number	Distance	Total Shot Weight	Total Number of Holes	Max. Holes per Delay (w/in 8ms)	Total Shot Time	Charge Weight per Hole	Charge Weight/ Delay (Calc.)	Charge Weight/ Delay (Given)	Scaled Distance (Calc.)	Scaled Distance (Given)	Number of Principal Pulses (Radial)	Number of Principal Pulses (Vertical)
		(ft)	(lb)			(ms)	(lb)	(lb)	(lb)	(ft/lb <sup>1/2</sup> )	(ft/lb <sup>1/2</sup> )		
5/20/2002	1	2429	8280	30	2	352	276	552	290	103.4	142.6	N/A	5
5/22/2002	2	2690	4409	16	1	470	276	276	303	162.0	154.5	N/A	5
5/22/2002	3	3625	8463	53	2	314	160	319	590	202.8	149.2	N/A	4
5/23/2002	4	2200	225	1	1	1	225	225	225	146.7	146.7	N/A	1
5/23/2002	5	2695	4719	27	5	59	175	874	885	91.2	90.6	N/A	3
5/23/2002	6	2260	177	1	1	1	177	177	177	169.9	169.9	N/A	1
6/3/2002	7	2510	7091	42	2	625	169	338	600	136.6	102.5	N/A	3
6/10/2002	8	2217	3920	48		557	147	358	600	117.2	90.5	N/A	4
6/17/2002	9	2693	10271	37	3	428	278	833	810	93.3	94.6	2	4
6/18/2002	10	2460	13216	40	2	287	330	661	800	95.7	87.0	3	2
6/26/2002	11	2430	7637	49	3	301	156	468	800	112.4	85.9	4	4
7/9/2002	12	2700	12261	46	3		267	800	870	95.5	91.5	5	3
7/12/2002	13	2190	4785	50	4		96	383	450	111.9	103.2	1	1
7/16/2002	14	2430	7957	41	3	310	194	582	400	100.7	121.5	5	2
7/22/2002	15	2376	13081	51	3	419	256	769	810	85.7	83.5	7	4
7/26/2002	16	2323	10649	54	N/A		197	N/A	600		94.8	7	5
7/30/2002	17	2240	8965	74	4	441	121	485	800	101.8	79.2	7	6
8/2/2002	18	2441	15345	59	3	436	260	780	660	87.4	95.0	2	5
8/15/2002	19	2490	7495	37	4	215	203	810	840	87.5	85.9	3	3
8/23/2002	20	2420	12804	40	2		320	640	660	95.6	94.2	3	3
9/3/2002	21	2385	12507	38	2		329	658	690	93.0	90.8	1	2
9/10/2002	22	2270	13251	57	3	402	232	697	500	86.0	101.5	3	5
9/17/2002	23	2640	10627	36	3		295	886	825	88.7	91.9	2	3
9/20/2002	24	2693	9780	47	3	430	208	624	720	107.8	100.4	3	1

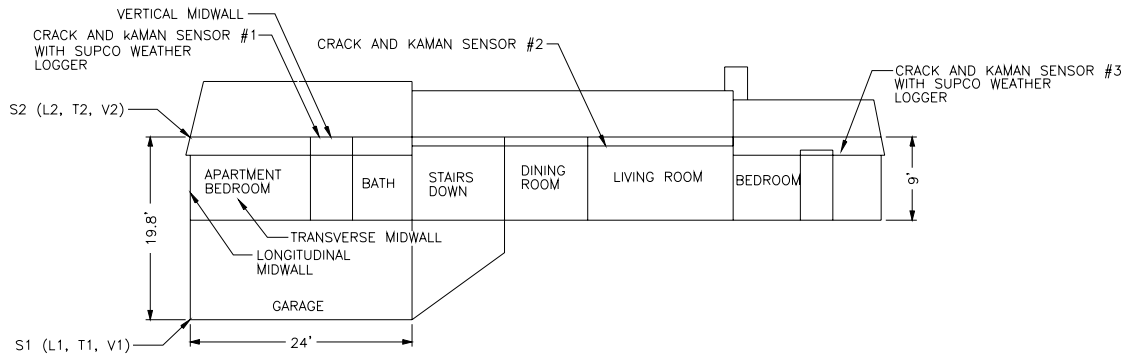
Table 2.2 Shot timing, scaled distances, explosive characteristics, and other pertinent logistical information for all blast events in Connecticut

**Structural Description**

Plan and elevation views of the Connecticut structure are shown in Figures 2.2 and 2.3. An apartment addition was constructed over the masonry block garage left of the main portion of the structure labeled “original structure” in Figure 2.2. The wood frame exterior is surfaced with wood shingle clapboard and on the interior with drywall. Photographs of the rear of the house in Figure 2.4 show its position on a hillside exposing the basement.



**Figure 2.2 Plan View of Connecticut House with Crack and Sensor Locations**



**Figure 2.3 Elevation View of Connecticut House with Crack and Sensor Locations**



(a)



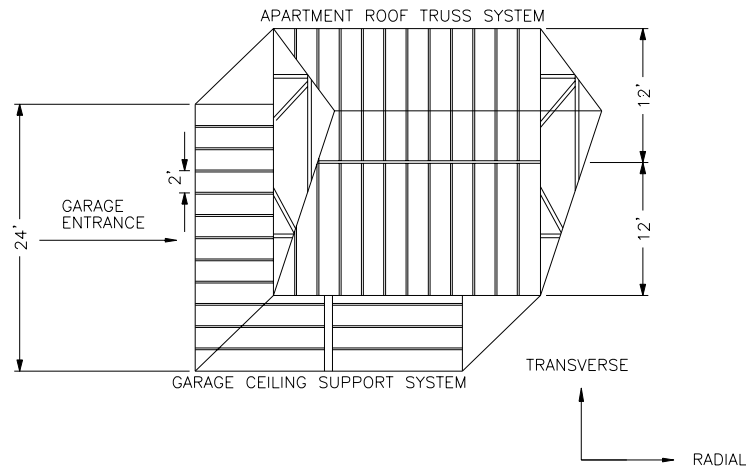
(b)

**Figure 2.4 Photos of Connecticut house (a) from the rear of the structure, and (b) outside the apartment/garage addition**

Figure 2.5 presents photographs of the garage ceiling under the addition, as well as the apartment roof/truss system. The unique feature of the addition is that the floor joists and ceiling joists run perpendicular to each other. Typically floor and ceiling joists are both oriented in the transverse direction of the house, which involves the shortest span. As shown in Figures 2.5 and 2.6, the floor joists between the garage and apartment run parallel to the longitudinal direction supported by a main steel beam in the transverse direction, while the ceiling joists run in the traditional transverse direction and are supported by interior walls. Thus there are several joists in the hallway that span the entire 24' width of the structure, otherwise all joist spans are on the order of twelve feet. This support configuration may have a significant effect on the vertical support of the apartment containing Crack 1.



**Figure 2.5 Photos of ceiling joist systems (a) garage ceiling/apartment floor system, (b) apartment addition ceiling truss system**



**Figure 2.6 Schematic of apartment/garage addition joist systems**

**Instrumentation**

All instrument locations are shown on Figures 2.2 and 2.3. Eight structural velocity transducers were mounted on the southern and western walls of the apartment addition, and an additional transducer was added to the mid-ceiling on August 9<sup>th</sup>, 2002. A tri-axial seismograph and an airblast transducer were installed outside of the southwest corner of the apartment addition.

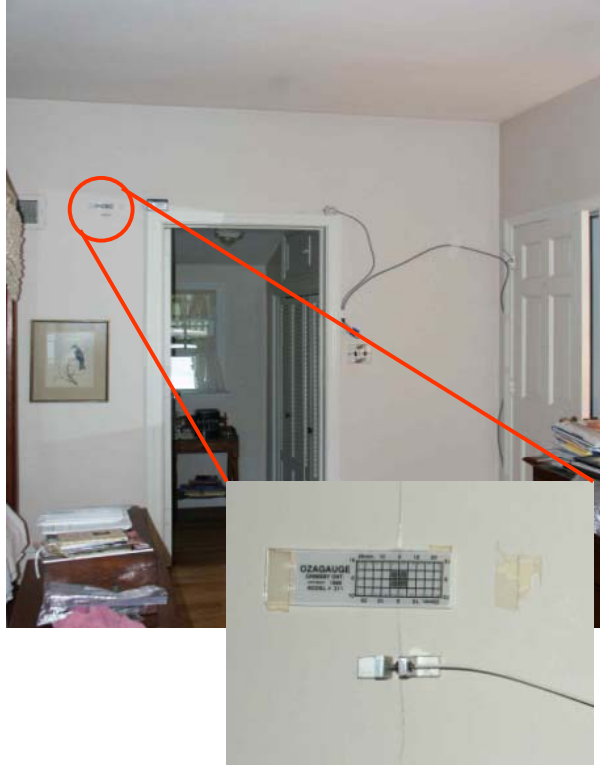
Three cracks were instrumented with eddy current “Kaman” micrometer displacement sensors for this case study. Crack #1 was in the apartment ceiling,

identified as Sensor 1 and is shown on the ceiling and in detail in Figure 2.7(a). Two other cracks on a wall and ceiling, were in the main portion of the structure and are identified as sensors 2 and 3, sensor 3 is shown in Figure 2.7(b). Crack 1 runs east-west at the mid-span of the unsupported joists at the beginning of a hallway leading between rooms in the addition. Crack 2 lies in the center of the living room ceiling, and Crack 3 runs vertically up an interior wall in the bedroom of the main portion of the structure.

For each blast, seismograph (ground), and velocity transducer data were collected for eight seconds. Any channel could trigger the entire system. Time correlated time histories of dynamic, blast induced crack displacements were measured by the Kaman sensors for 5 seconds. Temperature and humidity were recorded in each room containing a crack every 10 minutes by independent Supco weather loggers.

One of the challenges involved in this case study is the correlation of the responses. Typically, structure response is measured at the top ( $S_2$ ) and bottom ( $S_1$ ) of single-stories or structures with uniform framing and materials. In this case, however,  $S_1$  response is measured near the base of the concrete masonry unit garage wall, while  $S_2$  response is measured near the ceiling at the southwest corner of the wood frame and drywall apartment addition. There was no response data recorded at the junction of these two different wall types. Consequently, the gross wall, in-plane shear distribution cannot be assumed because the distribution of motion will differ due to radically different wall types. This challenge emphasizes the necessity for three sensor locations for two-story structures to ensure that the proper response mode shape is chosen for each story.

Furthermore, structural response was measured only in the southwest corner of the apartment addition. Crack 1 lies within this portion, supported by the garage underneath. Cracks 2 and 3, however, are located some distance away, within the main, un-instrumented portion of the house that includes no underlying basement. Therefore, the structural responses measured at this site are most applicable for the addition and response of Crack 1. Both of these challenges will be explored further when correlations of crack displacements and structural motions are investigated.



**Figure 2.7 Wall view and detail photographs of Cracks 1 (top) and 3 (bottom) with sensors**



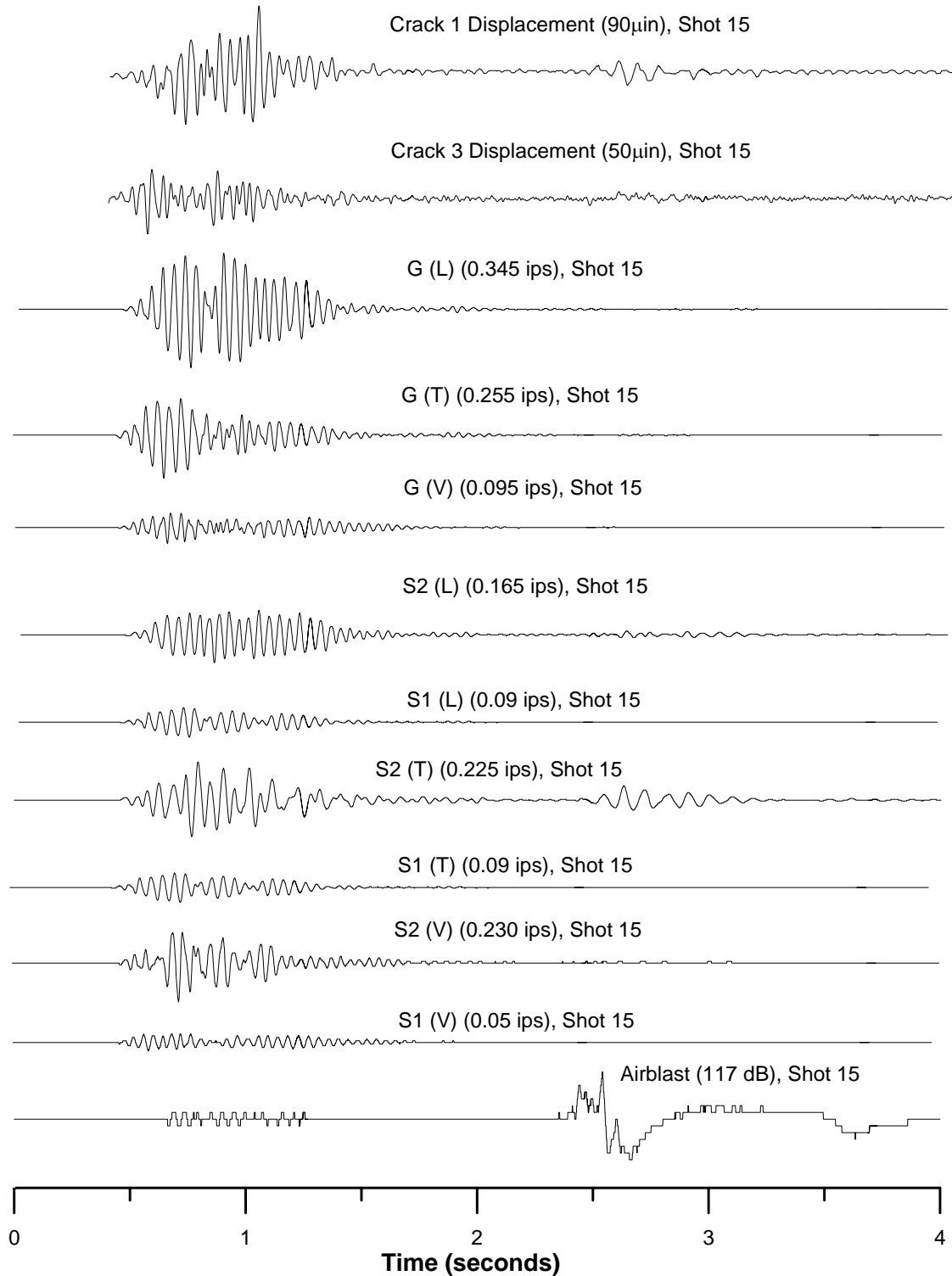
### **Blast Response**

Figure 2.8 shows longitudinal, transverse and vertical time histories of excitation ground velocities, structural response velocities, and the corresponding apartment and bedroom wall crack response for blast 15 with a peak particle velocity of 0.345 ips (8.8 mm/sec), measured in the longitudinal direction. Each waveform includes maximum measured values in parenthesis. These responses are parallel to the plane of the wall containing Crack 3, and therefore are employed to calculate gross wall displacements to compare with directly measured crack displacements. The top two graphs show measured apartment addition Crack 1 displacement and bedroom wall Crack 3 displacement, followed by longitudinal, transverse and vertical particle velocity, upper corner ( $S_2$ ) longitudinal, transverse, and vertical velocity response, lower corner ( $S_1$ ) longitudinal, transverse and vertical velocity response, and airblast. This event produced a peak Crack 1 displacement of  $90\mu\text{in}$  ( $2.29\mu\text{m}$ ), and peak Crack 3 displacement of  $50\mu\text{in}$  ( $1.25\mu\text{m}$ ).

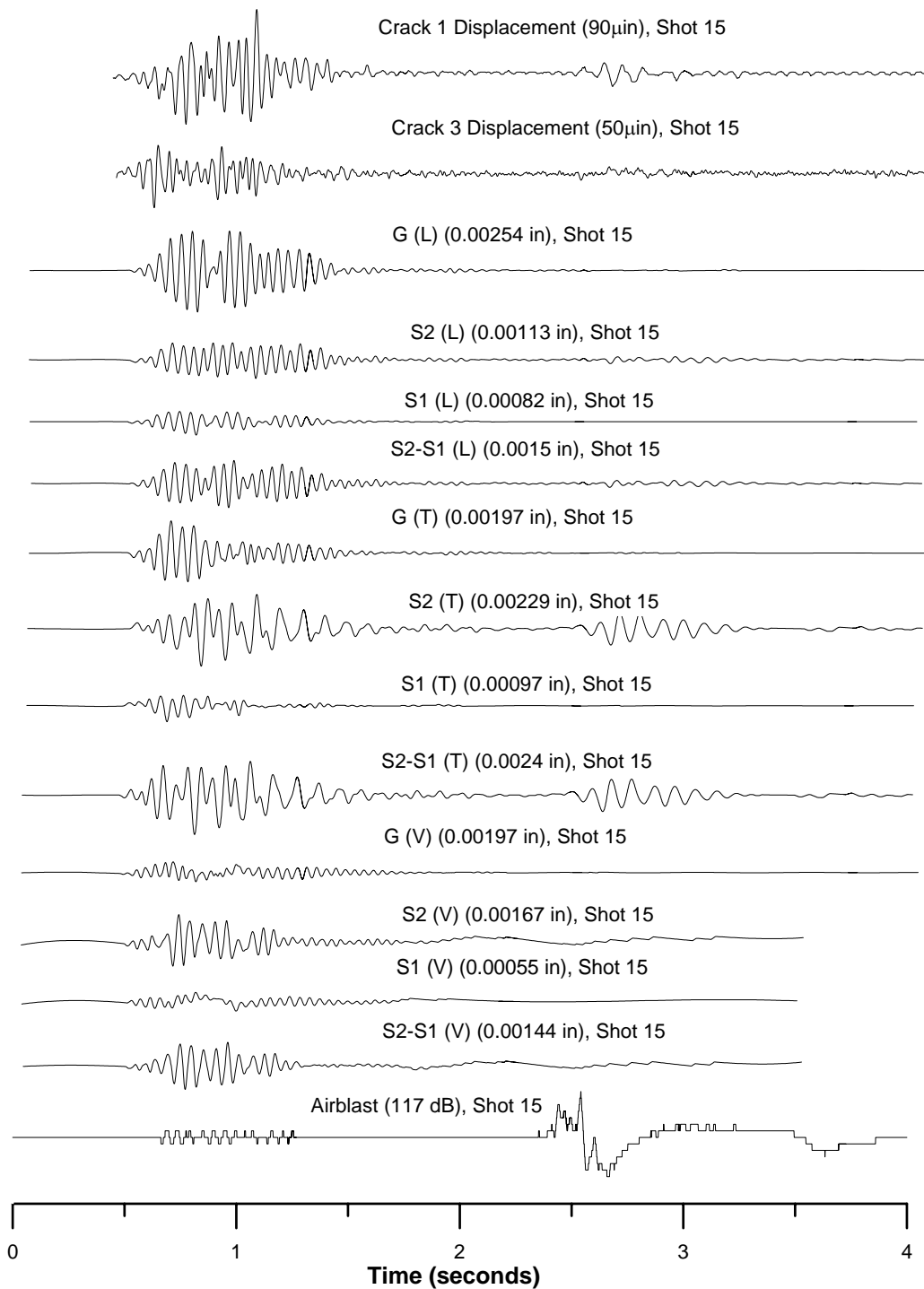
Figure 2.9 shows the same Crack 1 and Crack 3 displacements with longitudinal, transverse, and vertical ground,  $S_2$  and  $S_1$  displacements, as well as calculated longitudinal, transverse, and vertical relative ( $S_2-S_1$ ) displacement time histories, and airblast.

This event produced the largest overall ground motion and largest apartment Crack 1 displacement, but did not produce the largest Crack 3 displacement. Chapter 3 discusses the intricacies of blasting design, and goes further into detail on why certain shot geometries and timing produces relatively larger or smaller crack displacements.

Natural frequency and damping of the structure are important for calculating single degree of freedom response spectra. Thus, the dominant frequency of the structure must be estimated. Dominant frequencies of structure response (natural frequency) are estimated employing either the Fourier Frequency analysis or the zero-point-crossing method during free response (Dowding 1996). The natural frequency of the Connecticut structure was determined via both methods.

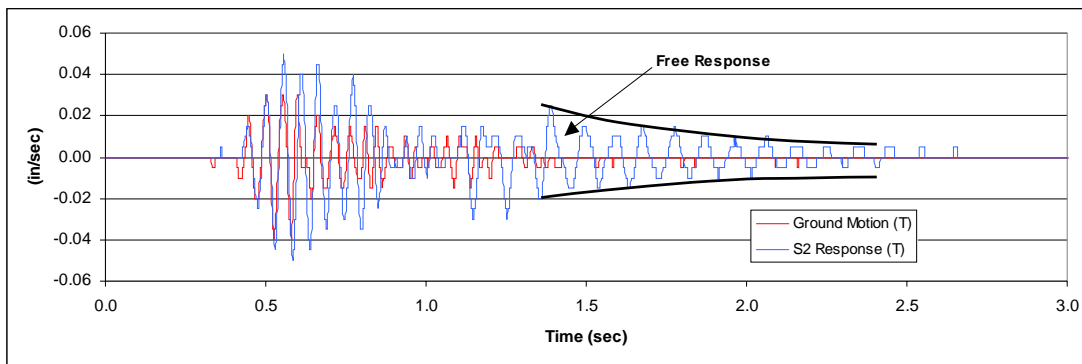


**Figure 2.8 Time histories of Crack 1 and 3 displacements from blast event 15 on July 22nd, 2002 at 11:49 AM compared to longitudinal, transverse, and vertical velocity ground excitation,  $S_2$  response velocity,  $S_1$  response velocity, and air blast.**



**Figure 2.9 Time histories of Crack 1 and 3 displacements from blast event 15 on July 22nd, 2002 at 11:49 AM compared to longitudinal, transverse, and vertical ground displacements,  $S_2$  displacement response,  $S_1$  displacement response,  $(S_2-S_1)$  relative displacement, and air blast.**

“Free response” is defined as structural response continuing after the cessation of ground motion. Where free response occurs, as shown in Figure 2.10, the zero-point-crossing method may be employed. Free response is necessary for the development of this method because the natural frequency of a structure is most easily identified when it is no longer excited by ground motion. The inverse of twice the time between successive zero-crossings, or the period, results in an estimated dominant/natural frequency of the structure. Natural frequencies estimated from  $S_2$  horizontal, radial time histories, during free response averaged 11-12 Hz.



**Figure 2.10 Free response of an  $S_2$  velocity time history in the Connecticut house**

The Fourier Frequency Spectra approach must be employed when little or no free response is detected in a  $S_2$  time history. For the purposes of this case study, Fourier Frequency Transforms (FFT) are calculated with the dedicated software White Seismograph Data Analysis (White Industrial Seismology 1998), and Northwestern University Vibration Analysis, or NUVIB (Huang 1994). White Seismograph Data analysis only accepts data files secured from White seismographs. Therefore, NUVIB must be employed to transform crack displacement time histories. The ratio of structural response and ground motion FFT amplitudes of the same component provides the means to determine the dominant frequency of the structure, and is shown in Figure 2.11 (a) for blast event 15 on July 22<sup>nd</sup> in the longitudinal direction, and Figure 2.11 (b) for event 8 on June 10<sup>th</sup> in the vertical direction. In this figure, the longitudinal  $S_2$  velocity spectra (middle), is divided by the longitudinal ground motion spectra (bottom) to obtain the dominant frequency spectra ratio (top). False peaks may develop when small structural amplitudes are divided by much smaller ground motion amplitudes. To prevent these large ratios of insignificant response and excitation, broad-frequency band, low amplitude

noise should be added to both the structural and ground motion amplitudes (Dowding 1996). Alternately, these false peaks can be filtered out by replacing spectra amplitudes less than ten percent of the peak amplitude with a value of exactly ten percent of the peak. This latter approach was followed in this case.

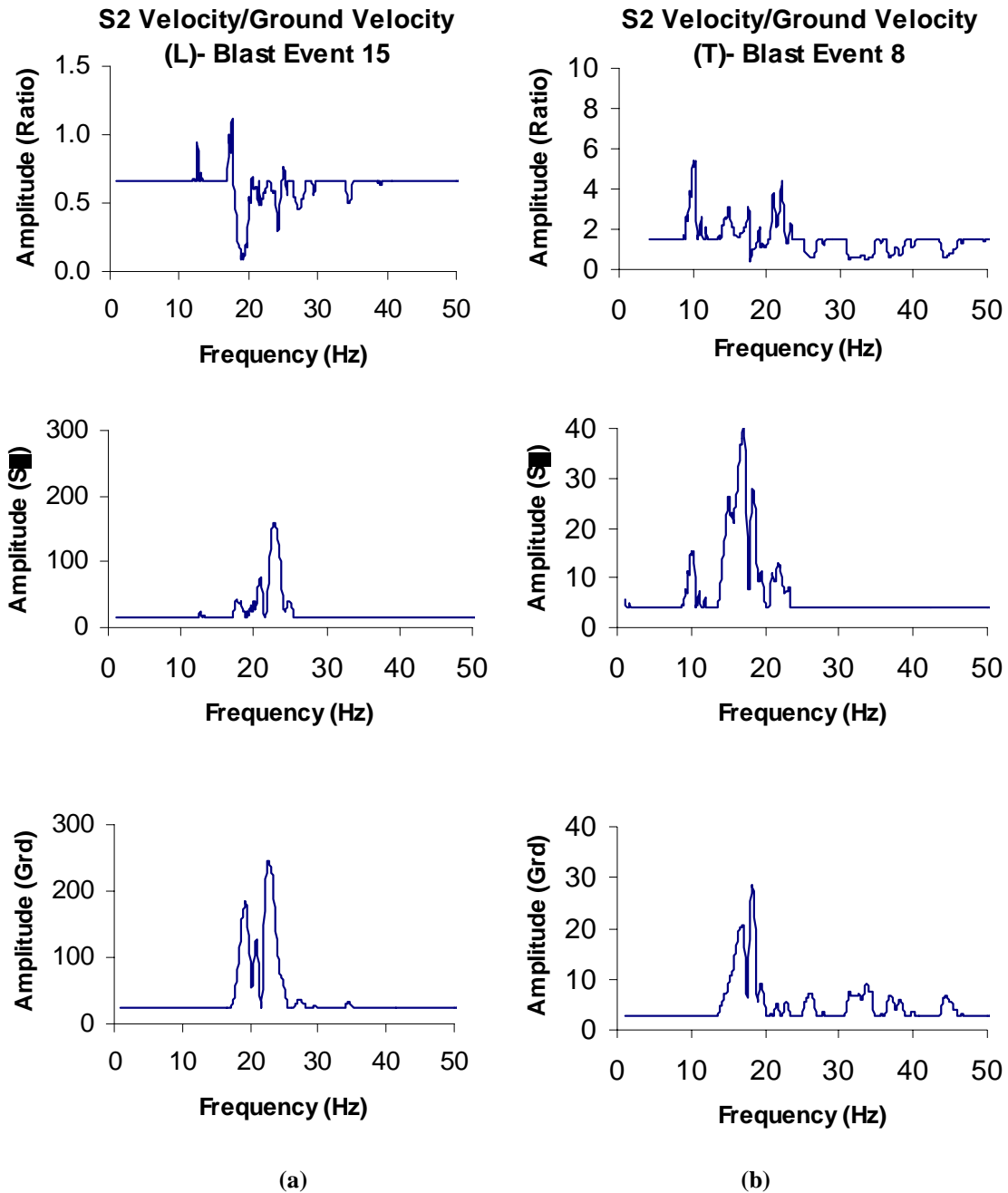
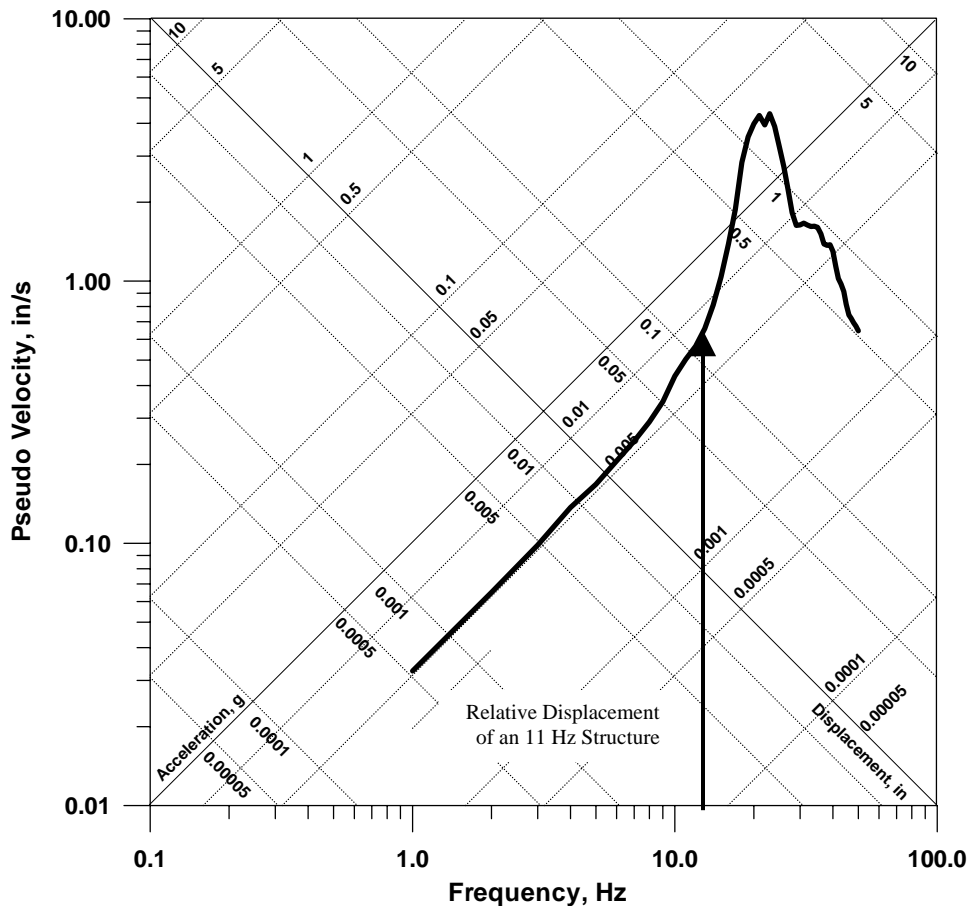


Figure 2.11 Spectra of  $S_2$  velocity frequency Fourier transform with ground velocity frequency Fourier transform ratio (top),  $S_2$  velocity frequency Fourier transform (middle) and ground velocity frequency Fourier transform (bottom) (a) for longitudinal response, Blast 15 and (b) for transverse response, Blast 8

Single degree of freedom (SDOF) responses were calculated for all longitudinal and vertical ground motions produced by the blast events to estimate relative displacements of the structure. A pseudo velocity response spectrum curve is generated from SDOF analyses that represent the response of structures, of varying natural frequencies, to the same ground motion (Dowding 1996).

The SDOF response spectrum for the longitudinal ground motion produced by blast event 15 is shown in Figure 2.12. A damping coefficient of 5% was assumed in determining the response spectra for all of the ground motions analyzed, based on average values from previous studies (Dowding, 1996). The estimated natural frequency of the superstructure is 10 to 12 Hz, while the estimated natural frequency of the wall is 17 to 18 Hz. Therefore the calculated displacement of the structure relative to the ground at 11 Hz is 7000  $\mu\text{in}$  (178  $\mu\text{m}$ ), while the average displacement between 10 and 18 Hz. is 13620  $\mu\text{in}$  (346  $\mu\text{m}$ ) via the SDOF spectrum in Figure 2.12.



**Figure 2.12 Single degree of freedom response spectrum of longitudinal ground motion produced by Blast 15 on July 22<sup>nd</sup>, 2002 at 11:49 AM, showing the estimated relative displacement of an 11 Hz structure**

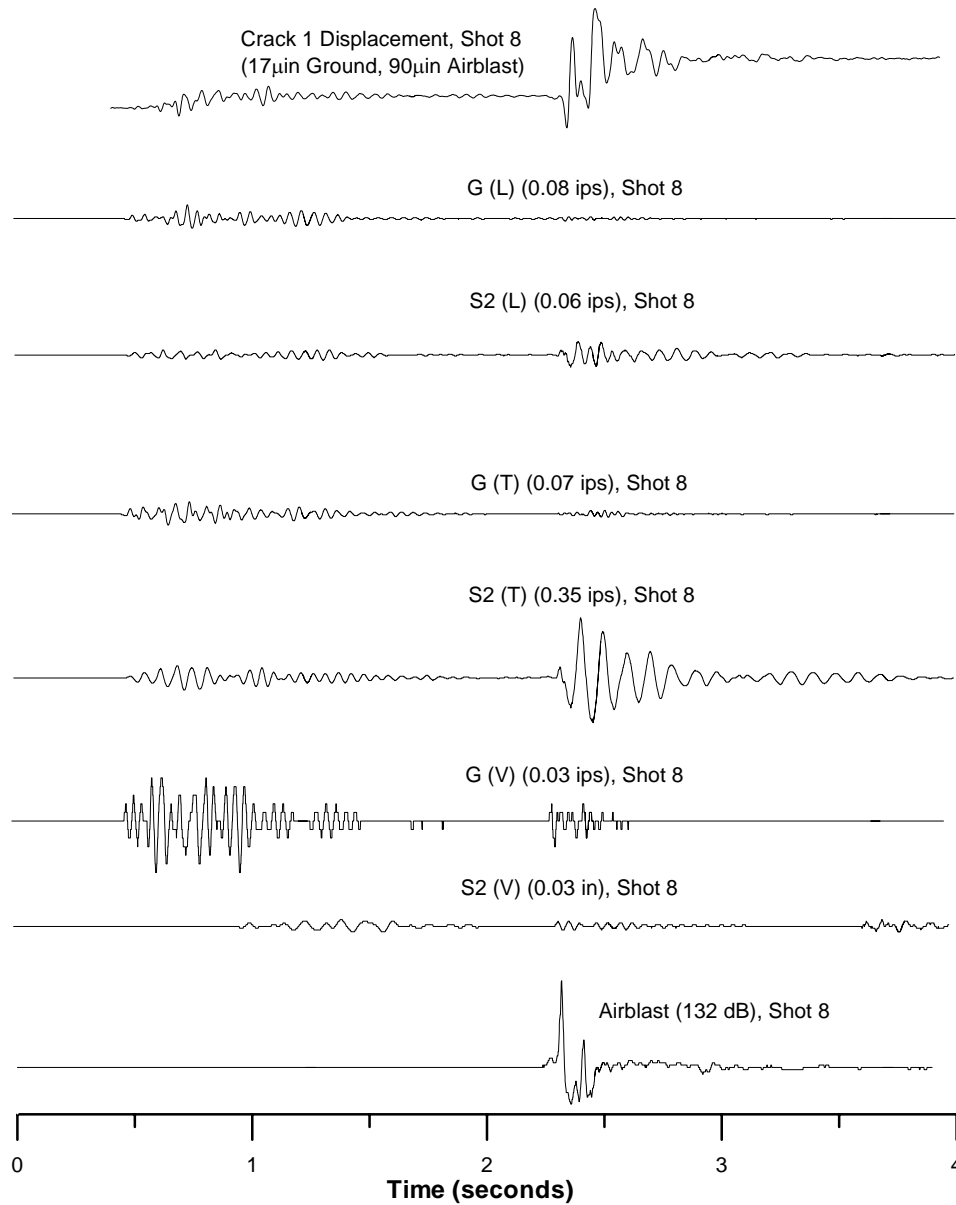
Figure 2.13 shows longitudinal, transverse, and vertical time histories of excitation ground motion,  $S_2$  structural response, airblast, and the corresponding addition ceiling Crack 1 response associated with blast 8 (June 10<sup>th</sup>, 2002). One of the inherent challenges in Connecticut was establishing correlation criteria for Crack 1, which lies in the ceiling of the apartment. The same analysis employed for the longitudinal responses and correlated to the bedroom wall Crack 3 in the longitudinal plane, was performed for all three axes of excitation and response, and correlated with Crack 1 response. Figure 2.11 (b) above shows the transverse FFT spectra for blast 8 and Figure 2.14 shows the longitudinal, transverse and vertical single degree of freedom response spectrum.

The response of Crack 1 illustrates the importance of airblast control. For many of the shots involved in this study there are two separate Crack 1 responses, one resulting from the relatively low amplitude vertical ground motions (compared to the longitudinal motions), and another resulting from the trailing airblast. Blasting at the quarry was roughly 2500 feet from the house, therefore the airblast arrives roughly 1.8 to 1.9 seconds after the ground motion, causing a completely independent displacement in Crack 1. Crack 3 did not respond with nearly the same vigor to airblast events as Crack 1. The quarry is situated such that the trailing air pressure impacts the long face of the structure, causing a significant transverse response. Figure 2.13 clearly shows the effect of this transverse motion on Crack 1.

The time history in Figure 2.13 shows the effect of a 132 dB airblast pressure intersecting the structure. The corresponding 90  $\mu$ m peak-to-peak Crack 1 displacement is the largest recorded single crack displacement for this project. Of the 24 blast events recorded, 11 of them produced airblast events over 115 dB, and 7 of them over 120 dB. Crack 1 was far more sensitive to these airblast events than Crack 3, and was typically more sensitive to them than the ground motion. The existence of large airblast overpressures is often a significant concern in mining and quarry operations.

Midway through the course of the project, it became apparent, however, that the methods of measuring structural displacement,  $S_2$  and  $S_1$  due to ground motions, do not accurately represent the true relative displacement of the ceiling, or of Crack 1 under these conditions. For the final six events, a transducer was placed next to Crack 1, in the center of the ceiling, to measure vertical mid-ceiling response, and obtain an out-of-plane

ceiling response. The correlation of displacements calculated from motions at this ceiling velocity transducer to Crack 1 displacement will be presented and evaluated later in the chapter.



**Figure 2.13 Time history of Crack 1 displacement from Blast event 8 on June 10<sup>th</sup>, 2002 at 11:49 AM compared to longitudinal, transverse, and vertical ground excitation, longitudinal, transverse, and vertical S<sub>2</sub> response, and air blast.**



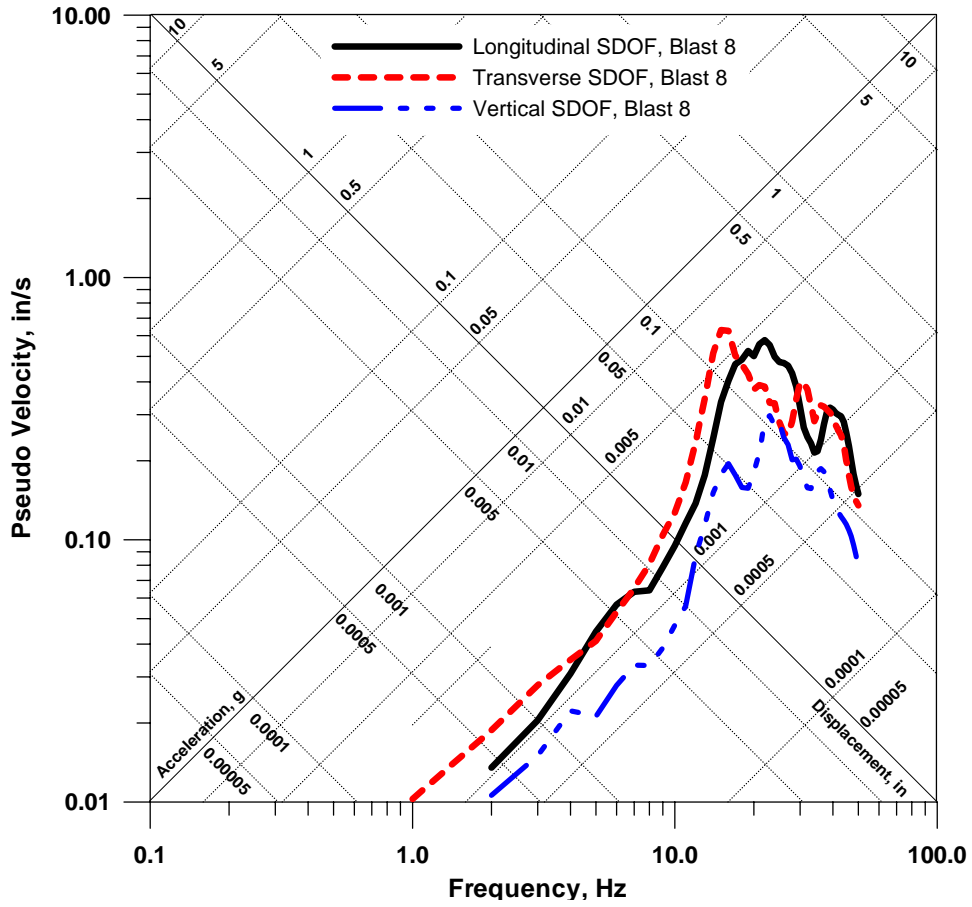


Figure 2.14 Single degree of freedom response spectrum of longitudinal, transverse, and vertical ground motion produced by Blast 8 on June 10<sup>th</sup>, 2002 at 11:49 AM

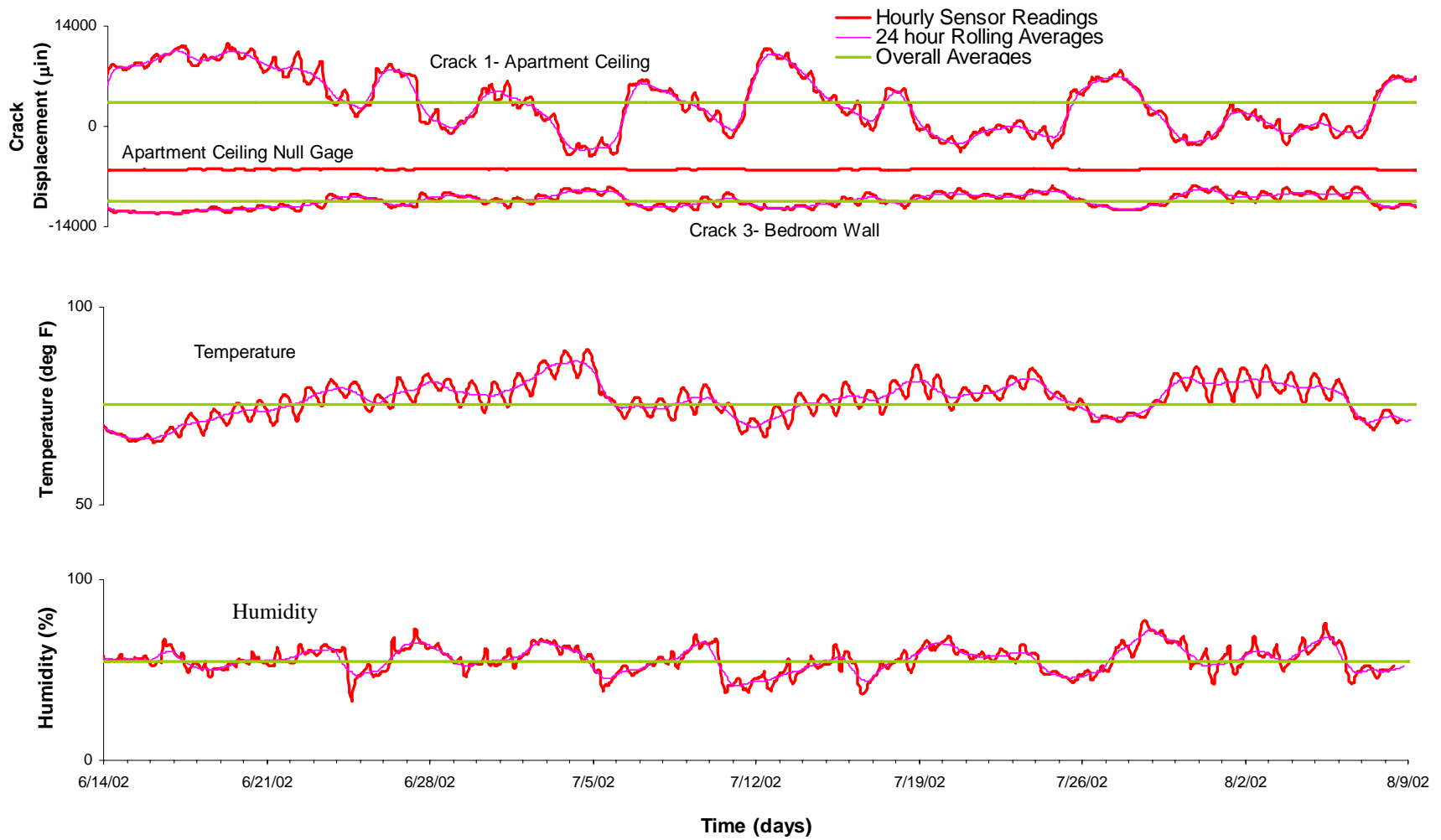
### **Crack Response to Environmental Effects**

Figure 2.15 compares the long-term response of the bedroom wall crack to the long-term fluctuation of temperature and humidity. Long-term crack displacement was measured hourly for the duration of the monitoring period, while temperature and humidity were measured every ten minutes and averaged to obtain one sample per hour. Some sharp changes are observed in the temperature, humidity and crack displacement during the monitoring period. Large, simultaneous changes in temperature and humidity, such as those on July 5<sup>th</sup>, July 12<sup>th</sup>, and July 24<sup>th</sup>, produce the largest changes in crack displacement.

A 24-hour rolling average of temperature, humidity and crack displacement were systematically calculated at each hourly measurement by averaging the data 12 hours before and 12 hours after (24 hours in total) each individual sample. See McKenna (2002) for details. Overall averages of crack displacement, temperature and humidity for the duration of the monitoring period are presented as horizontal solid lines in Figure 2.15.

Field measurements, 24-hour and overall averages are employed collectively to quantify micro-inch crack response to weather effects. Weather effects are analyzed for three different effects; frontal movements that change overall temperature and humidity for periods of several days to weeks, daily response to changes in average temperature and solar radiation, and extremes of unusual weather or other environmental effects (McKenna 2002). Table 2.3 lists average and maximum values for the frontal, daily, and total weather effects alongside values of crack response to maximum ground motions associated with quarry blasting to compare the difference in magnitude of environmentally induced and blasting induced crack response.

The frontal effect is defined as the deviation of a peak 24-hour average value from the overall average. In other words, between instances of a 24-hour average curve and overall average curve crossing, the frontal effect is measured as the peak, absolute deviation of the 24-hour average from the overall average. The daily effect is defined as the deviation of a peak field measurement from the corresponding 24-hour average. Between each crossing of the field measurement curve and the 24-hour average, the daily effect is measured as the peak, absolute deviation of the field curve from the 24-hour



**Figure 2.15 Long-term Crack 1, Crack 3, and null gage displacement, temperature, and humidity versus time**

average. The final descriptor, the total weather effect, is defined as the difference in the peak field measurement from the overall computed average. Between each crossing of the field measurement curve and the overall average the weather effect is computed as the peak, absolute value of the field measurement minus the overall average. The average and maximum values of these three effects on crack displacement, temperature and humidity are presented in Table 2.3.

	Temperature Change (DegF)	Humidity Change	Crack Displacement (μin)	Crack Displacement (μm)
<b>Crack Sensor 1- Apartment Ceiling</b>				
<b>Frontal Effect</b>				
Average deviation of 24-hour average from overall average	5	4	3346	85
Maximum deviation of 24-hour average from overall average	12	14	7283	185
<b>Daily effect</b>				
Average deviation of field measurement from 24-hour average	1.5	1.5	669	17
Maximum deviation of field measurement from 24-hour average	5	11	5748	146
<b>Weather Effect</b>				
Average deviation of field measurement from overall average	5	4	3543	90
Maximum deviation of field measurement from overall average	15	15	8307	211
<b>Blisting Effect</b>				
Typical vertical ground motion (PPV=0.10 ips)	-	-	35	0.89
Maximum ground motion (PPV= 0.345 ips)	-	-	90	2.89

	Temperature Change (DegF)	Humidity Change	Crack Displacement (μin)	Crack Displacement (μm)
<b>Crack Sensor 3- Bedroom Wall</b>				
<b>Frontal Effect</b>				
Average deviation of 24-hour average from overall average	4	6	669	17
Maximum deviation of 24-hour average from overall average	13	23	1693	43
<b>Daily effect</b>				
Average deviation of field measurement from 24-hour average	2	2	217	5.5
Maximum deviation of field measurement from 24-hour average	6.5	15	1142	29
<b>Weather Effect</b>				
Average deviation of field measurement from overall average	4	6	748	19
Maximum deviation of field measurement from overall average	14	26	2165	55
<b>Blisting Effect</b>				
Typical Radial ground motion (PPV=0.10 ips)	-	-	30	0.74
Maximum ground motion (PPV= 0.345 ips)	-	-	72	1.82

**Table 2.3 Computed crack displacements due to long-term weather phenomena, typical ground motion and maximum ground motion in Connecticut**

Figure 2.16 displays in detail the relatively dominant effect of weather phenomena on crack displacement versus that of blast-induced ground excitation. This Figure shows the response of Crack 1 in the apartment over two full days, June 10<sup>th</sup> and 11<sup>th</sup>, 2002, during which significant blast event #8, time histories of which are shown in Figure 2.17, occurred with a peak particle velocity of 0.08 ips and airblast of 132 dB. This daily response is compared with the response during the entire period of observation. The blast effect on Crack 1 is circled and labeled on Figure 2.16.

Blast event 8 and its resultant displacement time history for Crack 1 are unique to those previously analyzed for two reasons. First, the 132 dB airblast that occurs approximately 2 ½ seconds into the monitoring period produces a peak crack displacement of 180 micro-inches, far exceeding the 35 micro-inch displacement induced by the ground motion. Second, this airblast also produces what appears to be a 90 micro-inch offset during the 8-second record. However, the magnified hourly crack response in Figure 2.16 shows that the daily weather phenomena on June 10<sup>th</sup> produced a maximum crack displacement in the apartment ceiling of 1500 micro-inches, which is an order of magnitude greater than that produced by the event. This crack, even if it were offset by 90 micro-inches, was returned to its pre-blast displacement less than four hours later as a result of the temperature and humidity change.

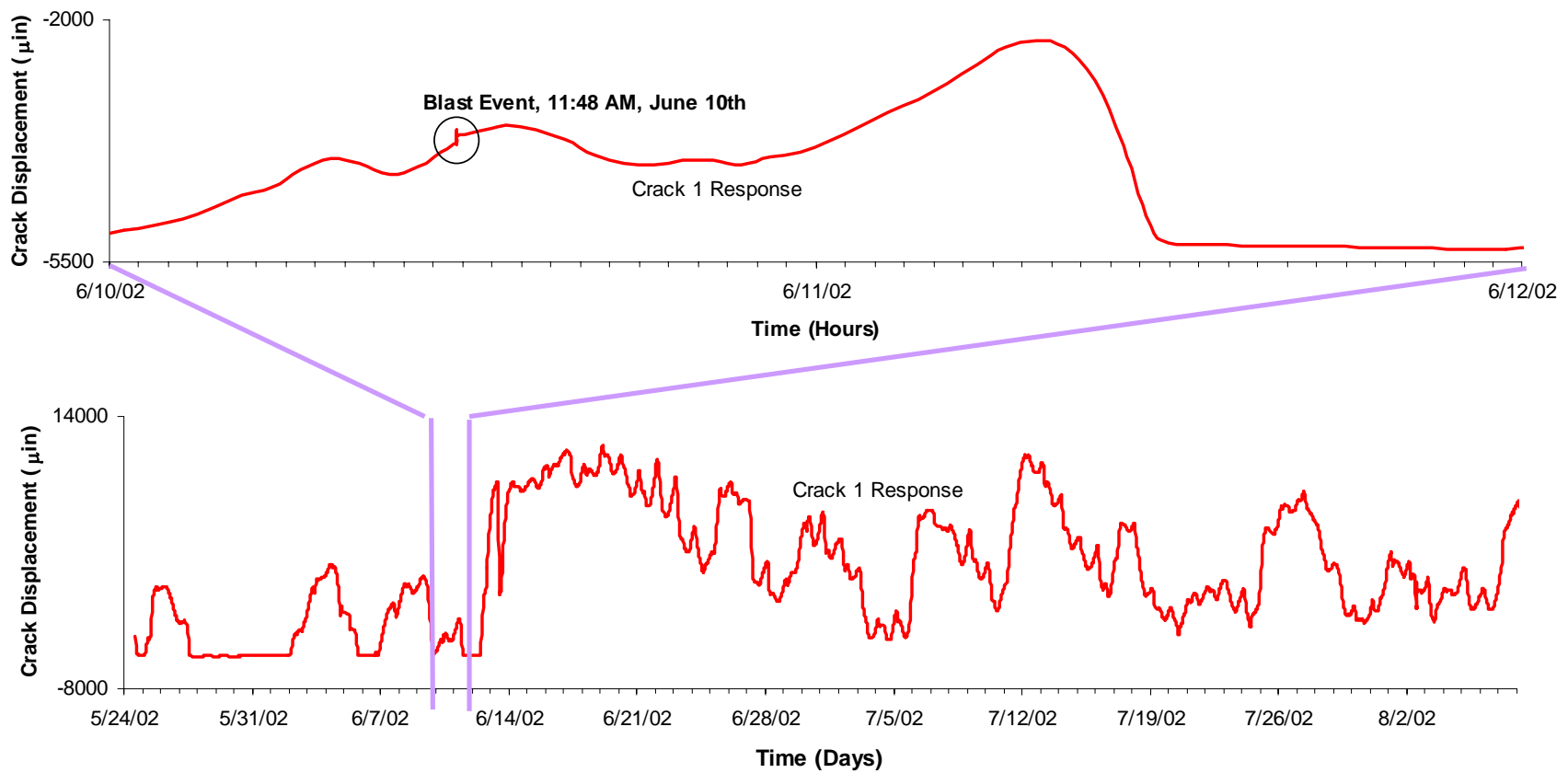


Figure 2.16 Long-term Crack 1 displacement magnified to show 48-hours of response data surrounding blast event 8 on June 10<sup>th</sup>, 2002

### **Comparison of Computed and Measured Crack Displacements**

The maximum measured crack displacement produced by each shot is compared in the longitudinal and vertical directions in Table 2.4 (a) and (b) to various computed wall displacements based on structure response and peak ground motions. Structure/wall displacements were computed using a number of methods such as the integration of velocity time histories and the single degree of freedom response spectrum method. Response of bedroom wall Crack 3 correlated best with ground motions and displacements in the longitudinal direction, and displaced with zero correlation to airblast. Crack 3 is on an interior wall, so this poor correlation is expected. Apartment ceiling Crack 1, however, responded better to certain ground motion and structural displacements in various axial directions. Correlations of Crack 3 responses are presented in Figure 2.17. Correlations of Crack 1 displacement in the longitudinal direction are presented in Figure 2.18, transverse in 2.19, and vertical in 2.20. In addition, Figure 2.20 presents the correlation between Crack 1 displacement and measured airblast. Details pertaining to the aforementioned methods in the computation of structural displacement are presented below.

Date Shot #	Relative Displacement of Structure, $\delta$ , by Method ( $\mu\text{in}$ )										Velocities (in/sec)			Crack Displacements	
	Integration of Velocities							SDOF Method			Peak Particle Velocity	Upper Structure ( $S_2$ ) <sub>max</sub>	Vertical Midceiling ( $S_{\text{mid}}$ ) <sub>max</sub>	Crack 1 ( $\mu\text{in}$ )	Crack 3 ( $\mu\text{in}$ )
	Vertical ( $S_{\text{midceiling}}$ )	Vertical ( $S_2$ ) <sub>max</sub> ( $S_2-S_1$ ) <sub>max</sub>	Vertical (G) <sub>max</sub> ( $S_2-G$ ) <sub>max</sub>	Transverse ( $S_2$ ) <sub>max</sub> ( $S_2-S_1$ ) <sub>max</sub>	Transverse (G) <sub>max</sub> ( $S_2-G$ ) <sub>max</sub>	Longitud. ( $S_2$ ) <sub>max</sub> ( $S_2-S_1$ ) <sub>max</sub>	Longitud. (G) <sub>max</sub> ( $S_2-G$ ) <sub>max</sub>	Vertical	Transverse	Longitudinal					
								$f_n = 12\text{Hz}$ $f_n = 10-20\text{Hz}$	$f_n = 12\text{Hz}$ $f_n = 10-20\text{Hz}$	$f_n = 12\text{Hz}$ $f_n = 10-20\text{Hz}$					
5/20/2002 1	N/A	1000	460	2600	1690	1600	1460	3243	8816	6529	0.180 (L) 0.165 (T) 0.055 (V)	0.160 (L) 0.240 (T) 0.115 (V)	N/A	41	N/A
5/22/2002 2	N/A	300	210	1200	410	400	260	1422	2430	1221	0.030 0.050 0.025	0.035 0.090 0.040	N/A	14	N/A
5/22/2002 3	N/A	600	80	700	400	300	500	300	1600	2097	0.065 0.050 0.020	0.035 0.065 0.030	N/A	19	N/A
5/23/2002 4	N/A	1300	600	2300	1100	600	800	1800	4500	3571	0.105 0.150 0.075	0.070 0.160 0.180	N/A	39	N/A
5/23/2002 5	N/A	900	700	2000	900	900	1400	2000	3000	3200	0.125 0.085 0.065	0.060 0.140 0.070	N/A	44	N/A
5/23/2002 6	N/A	700	560	2000	780	900	780	2000	3990	4400	0.100 0.085 0.055	0.080 0.135 0.060	N/A	38	N/A
6/3/2002 7	N/A	1150	300	1100	660	500	640	1300	3561	2000	0.080 0.080 0.040	0.045 0.110 0.070	N/A	30	N/A
6/10/2002 8	N/A	600	300	4700	720	1000	520	1154	3132	1827	0.080 0.070 0.030	0.030 0.035 0.030	N/A	17	N/A
6/17/2002 9	N/A	660	380	2520	1000	620	1000	2300	4852	3553	0.145 0.125 0.055	0.070 0.195 0.080	N/A	29	25
6/18/2002 10	N/A	830	310	1270	900	400	600	544	3300	2014	0.080 0.110 0.030	0.040 0.125 0.060	N/A	35	19
6/26/2002 11	N/A	510	410	790	500	360	600	1737	2578	1800	0.080 0.065 0.050	0.035 0.080 0.050	N/A	25	15
7/9/2002 12	N/A	1390	910	1910	120	1170	1000	2600	3421	3185	0.125 0.160 0.160	0.135 0.185 0.190	N/A	41	31

**Table 2.4 (a) Traditional structure response and ground motion controls with Crack 1 and 3 displacements for blast events 1 through 12 in Connecticut**



Date Shot #	Relative Displacement of Structure, $\delta$ , by Method ( $\mu\text{in}$ )								Velocities (in/sec)			Crack Displacements			
	Integration of Velocities						SDOF Method			Peak Particle Velocity	Upper Structure $(S_2)_{\text{max}}$	Vertical Midceiling $(S_{\text{mid}})_{\text{max}}$	Crack 1 ( $\mu\text{in}$ )	Crack 3 ( $\mu\text{in}$ )	
	Vertical $(S_2)_{\text{max}}$	Vertical $(G)_{\text{max}}$	Transverse $(S_2)_{\text{max}}$	Transverse $(G)_{\text{max}}$	Longitud. $(S_2)_{\text{max}}$	Longitud. $(G)_{\text{max}}$	Vertical $f_n = 12\text{Hz}$	Transverse $f_n = 12\text{Hz}$	Longitudinal $f_n = 12\text{Hz}$						
	$(S_2-S_1)_{\text{max}}$	$(S_2-G)_{\text{max}}$	$(S_2-S_1)_{\text{max}}$	$(S_2-G)_{\text{max}}$	$(S_2-S_1)_{\text{max}}$	$(S_2-G)_{\text{max}}$	$f_n = 10-20\text{Hz}$	$f_n = 10-20\text{Hz}$	$f_n = 10-20\text{Hz}$						
7/12/2002 13	N/A	750	460	1480	700	570	700	976	1734	2236	0.080 (L) 0.065 (T) 0.045 (V)	0.050 (L) 0.085 (T) 0.060 (V)	N/A	20	13
7/16/2002 14	N/A	1810	640	2280	1600	1290	1500	2614	6839	5365	0.190 0.180 0.095	0.145 0.230 0.230	N/A	60	59
7/22/2002 15	N/A	1670	660	2290	1970	1130	2540	1965	6709	7494	0.345 0.255 0.095	0.165 0.220 0.230	N/A	90	50
7/26/2002 16	N/A	980	670	1540	1180	1100	910	1411	4441	3170	0.140 0.140 0.070	0.145 0.180 0.130	N/A	50	28
7/30/2002 17	N/A	290	370	730	510	510	650	1078	1632	1447	0.075 0.055 0.035	0.050 0.080 0.040	N/A	23	12
8/2/2002 18	N/A	1390	840	3000	1430	1010	1510	2338	6481	4682	0.195 0.155 0.115	0.130 0.225 0.180	N/A	80	37
8/15/2002 19	3490	3080	660	1880	880	1230	1500	2051	3859	3671	0.215 0.130 0.085	0.155 0.170 0.180	0.460	38	33
8/23/2002 20	4420	2920	870	1640	1130	1180	1950	2125	4000	6433	0.285 0.140 0.095	0.115 0.175 0.320	0.590	47	38
9/3/2002 21	2000	5080	730	2220	1620	920	2220	4334	9721	7020	0.280 0.210 0.100	0.100 0.195 0.480	0.250	70	60
9/10/2002 22	3290	2440	940	1360	930	810	1880	3290	5017	6230	0.270 0.150 0.135	0.105 0.155 0.320	0.445	60	34
9/17/2002 23	2820	3170	720	2980	1190	1560	1830	4253	5000	5366	0.280 0.175 0.095	0.185 0.235 0.330	0.385	57	48
9/20/2002 24	1470	2240	360	1190	470	720	920	1328	1885	2400	0.155 0.060 0.055	0.090 0.105 0.210	0.220	27	28

Table 2.4 (b) Traditional structure response and ground motion controls with Crack 1 and 3 displacements for blast events 13 through 24 in Connecticut

### Velocity Time History Integration

Displacement time histories are calculated by integrating velocity time histories. By subtracting time correlated ( $\pm 0.001$  sec) pairs of integrated time histories, a relative displacement time history is produced. This difference was calculated for two pairs of displacement time histories: 1) upper corner response,  $S_2$ , minus lower corner response,  $S_1$ , and 2)  $S_2$  minus ground motion,  $G$ . Comparisons between measured crack displacements and the peak values of these resulting displacement time histories,  $(S_2 - S_1)_{\max}$  and  $(S_2 - G)_{\max}$ , are presented graphically in Figures 2.17 through 2.20.

Displacements were also estimated from the integrated ground velocity time histories exclusively. The comparison between measured crack displacement and the peak of these displacement time histories,  $S_{2\max}$  and  $G_{\max}$ , are presented in Figures 2.17 through 2.20.

### Single Degree of Freedom Response Spectrum Method

As described earlier in the chapter, by analyzing SDOF response spectra of blast-induced ground motions, relative displacements can be estimated for structures of different dominant frequencies. Two approaches were employed in estimating relative displacements via this method. The first was to run an SDOF at 11 Hz to calculate the relative displacement associated with the estimated natural frequency of the structure. These computed relative displacements are then compared with measured crack displacements and shown in Figures 2.17 through 2.20.

The second approach in estimating relative displacements based on the SDOF method was to average the relative displacements of structures with natural frequencies of 10-15 HZ found most likely from previous studies. Comparisons between the measured crack displacements and these average relative displacements are also presented in Figures 2.17 through 2.20.

### Correlations to Ground and Structural Motions

The square of the correlation coefficient,  $R^2$ , is employed to describe correlations. Microsoft Excel defines  $R^2$  as the square of the Pearson product moment correlation coefficient, which is the proportion of the variance in the value of measured crack response, depending on the variance in the estimator.

In previous case studies (McKenna, 2002), the best correlations with measured crack displacement were those produced by structural motion,  $S_2$  and  $S_2-S_1$  for single-story structures. These measured responses correlate the best because they give the most accurate representation of in-plane wall strains and relative wall displacements.  $S_2-G$  is less accurate than  $S_2-S_1$  as there is some transfer of energy between the ground and the house.

$S_2-S_1$  relative displacement calculations in this case, however, are less applicable because of the large construction and material differences between the lower concrete masonry unit garage walls and the upper wood frame apartment walls. Thus, values calculated for  $S_2-S_1$  response in the Connecticut house represent relative displacements of wall components of two different construction techniques and the mode shapes and response cannot be properly estimated without a transducer,  $S_{1.5}$ , at the interface between the upper and lower stories. In addition, structural response data were recorded only in the apartment/garage addition and Crack 3 is located on an interior wall in the bedroom at the east, opposite end of the main unit of the house. This portion of the house is founded differently. Therefore it will experience an  $S_2$  response different than that measured above the garage. Were these situations to be corrected ( $S_1$  recorded at the base of the apartment wall and structural response recorded in the bedroom of the main unit), these correlations would be expected to improve significantly.

Crack 3 displacement responds best to ground and structural motions in the longitudinal direction, and does not correlate at all to airblasts, as shown in Figure 2.17. This result is expected, as Crack 3 lies in the longitudinal plane and sits on an interior wall, thus negating the relative effect of airblasts. The highest correlations are those that involve peak particle velocity and integrated ground velocity. The best structural correlations are the single degree of freedom response spectra relative displacement calculations.

Crack 1 displacement correlates with both longitudinal and transverse motions almost equally; being highly responsive, it is most likely to give the appearance of responding to axial motions that produce significantly greater particle velocities. Crack 1 displacements are also divided into two separate categories; those resulting from ground motion, and those resulting from airblasts. The correlation between airblast-induced

Crack 1 displacement and airblast-induced transverse  $S_2$  response (also divided into two categories), is very good,  $R^2=0.94$ . The correlation between airblast-induced Crack 1 displacement and airblast decibel level is also quite good,  $R^2=0.82$ .

The highest correlation (0.96) for Crack 1 response occurs with the summation of integrated velocities,  $S_2+S_{\text{midceiling}}$  shown in Figure 2.21 in the vertical direction. Figure 2.22 shows time histories of Crack 1 displacement, Crack 3 displacement, longitudinal, transverse, and vertical particle velocity, vertical ground displacement, vertical  $S_2$  velocity and displacement,  $S_{\text{midceiling}}$  velocity and displacement, vertical  $S_2+ S_{\text{midceiling}}$  relative displacement and airblast for Shot 20. The midceiling responses were obtained only for the final six shots. Figure 2.22 shows time histories for blast event 20, and includes ground velocities in all three directions, ground displacements in the long and vertical directions, upper corner structure ( $S_2$ ) velocity and displacement in the long and vertical directions, and vertical midceiling ( $S_{\text{midceiling}}$ ) velocity and displacement. The midceiling response during this event is greater than that at the corner ( $S_2$ ), and it can be seen that Crack 1 displaces more in tandem with the  $S_2+S_{\text{midceiling}}$  response than to any other time history. Thus, a continuing displacement at approximately the 1-second mark is seen for both the measured crack displacement and that calculated as  $S_2+S_{\text{midceiling}}$ .

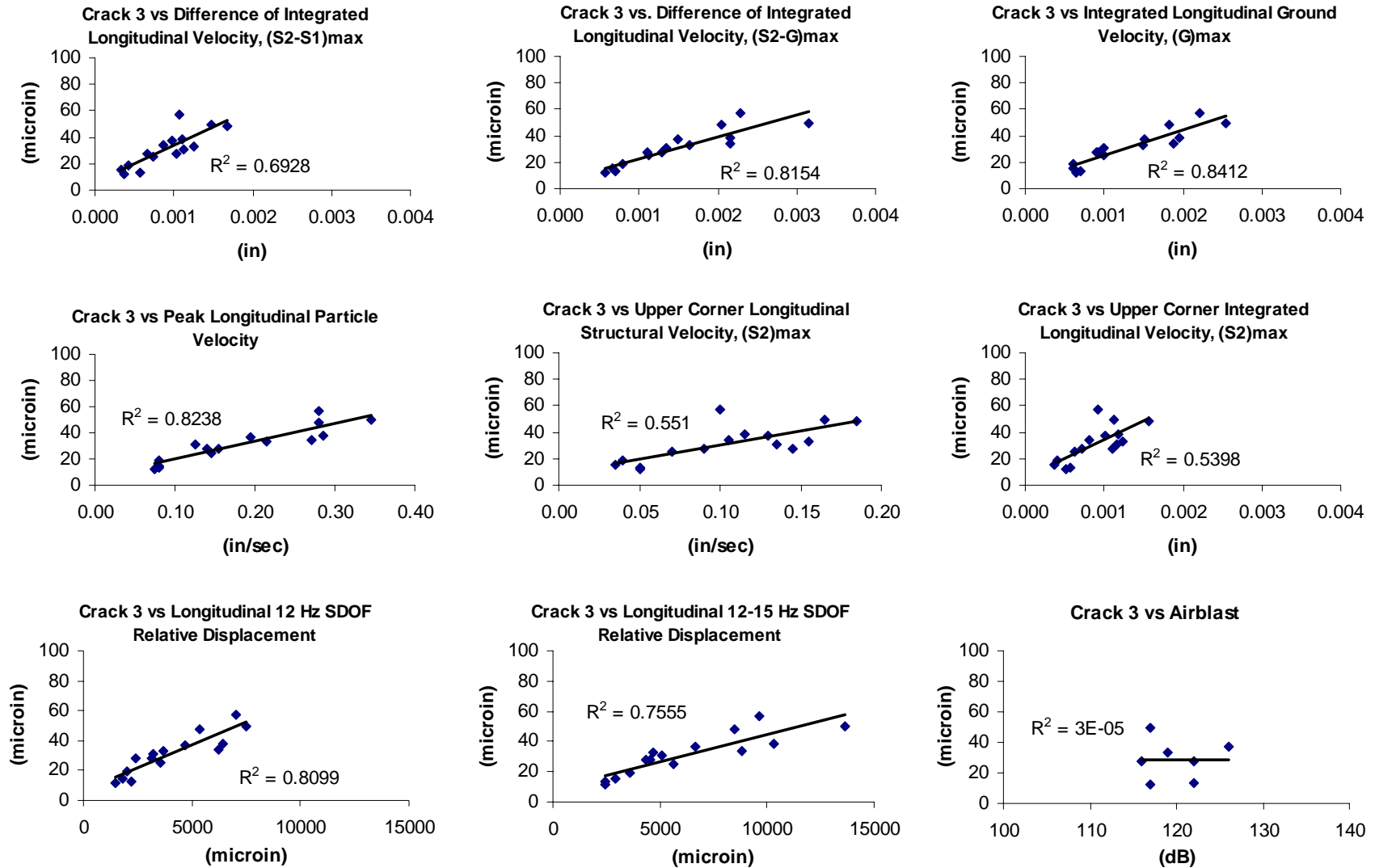
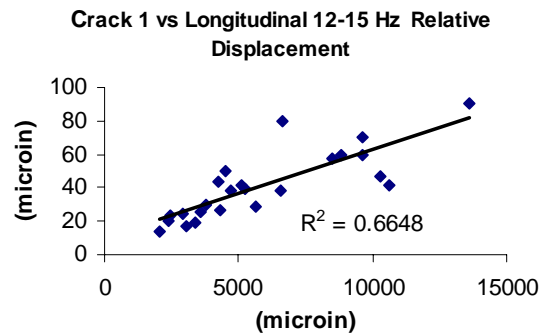
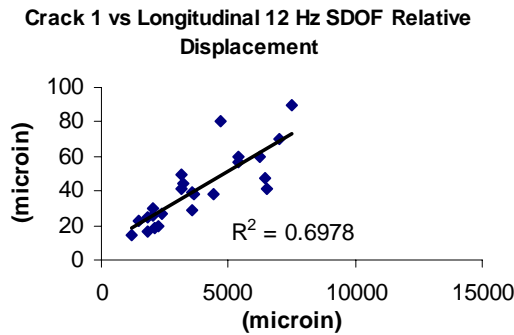
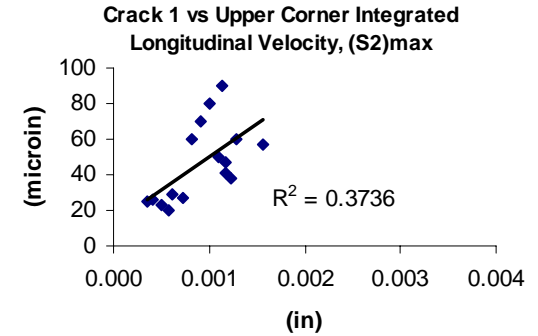
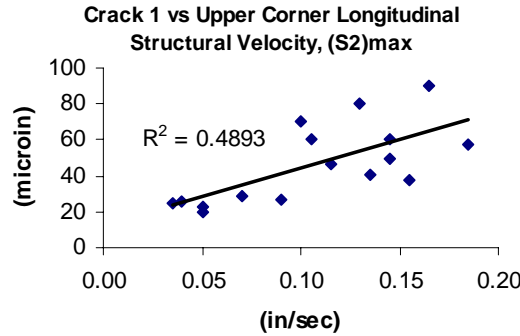
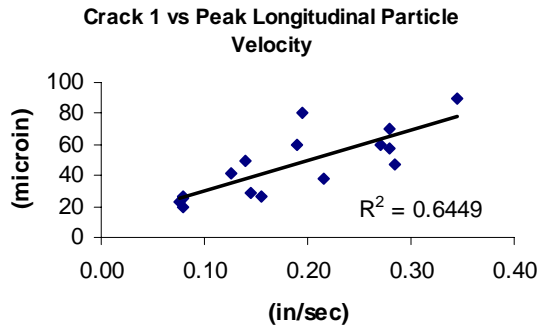
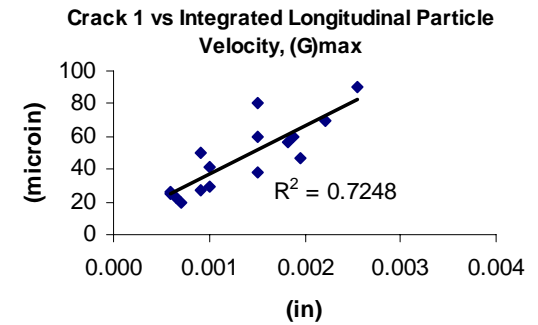
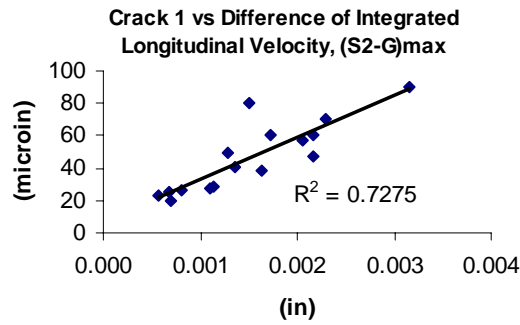
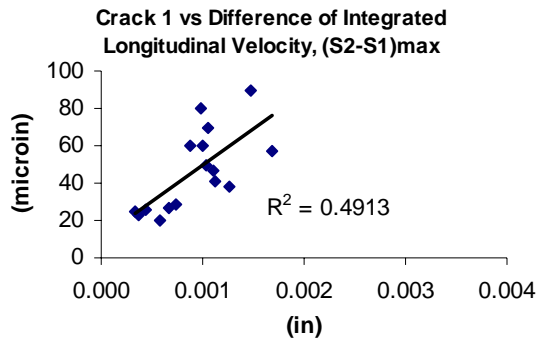


Figure 2.17 Correlations between measured Crack 3 displacement and longitudinal ground motions, relative structural displacements, and airblast



**Figure 2.18 Correlations between measured Crack 1 displacement and longitudinal ground motions and relative structural displacements**

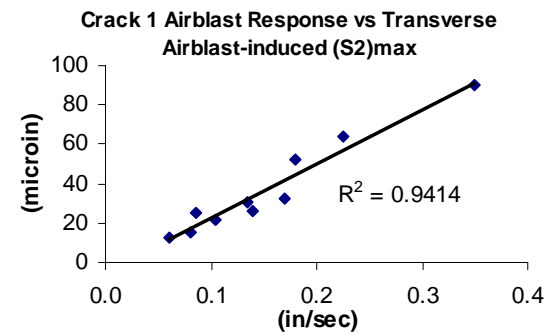
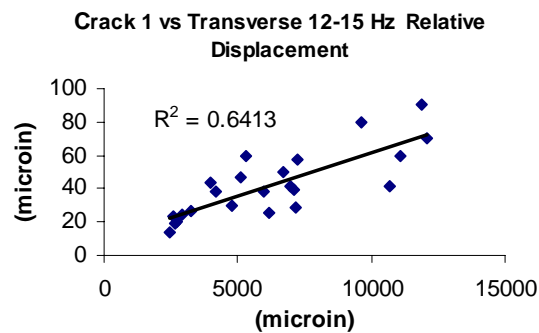
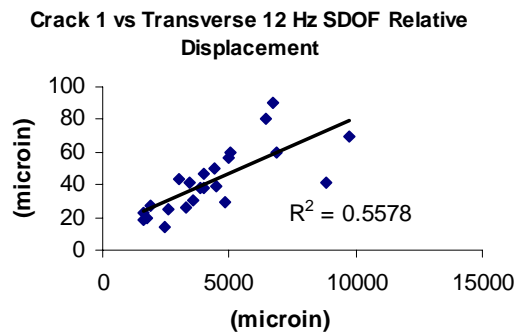
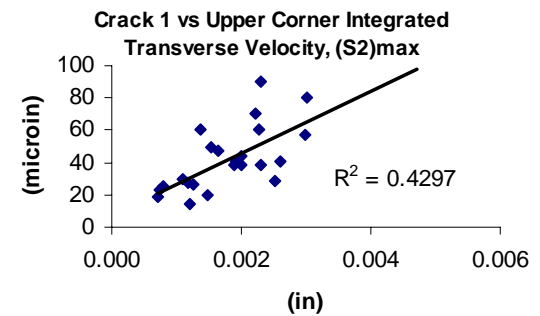
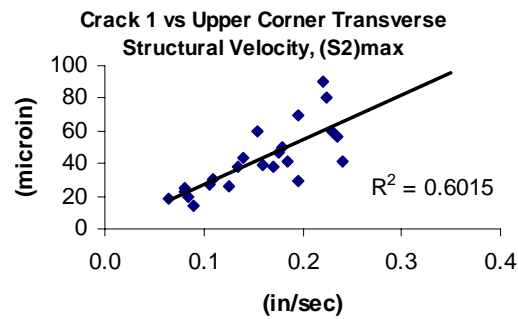
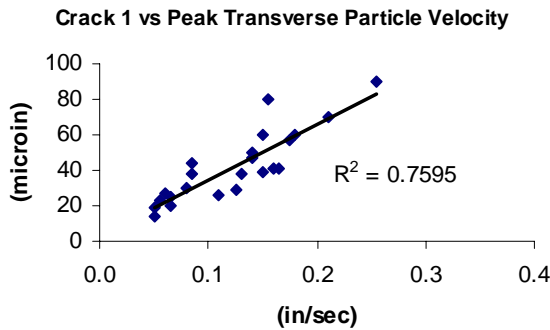
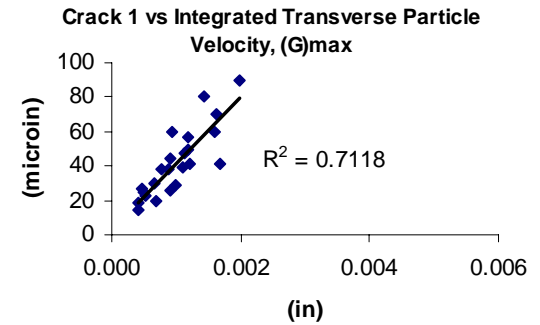
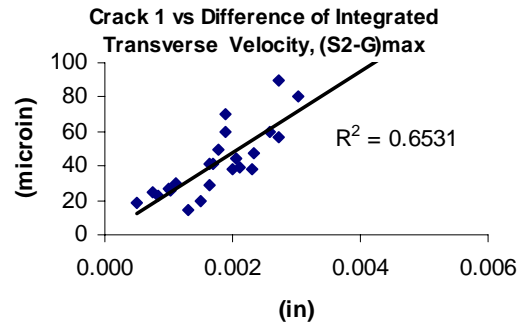
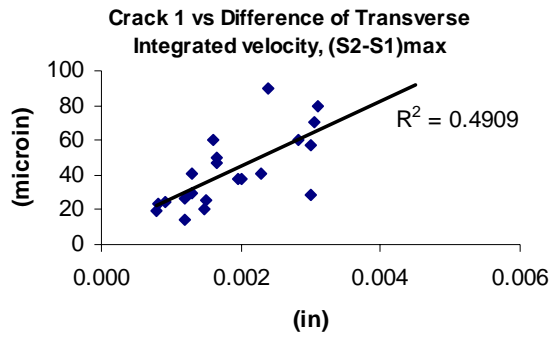


Figure 2.19 Correlations between measured Crack 1 displacement and transverse ground motions, relative structural displacements, and Crack 1 airblast response versus airblast induced upper structure response

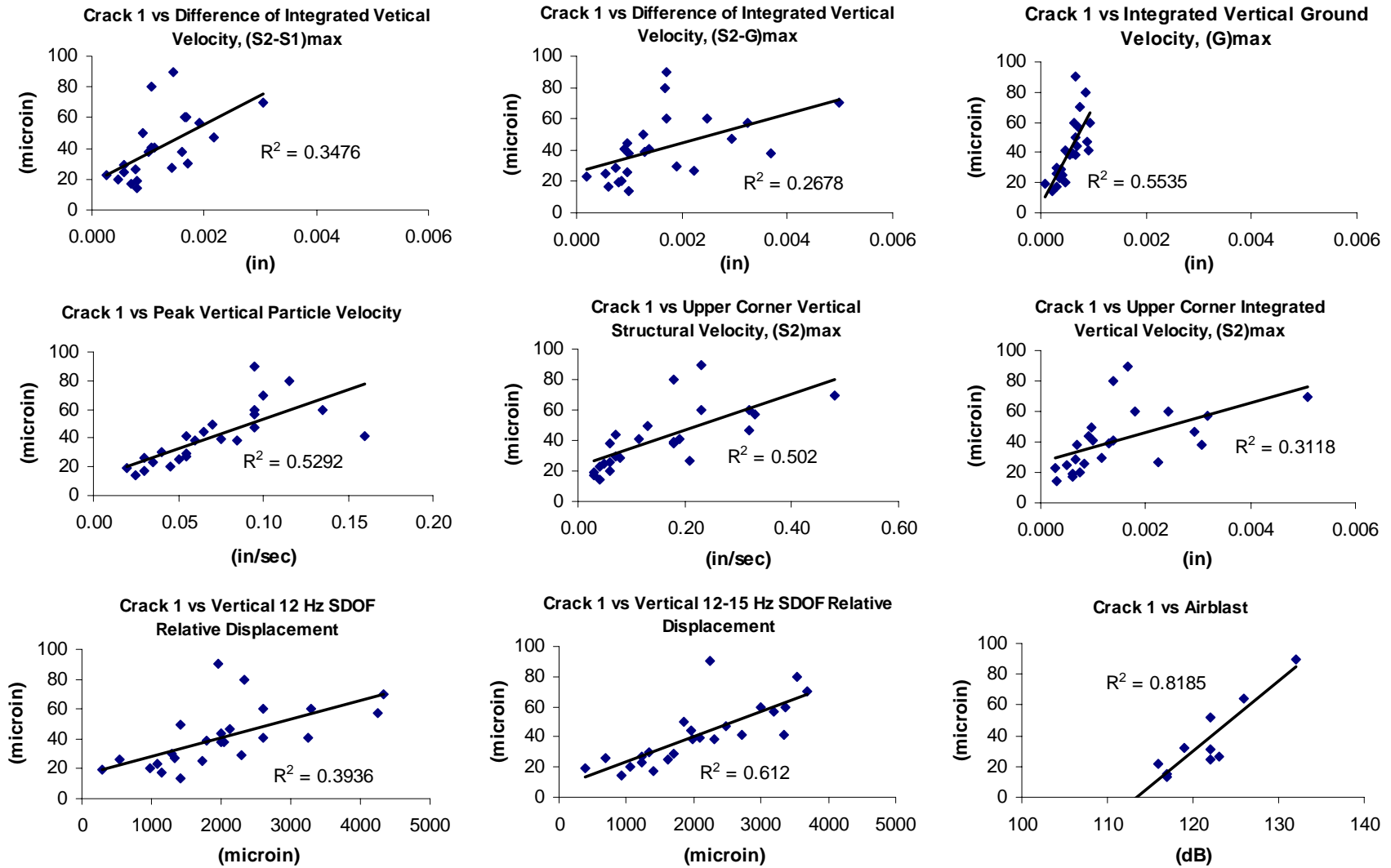
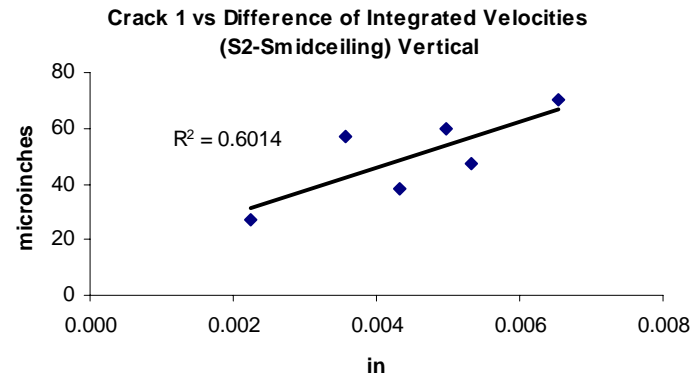
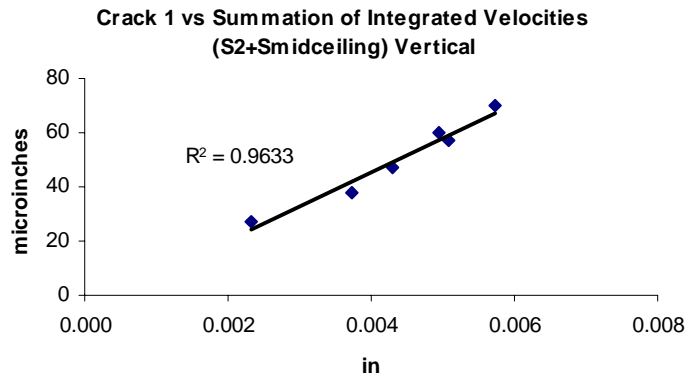
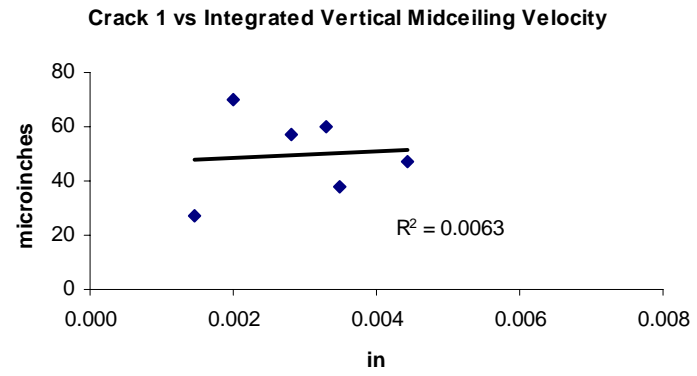
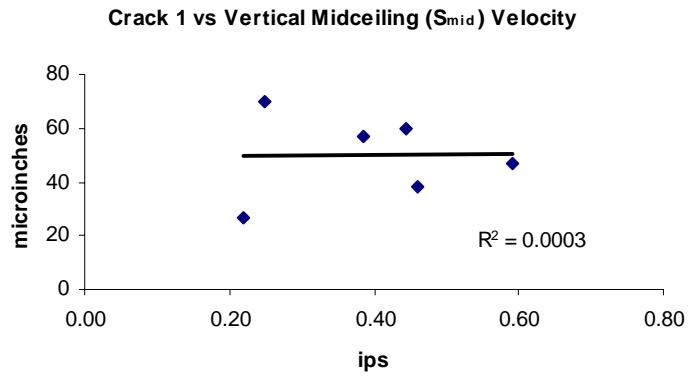
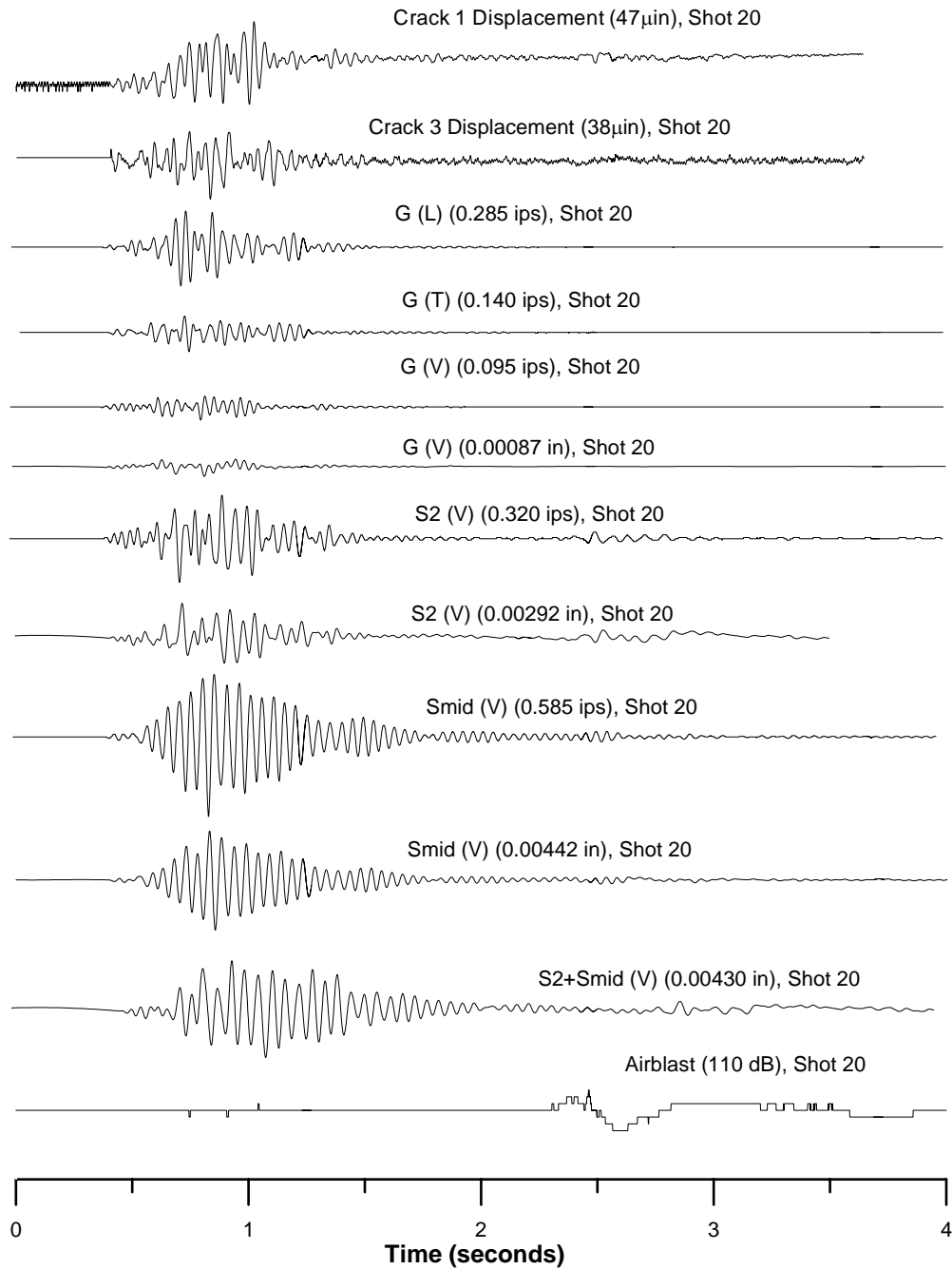


Figure 2.20 Correlations between measured Crack 1 displacement and vertical ground motions, relative structural displacements, and airblast





**Figure 2.21 Correlations between measured Crack 1 displacement and vertical structural and midceiling velocity and displacements**



**Figure 2.22 Time histories of Crack 1 and 3 displacements from blast event 20 on August 23<sup>rd</sup>, 2002 at 11:49 AM compared to longitudinal, transverse, and vertical ground velocity, vertical ground displacement, S<sub>2</sub> response velocity and displacement response, S<sub>midceiling</sub> velocity and displacement response, (S<sub>2</sub>+S<sub>midceiling</sub>) relative displacement, and air blast.**

## *Chapter 3*

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### *Effect of Blast Design on Crack Displacement*

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#### *Introduction*

Two principal factors determine peak particle velocities from blasting; 1) maximum charge weight detonated per delay, or the quantity of energy released into the ground at any moment, and 2) the absolute distance between the blast and the target. These two parameters are normalized by dividing the distance by the square root of the charge weight per delay (ft/lbs<sup>1/2</sup>). The smaller the scaled distance factor, the larger the ratio of explosive energy to distance and the “stronger” the shot. There are, however, many other secondary factors that have significant effects on peak particle velocity, structural response and crack displacements in target structures. These factors include propagation velocity and attenuation characteristics of the soil, borehole layout and timing patterns, the distance between charges at the source, the direction and orientation in which the detonation progresses, and the coupling of the released energy with the transmitting soil.

This chapter describes differences in structural and crack responses that resulted from blasts at similar scaled distance or expected particle velocities. Interest in this investigation stems from the results of a previous study (Aimone, 2000) that involved measurement of structural and ground responses at the Stiles Road quarry. This study concluded with the following suggestions for modification of blasting practices:

increased stem length, increased front row burden, minimum 25 millisecond delay time per hole, elimination of base primers, initiation of blast events from north to south for benches on north high walls, and design of blasts on south faces. Since the first study, the ACM system has been developed and additional house response measurement techniques have been adopted, both of which allow more efficient measurement. Installation of the ACM system and additional velocity transducers has allowed measurement of the effects of some of these blast design changes on house and crack response. This chapter describes these effects.

This chapter will focus on 4 primary components of blast design that have significant effects on resulting crack displacements in structures. These four components are borehole and free face geometry, designs that produce significant ground motions versus significant airblasts, blasts at different dominant frequencies, and shot timing and overlapping delay times which result in differences in the number of significant pulses.

Four pairs of blast events and their respective ground, structure and crack responses at the Connecticut structure described in Chapter 2 will be compared to describe the effects of variable blast designs. Table 3.1 summarizes the pertinent information associated with the following events comparing differences in:

- Face geometry (Shot 1, narrow V, single face versus Shot 9, wide V, two face)
- Stemming depth (Shot 8, shallow stemming with 132 dB air overpressure versus Shot 15, 0.345 ips ground motion)
- Frequency effect (Shot 14, 19 Hz versus Shot 22, 27 Hz in the longitudinal direction)
- Total shot time and number of boreholes (Shot 18, 436ms total shot time, 59 holes versus Shot 19, 215ms total shot time, 37 total holes)

Figure 3.1 is an aerial photograph of the quarry and the surrounding area with the location of the Connecticut house within the smaller circle, and the location of the blasts in the larger circle to the south. Figure 3.2 is the USGS topographic relief map of the same area shown in Figure 3.1 with the same location identification and Figure 3.3 shows close-up relief maps on both benches of the blasting area with the locations of each blast outlined in black and the shot numbers adjacent in red. In general, all blast events were

approximately 762 m (2500 ft) from the test structure. They were located on the same side of the quarry as the house and shot on either south or east facing high-walls.

Scaled distance factors for these shots had to be calculated from information received via the blasting company. Borehole geometries and timing patterns were pieced together to calculate maximum charge weights per delay. Unless otherwise noted (as in comparison 4), it will be assumed that one “delay” consists of any number of holes within 8 ms of each other. Time delay in detonations is for the most part dependent of timing delays within the blasting caps, up-hole delays and non-electric shock tubes. However, often times these planned timing patterns are not achieved due to errors in the blasting caps (Dowding, 1996). The errors are a function of several various sources, including human error and manufacturing, and at one time were as high as 7 to 8% for the 500 ms in-hole delay. These timing issues play an important role in accurately calculating and assessing scaled distance factors.

Shot #	Date/Time	Raw Distance (ft)	# of Holes	Total Weight (lbs)	Weight Per Hole (lbs)	Charge Wt. Per Delay (Calculated) (lbs)	Charge Wt. Per Delay (Given) (lbs)	Scaled Distance (Calculated) (ft/lb <sup>1/2</sup> )	Scaled Distance (Given) (ft/lb <sup>1/2</sup> )	Pattern	Maximum Holes/Delay (w/in 8ms)	Total Shot Time (ms)
1	20-May	2429	30	8280	276	552	290	103	143	Wide "V"	2	352
9	17-Jun	2693	37	10271	278	833	810	93	95	Row	3	428
8	10-Jun	2217	48	3920	147	358	600	117	91	Row		557
15	22-Jul	2376	51	13081	256	769	810	86	84	Wide "V"	3	419
14	16-Jul	2430	41	7957	194	582	400	101	122	Row	3	310
22	10-Sep	2270	57	13251	232	697	500	86	102	Wide "V"	3	402
18	2-Aug	2441	59	15345	260	780	660	87	95	Wide "V"	3	436
19	15-Aug	2490	37	7495	203	810	840	87.5	86	Tight "V"	4	215

**Table 3.1 Summary of pertinent blast design and borehole geometry information for blast events 1, 9, 8, 15, 14, 22, 18, and 19**



Figure 3.1 Aerial photograph outlining Stiles Road Quarry boundary with blasting area and Connecticut monitoring house encircled

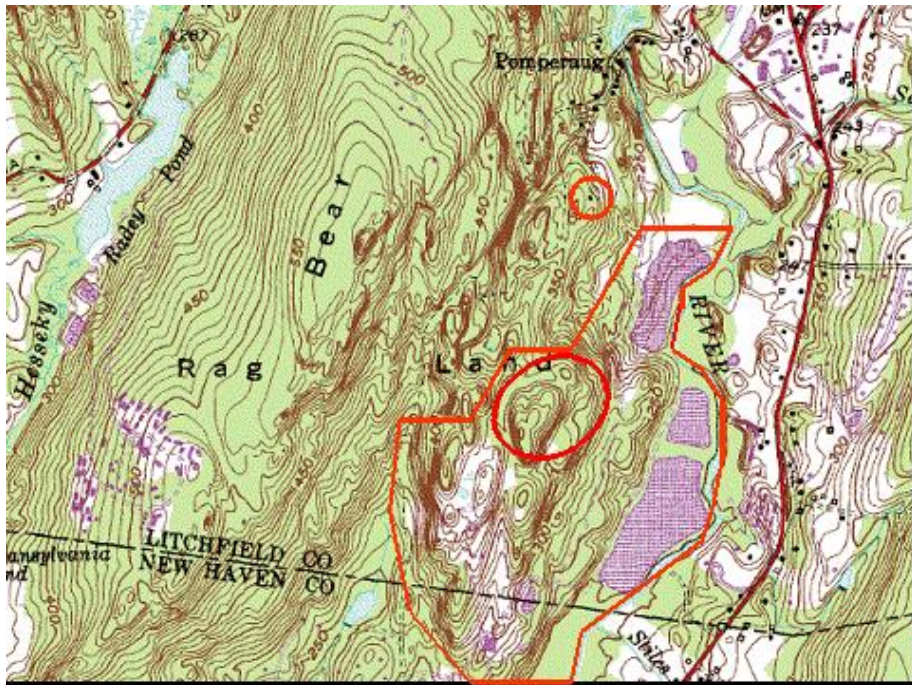
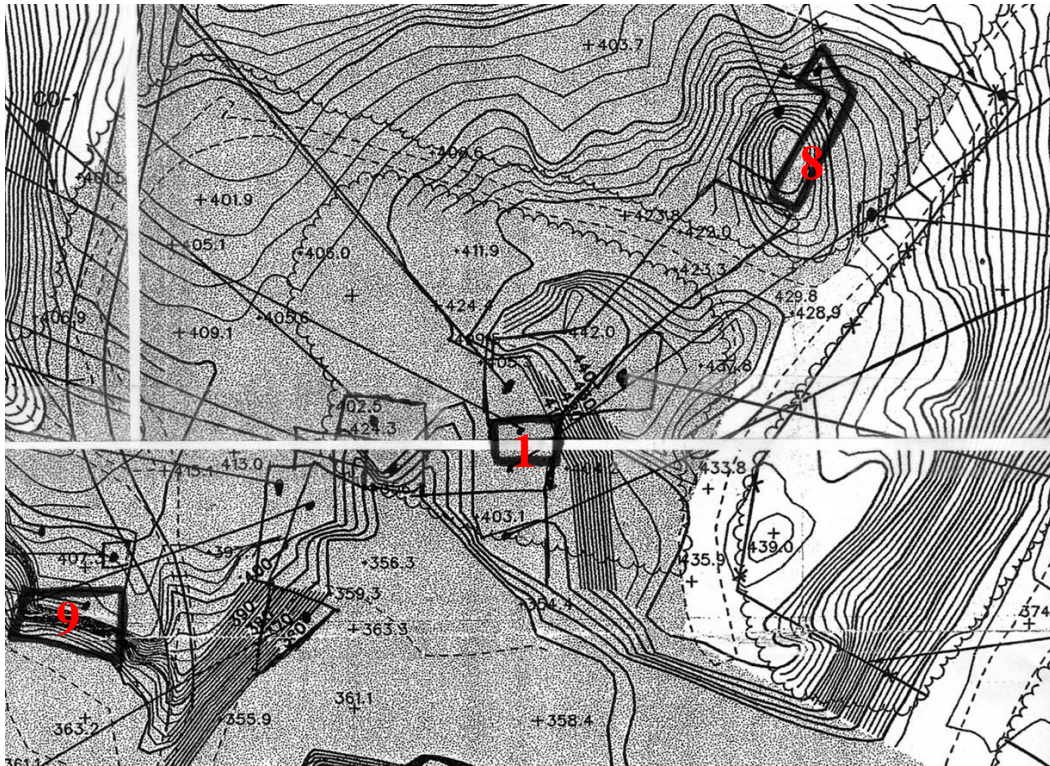
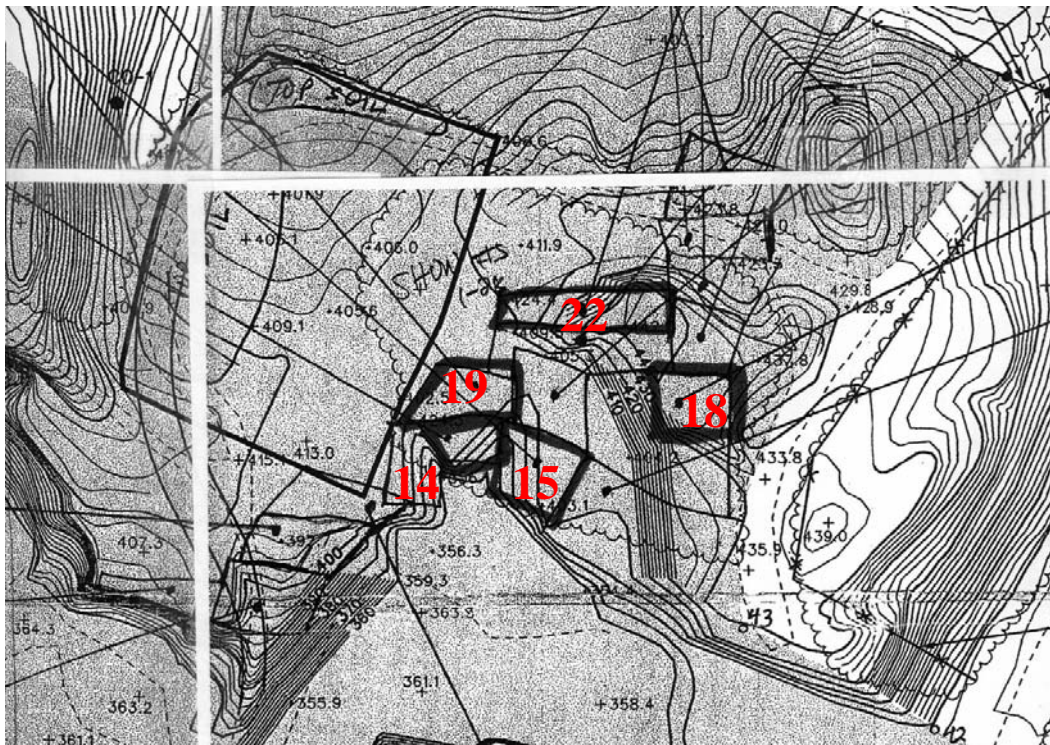


Figure 3.2 USGS Topographic map outlining Stiles Road Quarry boundary with blasting area and Connecticut monitoring house encircled





Upper Bench



Lower Bench

Figure 3.3 Detailed topographic relief maps of blast 1, 8, 9, 14, 15, 18, 19, and 22 locations (outlined in black, numbered in red) on upper and lower bench of Stiles Road Quarry



**Shot 1 (Narrow V, single face) vs. Shot 9 (Wide V, double face )**

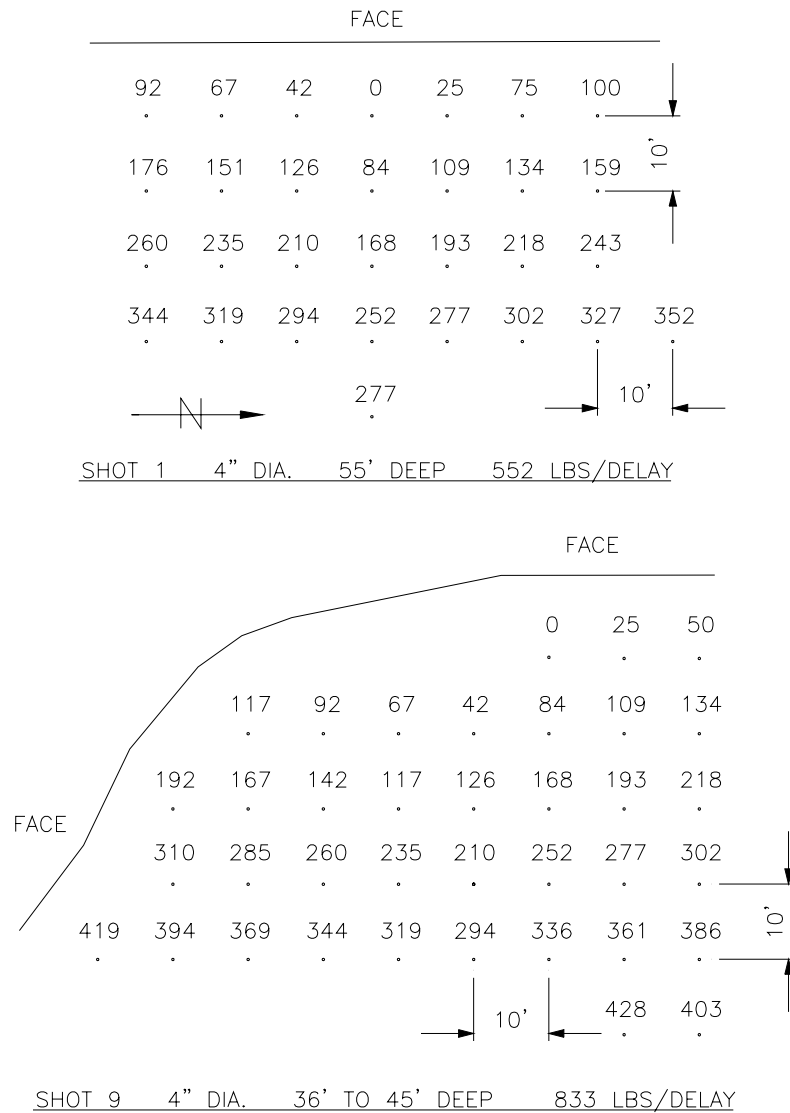
Shot 1 involved a maximum charge weight of 550 lb/delay at a distance of 2429 feet from the house, for a scaled distance factor of  $104 \text{ ft/lb}^{1/2}$ . It produced a peak particle velocity of 0.18 ips (longitudinal) and a Crack 1 displacement of  $41 \mu\text{in}$ . Shot 9 involved a charge weight of 833 lb/delay at a distance of 2693 ft for a scaled distance of  $94 \text{ ft/lb}^{1/2}$ , a peak particle velocity of 0.145 ips (longitudinal) and a Crack 1 displacement of  $29 \mu\text{in}$ . Both of these were standard production shots, and occurred at the same time of day.

The borehole layouts and timing patterns for these two shots are shown in Figure 3.4. Shot 1 is a narrow V pattern on a single face and maximum shot delay of 352ms (plus 500ms in hole). Accounting for the 8ms timing error discussed previously, there were a maximum of 2 holes per delay throughout the duration of the shot. With a total shot weight of 8280 lbs of explosive and 30 total holes, there are 276 lbs of explosive per hole. Shot 9 is a row-by-row or wide V pattern with two free faces and a maximum shot delay of 428ms (with 500ms in hole). There are a maximum of 3 holes per 8 ms delay, total shot weight of 10,271 lbs, 37 total holes and 278 lbs of explosive per hole.

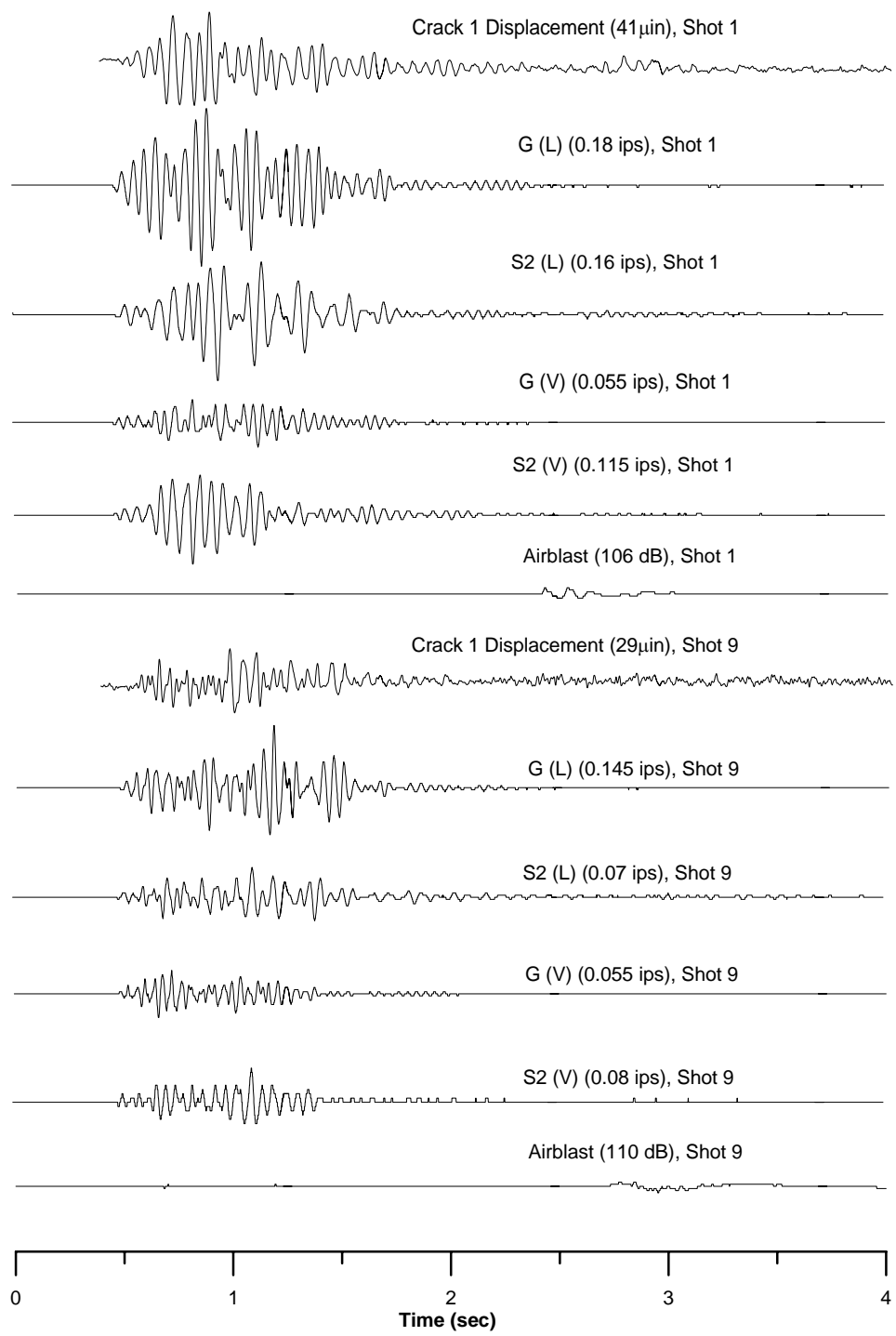
Figure 3.5 shows longitudinal and vertical time histories corresponding to shots 1 and 9, the top six show responses to Shot 1. Time histories are shown for Crack 1 response, longitudinal and vertical ground motion, longitudinal and vertical  $S_2$  structural response and decibel level airblast for shot 1, while those for Shot 9 are one the bottom. The peak longitudinal particle velocities for both these events are shown with shot 1 having a slightly higher value of 0.18 ips than 0.145 ips for shot 9. Both of these shots have identical peak particle velocities in the vertical direction of 0.55 ips. Despite these measurements, shot 1 produced a vertical structural response 44% higher (0.115 ips vs. 0.08 ips) and Crack 1 displacement 25% greater ( $41 \mu\text{in}$  vs.  $29 \mu\text{in}$ ) than Shot 9.

The significant structural and crack response differences in this comparison shows the effect of borehole layout, shot progression, free face employment and general shot design criteria. Under standard blast design criteria, the lower scaled distance factor for shot 9 should produce larger ground motions and more significant structural and crack response. However, the row-by-row geometry, shot progression and the existence of a curved/multi-face for shot 9 may have provided more relief. The timing progression in Figure 3.4 for Shot 9 shows five total rows, wherein each row contains only one

overlapping delay (within 8ms). Even though there are a maximum of three holes per delay, the holes are spread out and progress in opposite directions. Shot 1 is a narrower “V” pattern, which is also traditionally employed to distribute explosive energy in multiple directions, but there is only one free face for relief.



**Figure 3.4 Borehole geometry and delay timing patterns for blast events 1 and 9**



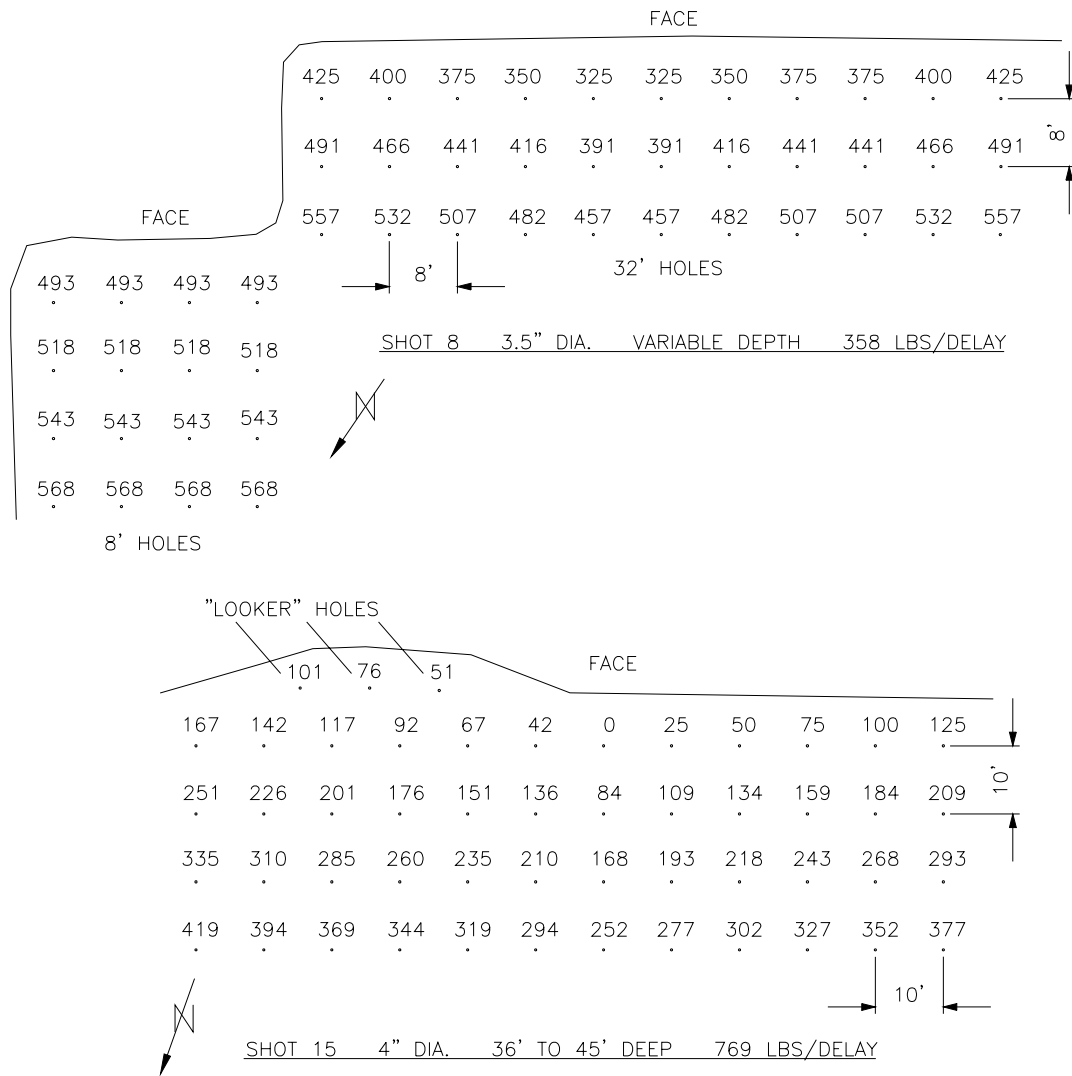
**Figure 3.5 Crack 1 displacement time histories for blast events 1 and 17 compared to longitudinal ground motion, longitudinal S<sub>2</sub> structural response, vertical ground motion, vertical S<sub>2</sub> structural response, and airblast**

**Shot 8 (Shallow stemming) vs. Shot 15 (Normal stemming)**

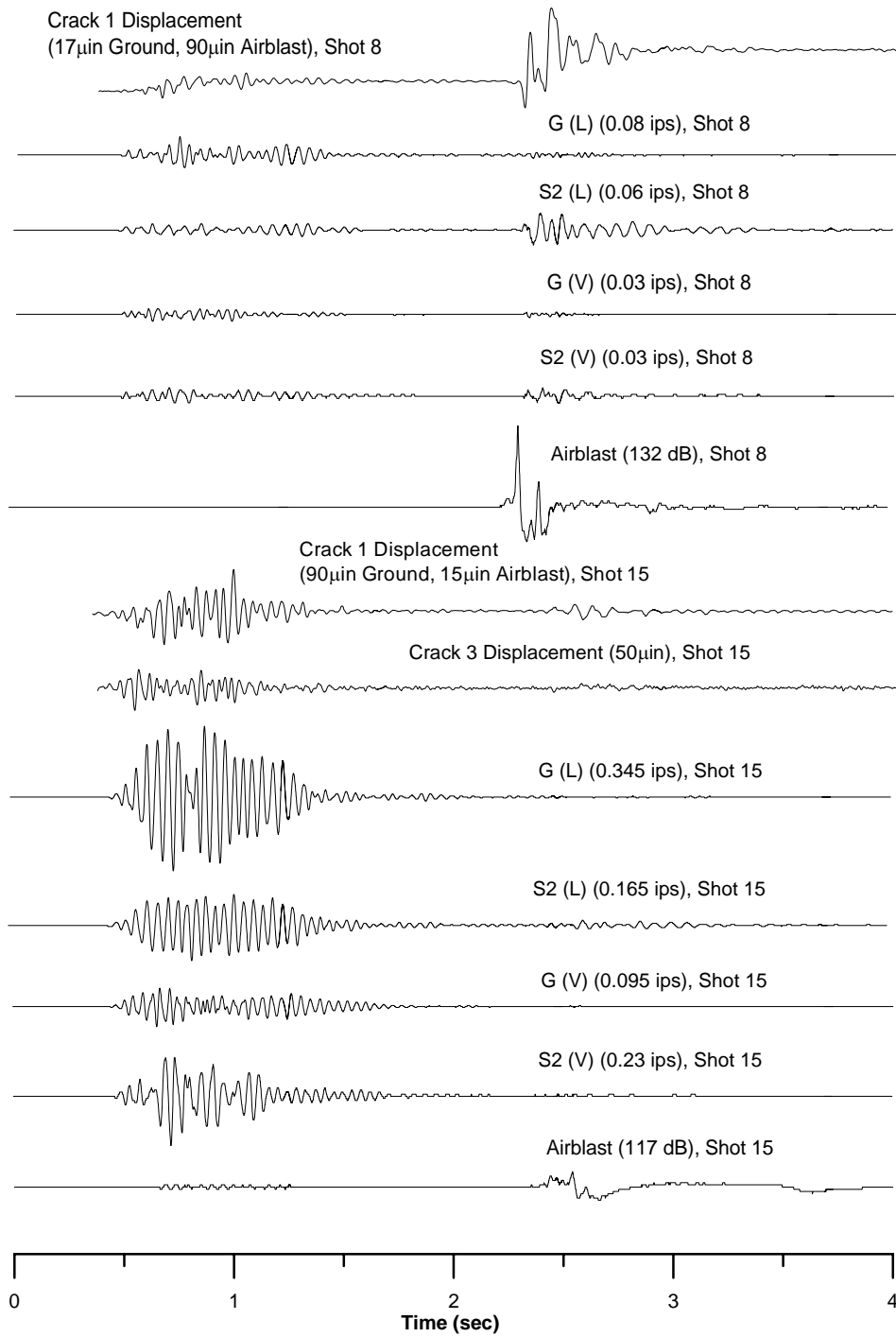
Shot 8 involved a charge per delay of 600 lbs, at a distance of 2217 feet, with a scaled distance factor of  $91 \text{ ft/lb}^{1/2}$ . It produced a peak particle velocity of 0.08 ips. Shot 15 with a charge per delay of 810 lbs, at a distance of 2376 feet, has a scaled distance factor of  $84 \text{ ft/lb}^{1/2}$  and the largest monitored peak particle velocity of 0.345 ips.

The borehole layouts and timing patterns for these two shots are included as Figure 3.6, and their respective time histories are presented as Figure 3.7 in the same format as Figure 3.5. Shot 8 produced a rare, maximum 132 dB airblast that caused a great deal of response in Crack 1. The reasons for this air pressure are unknown, but is most likely the result of a combination of blasting a knoll on the north end of the quarry whose free-face was at an elevation well above the structure and stemming too short to effectively contain the explosive gases. The blasting log for this event shows a grouping of 16, 3.5” holes drilled to an eight-foot depth with five feet of stemming. Conventional blasting design calls for stemming of approximately 30 times the diameter of the hole, or 7 to 8 feet for 3.5” holes.

In contrast to Shot 8, Shot 15, elicited the maximum peak particle velocity of any recorded event, 20% higher than the next highest PPV, but did not have the smallest scaled distance factor. This shot was designed with 4” holes drilled to 36 to 45 feet with 12 feet of stemming in a “V” pattern. Although this shot progressed to the north in the general direction of the structure and provided what appears to be an ideal circumstance for high-energy transfer. Typically the “V” pattern employed in this situation with a standard 25 ms delay and 10 foot burden usually directs the seismic energy in two directions, and consequently keeps vibrations to a minimum.



**Figure 3.6 Borehole geometry and delay timing patterns for blast events 8 and 15**



**Figure 3.7 Crack 1 displacement time histories for blast events 8 and 15 compared to longitudinal ground motion, longitudinal S<sub>2</sub> structural response, vertical ground motion, vertical S<sub>2</sub> structural response, and airblast**

**Shot 14 (19 Hz in long. direction) vs. Shot 22 (27 Hz in long. direction )**

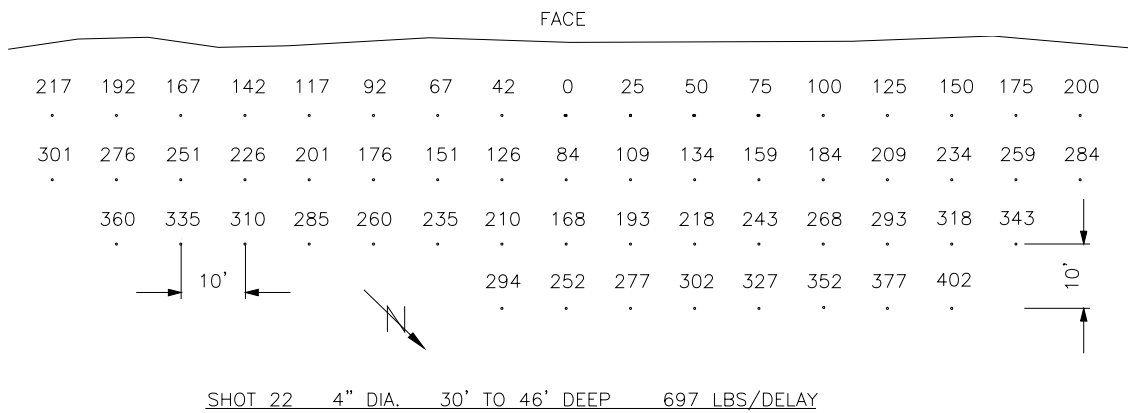
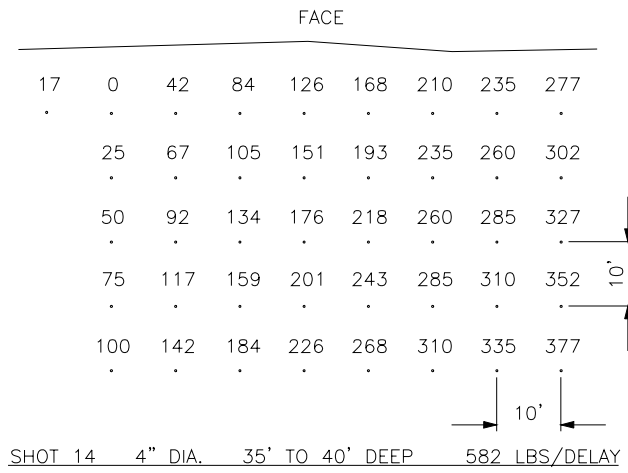
Shot 14 involved a maximum charge weight of 582 lb/delay at a distance of 2430 feet from the house, for a scaled distance factor of  $101 \text{ ft/lb}^{1/2}$ . It produced a peak particle velocity of 0.19 ips (longitudinal), Crack 1 displacement of  $60 \mu\text{in}$  and Crack 3 displacement of  $59 \mu\text{in}$ . Shot 22 involved a charge weight of 697 lb/delay at a distance of 2270 ft for a scaled distance of  $86 \text{ ft/lb}^{1/2}$ . It produced a peak particle velocity of 0.27 ips (longitudinal), Crack 1 displacement of  $60 \mu\text{in}$  and Crack 3 displacement of  $34 \mu\text{in}$ . Both of these were standard production shots, and occurred at the same time of day.

The borehole layouts and timing patterns for these two shots are shown in Figure 3.8. Shot 14 is a diagonal row pattern on a single face and maximum shot delay of 310ms (plus 500ms in hole). Accounting for the 8ms timing error discussed previously, there were a maximum of 3 holes per delay throughout the duration of this shot. With a total shot weight of 7957 lbs and 41 total holes, there are a relatively low 194 lbs of explosive per hole. Shot 22 is a classic wide V pattern on a single face and a maximum shot delay of 410ms (with 500ms in hole). There are a maximum of 3 holes per 8 ms delay, total shot weight of 13,251 lbs, 57 total holes and 232 lbs of explosive per hole.

Figure 3.9 shows longitudinal and vertical time histories corresponding to shots 14 and 22; the top seven time histories Crack 1 response, Crack 3 response, longitudinal and vertical ground motion, longitudinal and vertical  $S_2$  structural response and decibel level airblast for shot 14. The bottom seven show identical information for shot 22. The peak particle velocities for both these events occur in the longitudinal direction, with shot 22 producing 0.27 ips versus 0.19 ips for shot 14.

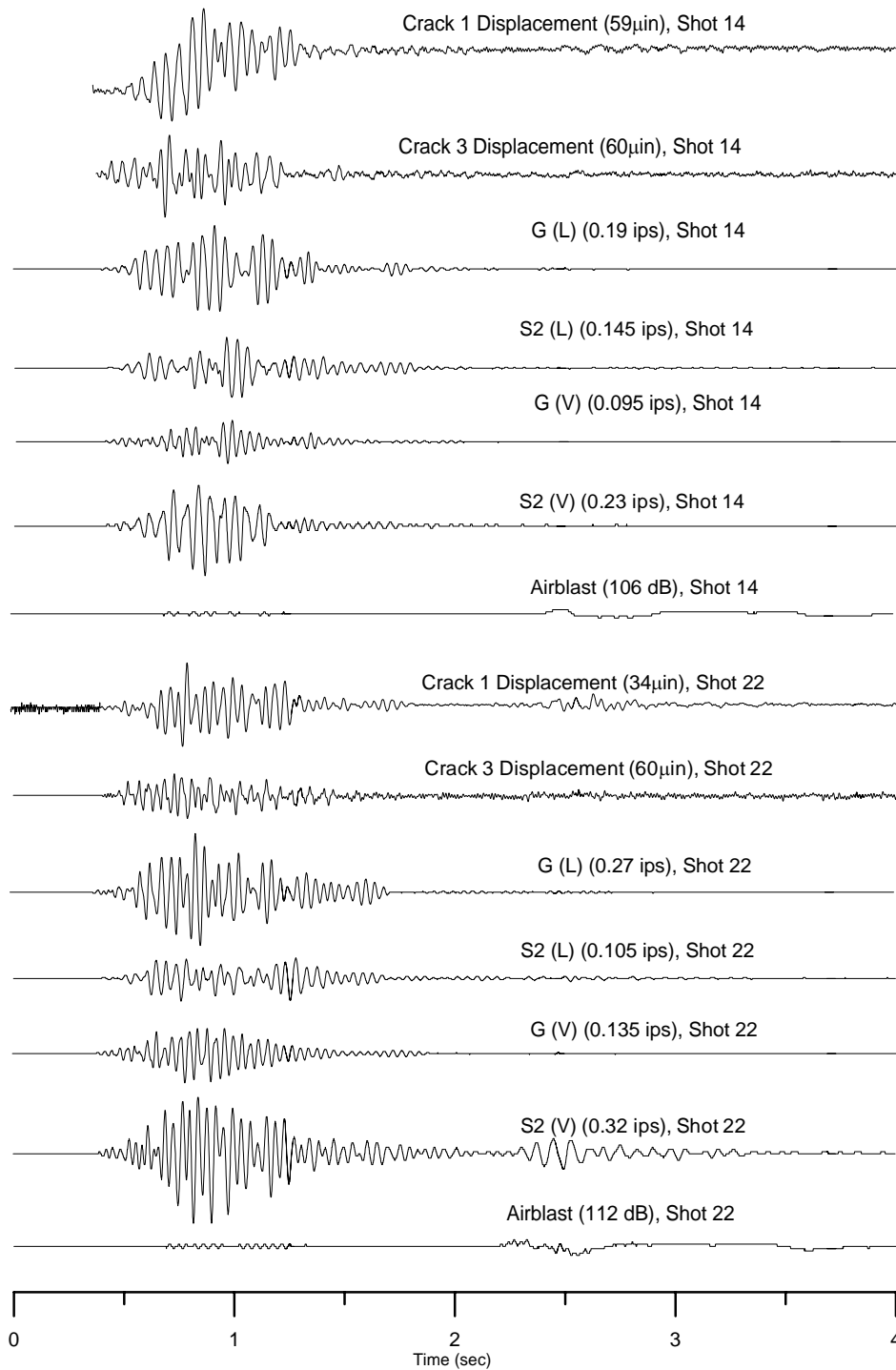
This comparison introduces the concept of crack displacement normalized to peak particle velocities for the purpose of showing the effect of alternatively significant measured data. For the purposes of normalization, individual crack displacements are divided by the peak particle velocities, Crack 1 with vertical and Crack 3 with longitudinal, in order to divide out or normalize the effect of peak particle velocity. This comparison demonstrates the effect of differences in dominant frequency.

There are three primary methods of establishing the dominant excitation frequency of a blast event; the zero-peak crossing, fourier frequency transform (FFT), and single-degree of freedom methods described in Chapter 2. As the single-degree of



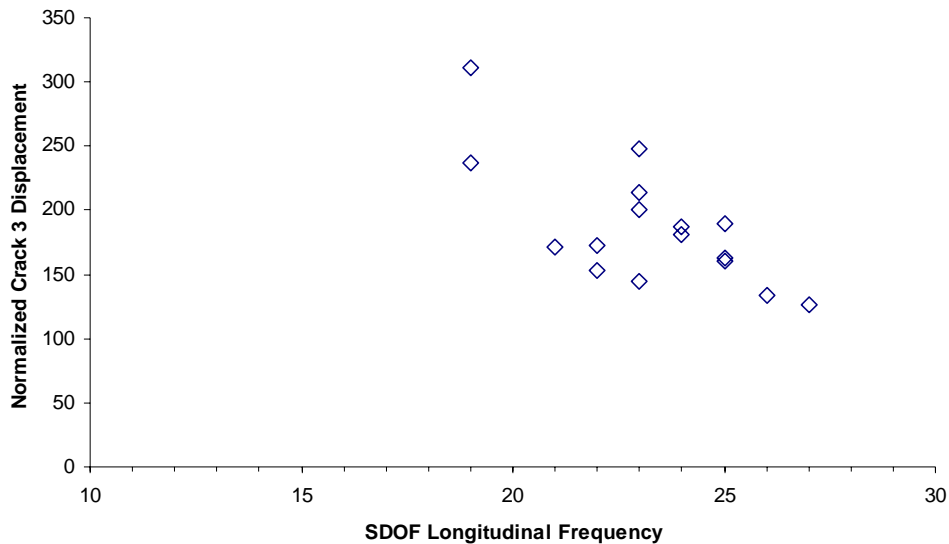
**Figure 3.8 Borehole geometry and delay timing patterns for blast events 14 and 22**





**Figure 3.9 Crack displacement 1 and 3 time histories for blast events 14 and 22 compared to longitudinal ground motion, longitudinal S<sub>2</sub> structural response, vertical ground motion, vertical S<sub>2</sub> structural response, and airblast**

freedom method best describes structure displacement as a function of frequency, it will be employed for the purposes of this discussion. Furthermore, only longitudinal frequencies and Crack 3 will be examined, as the frequencies in the vertical direction did not vary significantly for the shots monitored in this time period. In terms of measured ground motions and peak particle velocities, these two shots performed as expected. Shot 22 had a lower scaled distance factor and was a wide V, thereby producing a relatively high peak particle velocity of 0.27 ips versus 0.19 ips for shot 14. Crack 3 displacement normalized to longitudinal peak particle velocity for shot 22, however, was 126 $\mu$ in/ips, the lowest among any recorded shot. By comparison, normalized Crack 3 displacement of Shot 14 was 145 $\mu$ in/ips, falling somewhere in the middle of normalized response. A possible reason for this low relative response for shot 22 is the dominant frequency of 27 Hz in the longitudinal direction, as compared to a 19 Hz frequency from shot 14. In Chapter 2 the dominant frequency of the structure is shown to lie between 11 and 15 Hz, closer to the dominant frequency of Shot 14. Thus greater structural and crack response is expected for the shot whose excitation frequency more nearly matches the natural frequency of the structure. Figure 3.10 shows a plot of normalized Crack 3 displacement to dominant SDOF frequencies for all shots.



**Figure 3.10 Normalized Crack 3 displacement ( $\delta_3/L$  PPV) versus longitudinal single degree of freedom dominant frequency for blasting in Connecticut**

**Shot 18 (Total shot time of 436ms) vs. Shot 19 (Total shot time of 215ms )**

Shot 18 involved a maximum charge weight of 780 lb/delay at a distance of 2441 feet from the house, for a scaled distance factor of  $88 \text{ ft/lb}^{1/2}$ . It produced a peak particle velocity of 0.195 ips (transverse), a Crack 1 displacement of  $80 \mu\text{in}$  and a Crack 3 displacement of  $37 \mu\text{in}$ . Shot 19 involved a charge weight of 810 lb/delay at a distance of 2490 ft for a scaled distance factor of  $88 \text{ ft/lb}^{1/2}$ . It produced a peak particle velocity of 0.215 ips (longitudinal) a Crack 1 displacement of  $38 \mu\text{in}$  and a Crack 3 displacement of  $33 \mu\text{in}$ . Both of these were standard production shots, and occurred at the same time of day.

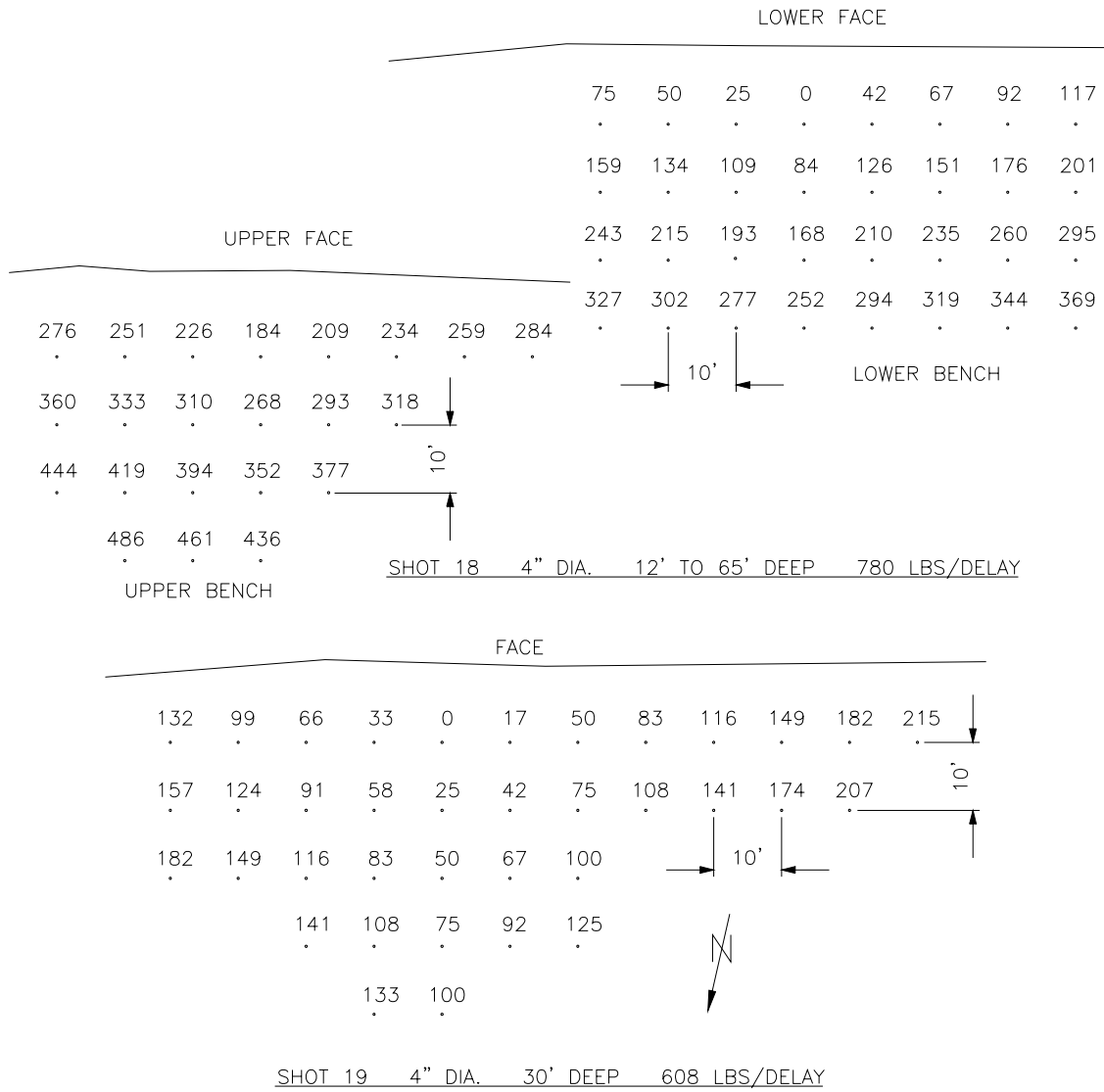
The borehole layouts and timing patterns for these two shots are shown in Figure 3.11. Shot 18 is a wide V pattern on a single face and maximum shot delay time of 410ms (plus 500ms in hole). Accounting for a possible 8ms timing error discussed previously, there were a maximum of 3 holes per delay throughout the duration of the shot. With a total shot weight of 15,345 lbs of explosive and 59 total holes, there are 260 lbs of explosive per hole. Shot 19 is a tight V pattern on a single face, but with a maximum shot delay of 215ms (with 500ms in hole). There are a maximum of 4 holes per 8 ms interval, total shot weight of 7495 lbs, 37 total holes and 203 lbs of explosive per hole.

Figure 3.12 shows time histories for shots 18 and 19; the top eight time histories are Crack 1 response, Crack 3 response, longitudinal ground motion, longitudinal  $S_2$  structural response, vertical ground motion, vertical  $S_2$  structural response, vertical  $S_{\text{mid}}$  mid-ceiling structural response and decibel level airblast for shot 19. The bottom seven time histories show identical information, minus the  $S_{\text{midceiling}}$  structural response for shot 18, as it was installed after the shot. The peak particle velocities for both these events lie in the longitudinal direction, with shot 19 producing a slightly higher value of 0.215 ips versus 0.195 ips for shot 18. Vertical PPV was 0.115 ips for shot 18, and for shot 19 only slightly lower at 0.085.

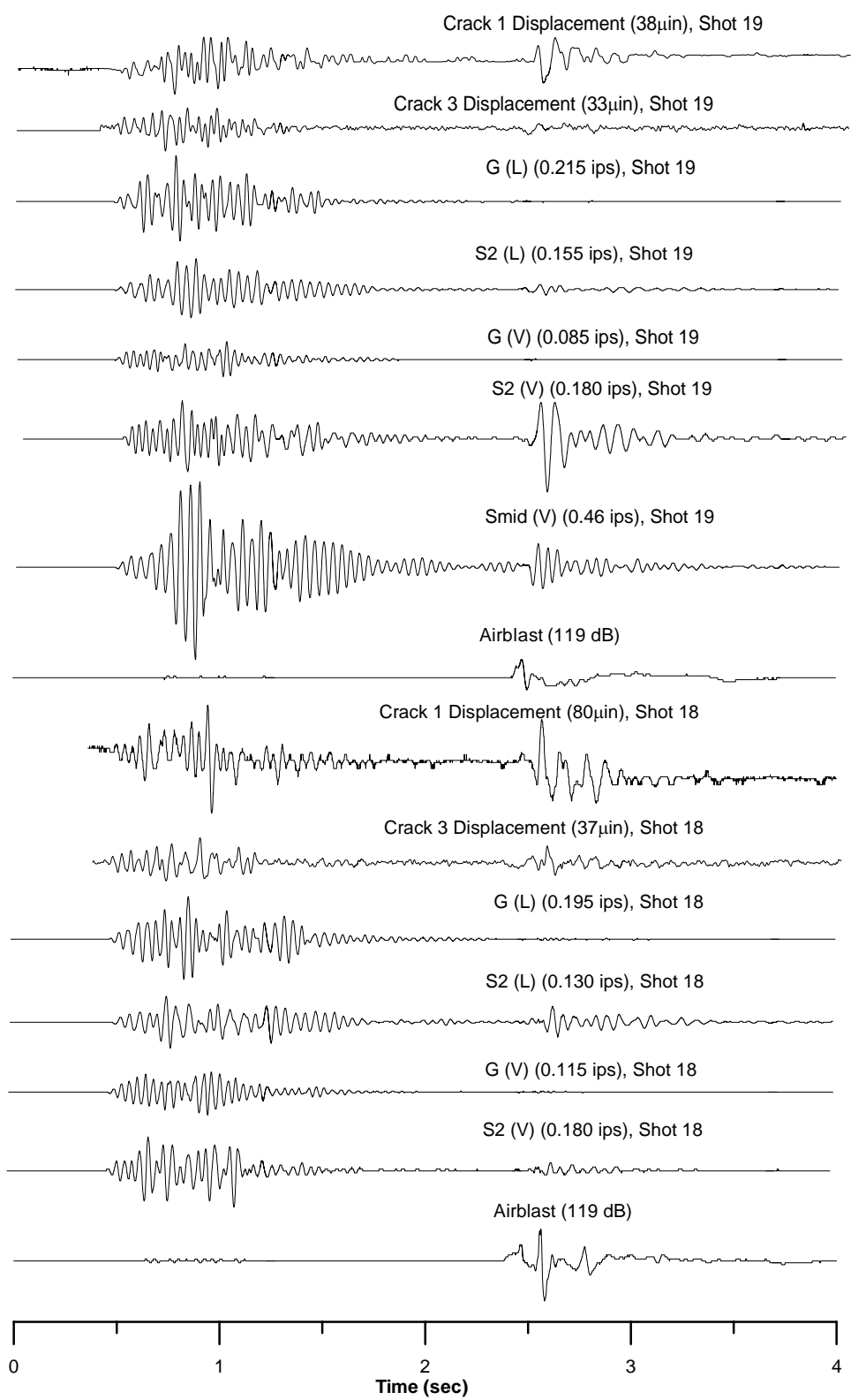
This comparison explores the final design component: total shot time, percentage error in delay timing and coupled seismic energy producing significant pulses in ground motion. Both shots 18 and 19 are designed for 3 and 4 holes per delay respectively, and have almost identical scaled distance factors. The frequencies of the shots are nearly

identical, the longitudinal SDOF frequencies being 22 and 25 Hz, vertical 25 and 26 Hz. Shot 19 is a tighter V pattern, but they were both shot on single faces and have identical spacing and burden, so the borehole layouts themselves should not significantly affect the results at the house. The issue here is the shot timing, and the question of “whether 15,000 lbs of explosive shot in 60 holes versus 7,500 lbs of explosive shot in 30 holes, with similar scaled distance factors will produce different displacement responses in the same crack”?

A factor that should always be considered carefully in blast design is the possibility for coupled seismic energy. Previous studies have shown that, with the use of non-electric delays, the error in delay timing increases as the number of holes and/or shot duration increases. Longer shot durations and timing errors also may involve larger numbers of significant pulses. Significant pulses (called principal pulses) are defined as ground motions in excess of 75% of the peak value. Shots 18 and 19 both produced 3 principle pulses in the longitudinal direction, but the additional coupled energy later in Shot 18 resulted 5 principal pulses in the vertical direction versus only 3 for Shot 19. This difference may also be a consequence of the longer duration in Shot 18. The normalized Crack 1 displacement from Shot 18 was 700 $\mu\text{in}/\text{ips}$ , as compared to 450 $\mu\text{in}/\text{ips}$  for Shot 19. Even with the same number of principal pulses, however, the normalized Crack 3 displacement from Shot 18 of 190 $\mu\text{in}/\text{ips}$  was still greater than the 154 $\mu\text{in}/\text{ips}$  for Shot 19.



**Figure 3.11 Borehole geometry and delay timing patterns for blast events 18 and 19**



**Figure 3.12 Crack displacement 1 and 3 time histories for blast events 18 and 19 compared to longitudinal ground motion, longitudinal S<sub>2</sub> structural response, vertical ground motion, vertical S<sub>2</sub> structural response, event 19 vertical S<sub>midceiling</sub> structural response, and airblast**

## *Chapter 4*

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### *Construction Vibrations, Las Vegas, Nevada*

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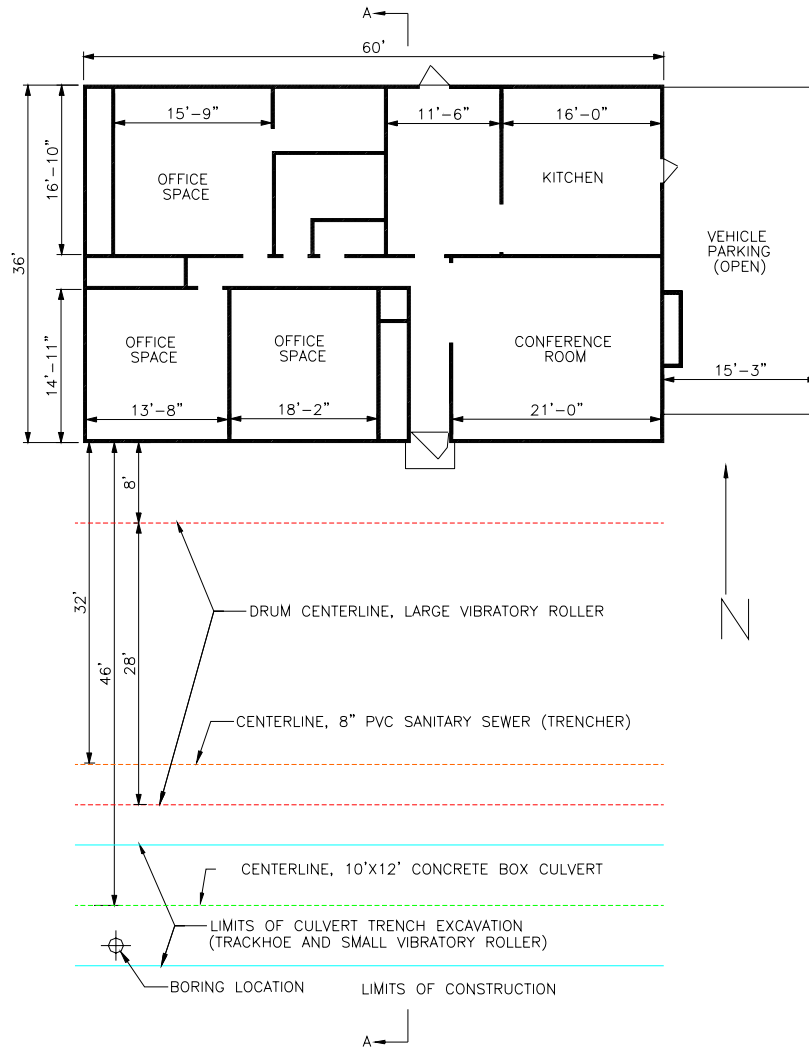
#### **Introduction**

The one-story residential test structure, shown in Figure 4.1, is located immediately adjacent to the widening and reconstruction of West Ann Road in Las Vegas, Nevada. Vibratory crack deformation resulted from ground motion produced by backhoe excavation, trenching and vibratory rolling on cemented desert colluvium.

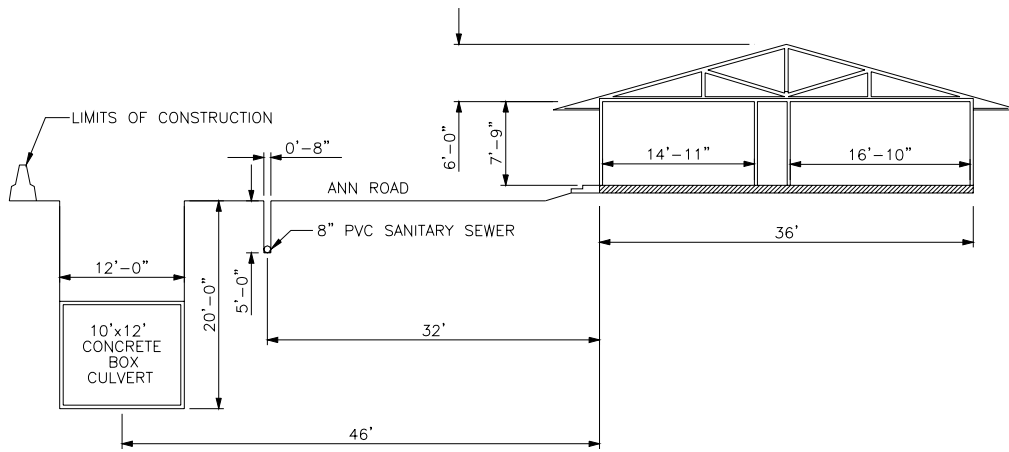


**Figure 4.1 Single-story house adjacent to Ann Road Construction in Las Vegas, Nevada**

Figure 4.2 shows plan and cross-sectional views of the test structure and its position relative to the construction activities on Ann Road. This slab-on-grade founded, one-story house is approximately 60 feet long, 36 feet wide, and eight feet tall, floor to ceiling. As shown in Figure 4.2 (a) two trenches were excavated: a 12-foot wide, 20-foot deep trench approximately 46 feet from the front of the house, and an 8-inch wide utility trench approximately 32-feet away. These excavations will be discussed in detail later in the chapter.



(a)



(b)

**Figure 4.2 (a) Plan view geometry of monitoring house, construction locations, and soil boring location, (b) cross-sectional view of house geometry and excavations (section A-A)**



### Ann Road Soil Profile

Three, 20-foot deep soil-borings along the centerline of Ann Road in the vicinity of the test structure reveal the soil conditions. Figure 4.2 shows the location of the boring directly in front of the house; the other two approximately 500 feet in either direction along the construction centerline. Figure 4.3 compares the standard penetration blow count versus depth for the three borings. While the penetration resistance varies widely, all three borings reveal variable depth, and at times thick, layer of caliche, a calcium-rich cemented soil formed by the evaporation of alluvial groundwater in desert climates. Above and between these random caliche deposits, the borings show thin layers of sandy gravel to silty clay fill over natural silty clay and sandy clay layers. These variable caliche deposits are important to this case study because of the relative energy necessary for their excavation, as well as their energy transfer characteristics. These topics will be covered in detail later in this chapter during the ground attenuation study discussion.

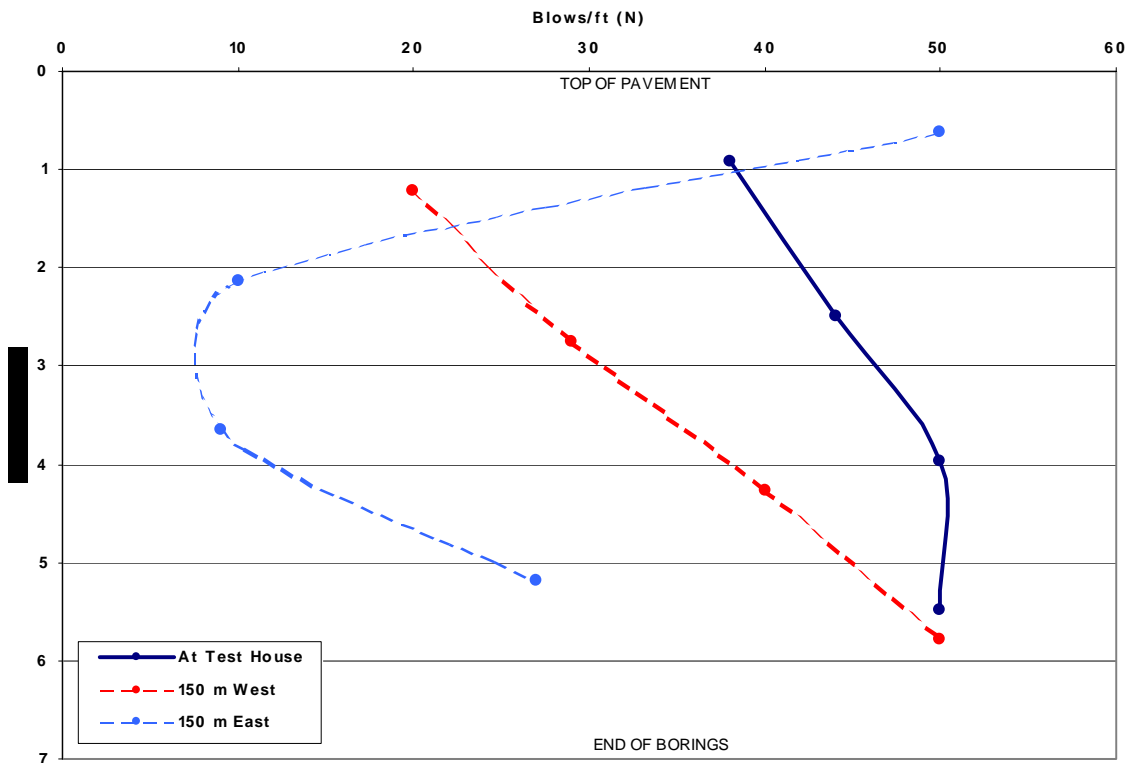


Figure 4.3 Blow count versus depth for three Ann Road soil borings in vicinity of test structure

**Construction Equipment**

Reconstruction and roadway improvements are divided into excavation and vibratory compaction activities for the purposes of monitoring construction vibrations. Excavation activities include excavation for a 10’x12’ reinforced box storm culvert with a Hitachi 1200 EX Super trackhoe and excavation for an 8” PVC sanitary line by Tesmec TRS-1175XL “chainsaw” trencher. Photos of both are shown in Figures 4.4 (a) and (b). Vibratory compaction activities include compaction of backfill materials employed in the 12’ wide culvert trench with a Dynapac CC 522 single-drum vibratory roller, and the compaction of roadway subgrade with an Ingersoll-Rand Pro-Pac Series SD-115F single-drum vibratory soil compactor. Figures 4.5 (a) and (b) show photos of both vibratory roller compaction machines. Specifications for all machines are found in Appendix B.



(a)

(b)

**Figure 4.4 Photographs of trenching equipment on Ann Road, (a) Hitachi trackhoe with dump truck and (b) Tesmec chain trencher**



(a)

(b)

**Figure 4.5 Photographs of vibratory compaction equipment on Ann Road, (a) Dynapac small single-drum roller and (b) Ingersoll-Rand large single-drum vibratory soil compactor**

### **Structural Details**

The interior walls of the Las Vegas structure are constructed of drywall over a wood-frame and the exterior is covered by southwestern-style stucco. The house is in generally good condition, with the majority of the cosmetic cracking is on the exterior stucco material.

Photographs of the roof and truss system are shown in Figure 4.6. This system consists of a series of 2"x4" ceiling joists placed at 2'6" centers. These joists are supported by a central 2"x8" beam running the 60-foot longitudinal length of the house.



**Figure 4.6 Roof and truss system for one-story Las Vegas structure**

### **Instrumentation**

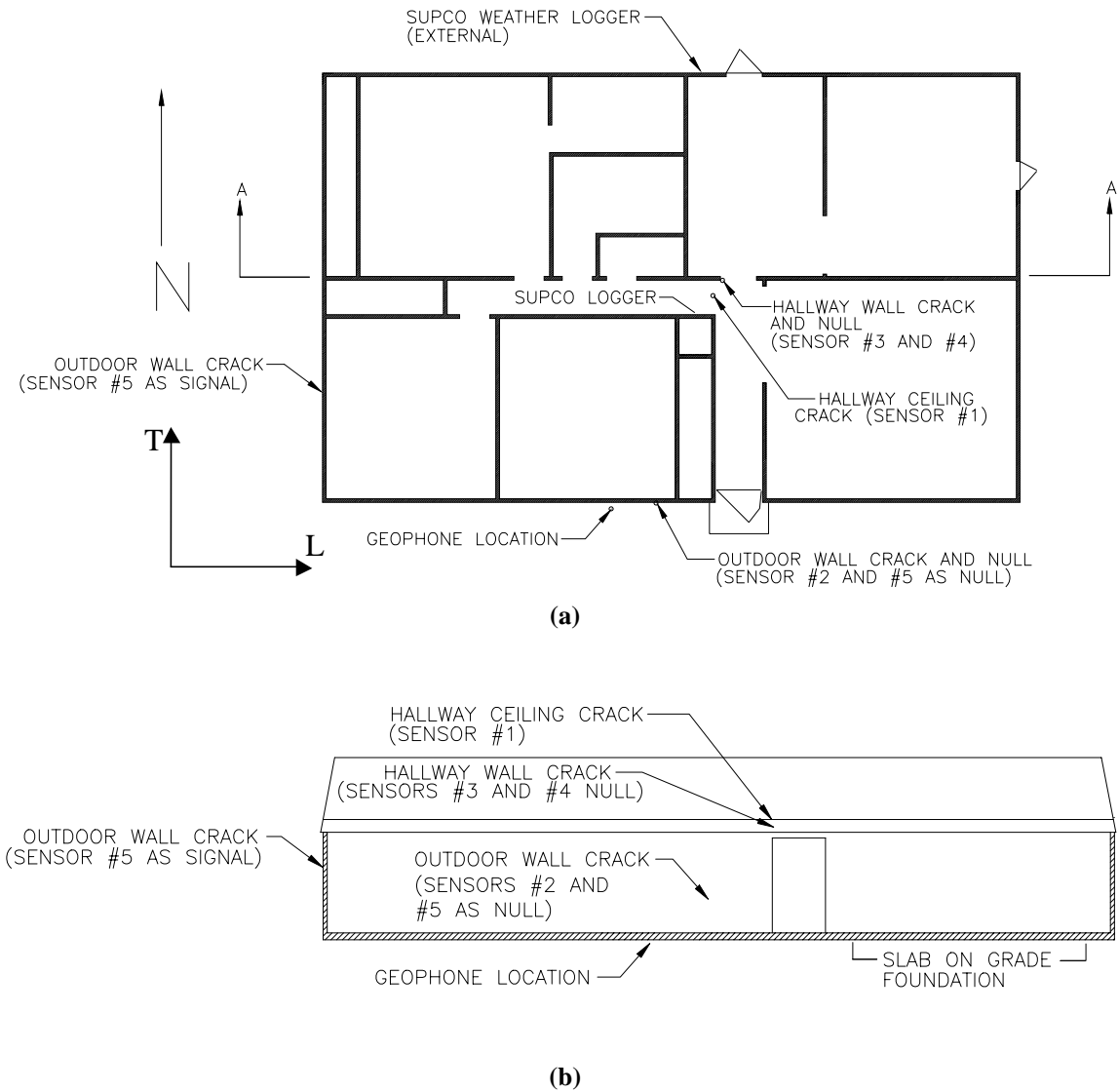
Instrument locations are indicated on the plan and elevation views in Figure 4.7. A tri-axial geophone block was installed approximately two feet from the South (construction) face of the structure to measure excitation ground motions in the longitudinal, transverse, and vertical directions. As with previous studies, the longitudinal direction is defined as parallel to the long axis of the structure. Four cracks were instrumented with Macro-sensor LVDT micro-inch crack displacement gages for this study. Three of the cracks and the interior null were in place for the full monitoring period. On August 12<sup>th</sup>, 2002, the external null sensor (#5) was moved to an external crack on the transverse west wall for the remainder of the project.

All of the crack sensors, as well as the geophone were wired to a Somat eDAQ data acquisition system. This eDAQ provides simultaneous triggering of crack sensors to acquire transient response whenever the geophone exceeds a predetermined excitation

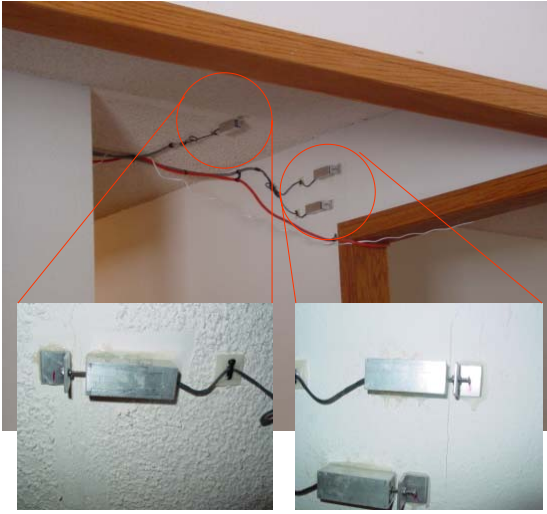
trigger threshold, as well as readings every hour to acquire long-term response to environmental effects. Hourly temperature and humidity were recorded internally and externally with independent Supco weather loggers. The data from these loggers was manually downloaded and correlated with the field measured crack data.

Three indoor sensors monitor cosmetic wall and ceiling cracks, plus the null wall response. The ceiling and wall cracks are labeled “Sensor 1” and “Sensor 3” in Figure 4.7 and the null gage, adjacent to sensor 3, is labeled “Sensor 4”. As shown by the detailed photographs of these three sensors in Figure 4.8, the wall crack is located high on the wall.

The two external stucco cracks are also located in Figure 4.7. Sensor 2, shown in Figure 4.8(b), spans a crack near the door running vertically up the southern wall, which is the closest, parallel wall to Ann Road. Sensor 5 in Figure 4.8(c) (originally selected as an external null gage until August 12<sup>th</sup>, 2002), spans a vertical stucco crack on the west, transverse face of the house.



**Figure 4.7 (a) Plan view of monitoring house and (b) profile view of house, showing Cracks and LVDT sensors 1-5, geophone, and weather logger instrument locations**



(a)



(b)



(c)

**Figure 4.8 LVDT micro-inch sensors and cracks (a) interior ceiling, Crack 1 and wall, Crack 3 with interior null sensor, (b) exterior west wall Crack 5, and (c) exterior south wall Crack 2**

**Ground Attenuation Study**

Attenuation of motions produced by representative construction equipment was measured in early September 2002 (Aimone-Martin, 2002). Equipment included a Dynapac model CA 151D vibratory roller running on both high and low frequency settings, a Tesmec TRS-1175 trencher, and an Hitachi EX 1200 Super trackhoe. Vibrations were monitored with linear arrays of either four or seven LARCOR seismographs oriented parallel and/or perpendicular to the direction of activity, recording longitudinal, transverse and vertical ground motions. Table 4.1 summarizes the three seismograph arrays employed for each machine monitored in this study. All the seismographs were wired in series, with the instrument closest to the monitored equipment acting as a trigger for the entire system. Trackhoe activities were monitored during the removal of existing asphalt pavement and the upper caliche soil level along Ann Road. The vibratory roller was placed in service specifically for this study, and thus was not monitored during construction activities along Ann Road. It was, however, operated over similar desert colluvial soils.

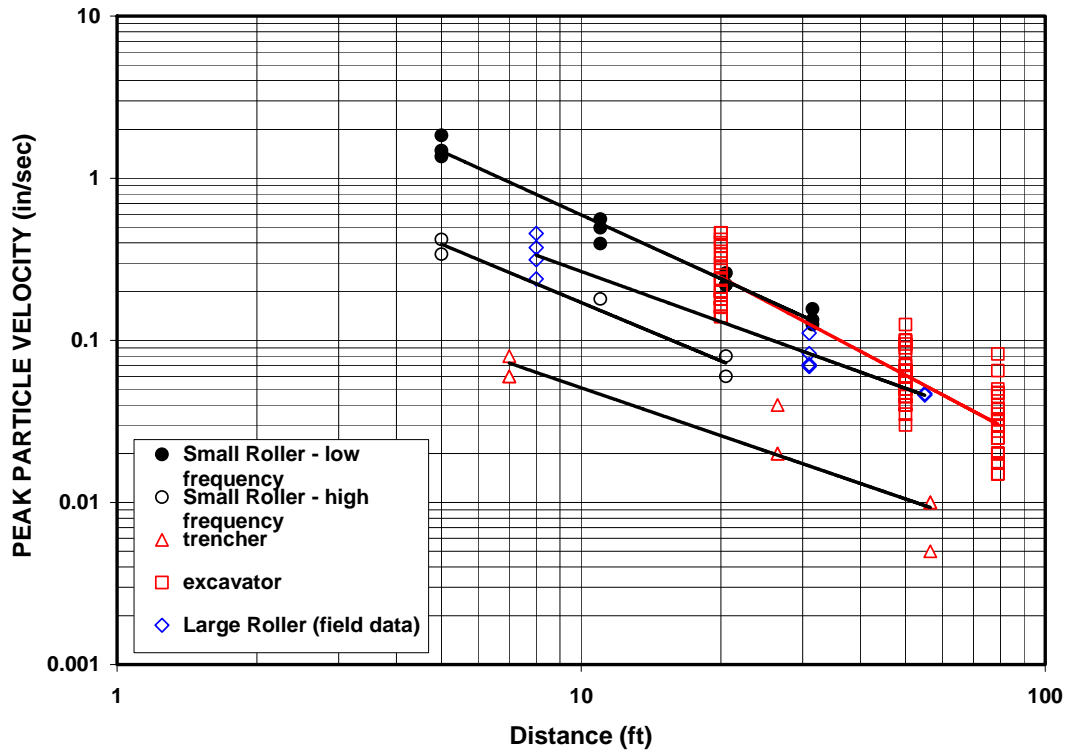
Machine Type and Model	Parallel Distances	Perpendicular Distances
	(ft)	(ft)
Hitachi 1200 Trackhoe	Variable 27 31	6 32 60 100
Tesmec TRS-1175 Trencher	None	7 27 43 57
Dynapac CA 151-D Roller	None	10 16 25 36

**Table 4.1 Summary of seismograph arrays employed in attenuation study on Ann Road**

Figure 4.9 is a log-log plot of peak particle velocity versus distance for the trackhoe, trencher and roller. A best-fit linear regression relationship determined for each machine describes the site-specific attenuation of the peak particle velocity. The attenuation of particle velocity in the ground is characterized by the negative exponent of



the distance term in the linear regressions shown on Figure 4.9. The y-intercept of this line, or the point representing a distance of 1 is proportional to the amount of relative energy transferred from the equipment to the ground. The trackhoe excavating the caliche layer imparts the highest level of energy into the ground, followed by the vibratory roller on the low frequency setting, high frequency setting (both operating over the caliche layer) and finally the trencher operating in relatively uncemented soils. The trencher was not excavating through the caliche layer at the time of monitoring.



**Figure 4.9 Peak particle velocity versus distance for ground vibration measurements adjacent to trackhoe, trencher, and vibratory roller construction activities**

Figure 4.10 is a log-log plot of peak particle velocity versus dominant frequency. The dominant frequency for each data set was calculated by Fast Fourier Transform (FFT) spectral analysis of velocity time histories. The trackhoe produced ground motions with dominant frequencies between 16 and 25 Hz. The factory prescribed operating frequencies for the Dynapac small roller are 29 Hz (low frequency) and 40 Hz (high frequency). The dominant frequencies of the resulting ground motions were 27.5 Hz low and 45.2 Hz high. The Ingersoll-Rand large vibratory roller has a frequency dial that adjusts from 18 to 32 Hz. The field data taken for this roller were recorded at either 23 or



32 Hz. Dominant frequencies presented in Figure 4.10 are constant out to the greatest distance measured for all data (up to 56 feet). Thus, frequency is shown to be relatively independent of distance, for activity within 60 feet. Excitation frequency at short distances should be considered constant when considering the frequency response characteristics of structures described in Chapter 2.

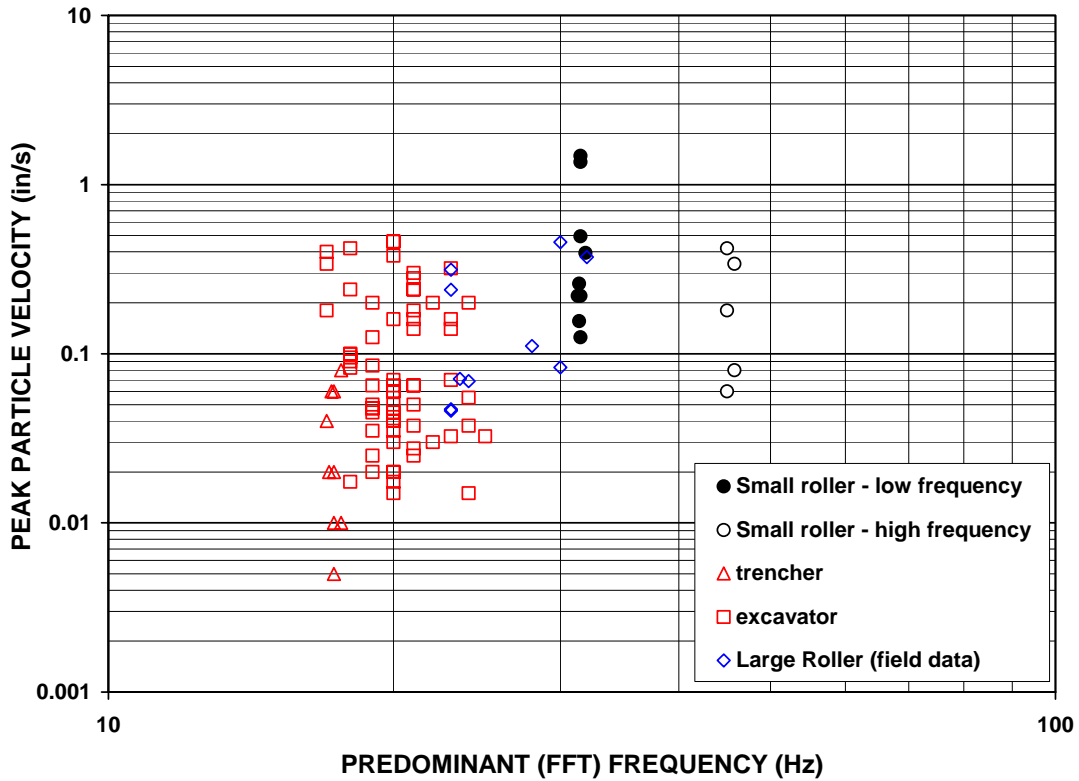


Figure 4.10 Peak particle velocity versus dominant frequency from fast fourier transform analysis for ground vibration measurements adjacent to trackhoe, trencher, and vibratory roller construction activities

## *Chapter 5*

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### *Construction Vibration Response Analysis and Recording*

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#### *Introduction*

One of the challenges of recording construction vibrations not found with blast vibrations is the necessity of measuring many (possibly hundreds) of daily events. While blasting vibrations typically occur only a few times a day or week, last for only a few seconds, and involve relatively standard ground motions and time histories, ground vibrations from construction machinery are less predictable and can last for relatively extensive periods of time. These differences in vibration environment brings forth the need for various and much more elaborate systems for data collection and triggering protocols to capture the full extent of ground and structural response.

Four different triggering mechanisms were employed to autonomously record the vibrations produced by construction on West Ann Road. This project, unfortunately, did not involve daily on-site inspection or personnel. Therefore it was of great importance that the instruments autonomously measure the varied and often continuous activity in the vicinity of the structure. Roadway reconstruction and utility improvements involve the services of several different heavy machines, thus the ability to differentiate each machine by waveform and correlated proximity and response are important to the success of any autonomous construction vibration study.

The first triggering mechanism enables long-term data collection. This mechanism triggers the system at a specific, pre-set time to collect one second of data at a frequency of 1000 Hz each hour for the duration of the test. Each one-second of data (1000 points) is then averaged, and becomes one data point, which is employed to define crack response to long-term and environmental effects. Only crack sensors will display long-term response, as velocity transducers are normally at rest and produce no measurable output.

A second triggering mechanism enables collection of ground velocity and crack displacement time histories of vibratory events. Collection begins when any component of ground motion exceeds the “trigger level” of 0.04 ips. While Somat data collection systems have enough memory to collect a significant number of time histories in a given run, the capacity is not infinite, but dependant on the size of PC data card available. For the purposes of this project and the equipment involved, the Somat eDAQ system was programmed to collect a maximum of 8 channels of 100, three-second time histories at 1000 Hz, or 2,400,000 data points. This system not only records individual time histories, but also allows collection of continuous time histories if events last longer than 3 seconds. This ability to continuously record is important in the monitoring of continuous vibrations produced by trenching and vibratory rolling.

The third triggering mechanism enables the collection of absolute peak values on each channel at either 1 or 10 Hz (depending on the situations that will be discussed later in detail) for the duration of the test run. These data allow an assessment of activity level for continuously vibrating sources such as vibratory rollers and trenching machines. This data stream will define the level of activity that produces vibrations less than the event trigger level of 0.04 ips. It can also define the occurrence of single-event activity (the second triggering mechanism) if the activity exceeds the 100-event data system capacity.

The final triggering mechanism involves the collection of significant events by manual trigger. This mechanism allows on-site personnel to collect data while construction occurs immediately adjacent to the test structure. Unfortunately, this manual assistance did not occur, and a summary had to be established through an unanticipated and heavy reliance on triggering mechanism three.

Vibratory crack and ground motion data collected between 9 June 2002 and 18 March 2003 is summarized in Tables 5.1, 5.2 and 5.3. The trackhoe, trencher and two vibratory roller activities produced peak particle velocities (PPV) between 0.03 and 0.456 inches per second and maximum crack response from 0 to 450 micro-inches. Weather conditions varied daily with indoor temperatures ranging between 62° and 86° F, humidity ranging from 14% to 35%, outdoor temperatures ranging from 27° to 124° F, and outdoor humidity ranging from 2% to 86%.

Machine Date Event #	Peak Particle Velocity (in/sec)	Relative Displacement of Structure, $\delta$ , by Method ( $\mu$ in)			Crack 2 Displacement ( $\mu$ in)	Crack 3 Displacement ( $\mu$ in)	Crack 5 Displacement ( $\mu$ in)
		Integration of Velocities $G_{max}$	SDOF METHOD $f_n = 20$ Hz	SDOF Method $f_n = 18-22$ Hz			
Trackhoe 8/29/2002 Event 1	0.034 (L) 0.035 (T) 0.073 (V)	300 (L) 200 (T) 460 (V)	580 (L) 320 (T) 1110 (V)	580 (L) 350 (T) 1190 (V)	44	2.8	16
Trackhoe 8/29/2002 Event 2	0.048 0.038 0.08	270 210 580	670 320 1510	690 350 1460	63	7.3	26
Trackhoe 8/29/2002 Event 3	0.041 0.033 0.076	180 120 330	620 360 1160	590 370 1110	41	8.1	25
Trackhoe 9/13/2002 Event 4	0.037 0.03 0.051	260 190 350	160 250 200	160 230 220	18	2.6	11
Trackhoe 9/13/2002 Event 5	0.034 0.02 0.049	110 60 230	600 190 960	550 220 920	11	2.5	7
Trackhoe 9/13/2002 Event 6	0.049 0.037 0.049	200 100 240	690 310 990	640 280 910	9.4	6	17
Trackhoe 9/13/2002 Event 7	0.043 0.044 0.03	190 130 220	160 150 490	150 150 480	7.1	2.6	6.4
Trackhoe 9/13/2002 Event 8	0.054 0.03 0.061	290 130 440	360 240 590	380 260 580	20	10	11

**Table 5.1 Summary of directional measured peak particle velocity, computed displacements by integration of velocity and single degree of freedom methods and measured crack 2, 3, and 5 displacements for all trackhoe events on 29 August 2002 and 13 September 2002**

Machine Date Event #	Peak Particle Velocity (in/sec)	Relative Displacement of Structure, $\delta$ , by Method ( $\mu$ in)			Crack 2 Displacement ( $\mu$ in)	Crack 3 Displacement ( $\mu$ in)	Crack 5 Displacement ( $\mu$ in)
		Integration of Velocities $G_{max}$	SDOF Method $f_n = 20$ Hz	SDOF Method $f_n = 18-22$ Hz			
Trencher 11/22/2002 Event 1	0.052 (L) 0.052 (T) 0.045 (V)	240 (L) 190 (T) 160 (V)	400 (L) 1010 (T) 600 (V)	430 (L) 940 (T) 510 (V)	13	8	17
Trencher 11/22/2002 Event 2	0.069 0.049 0.048	200 220 210	470 1130 540	430 1040 480	15	12	22
Trencher 11/22/2002 Event 3	0.064 0.044 0.044	190 210 220	420 670 460	410 700 520	14	10	21
Small Roller 11/8/2002 Event 1	0.059 0.019 0.043	270 70 240	480 160 480	500 170 480	15	7	12
Small Roller 11/8/2002 Event 2	0.05 0.025 0.05	230 110 240	390 170 430	400 180 440	23	11	17
Small Roller 11/8/2002 Event 3	0.146 0.063 0.147	560 220 540	950 430 840	920 420 830	48	16	34
Small Roller 11/8/2002 Event 4	0.05 0.02 0.045	250 90 230	460 140 470	470 150 470	13	10	15

**Table 5.2 Summary of directional measured peak particle velocity, computed displacements by integration of velocity and single degree of freedom methods and measured crack 2, 3, and 5 displacements for all trencher events on 22 November 2002 and all small roller events on 8 November 2002**

Machine Date Event #	Peak Particle Velocity (in/sec)	Relative Displacement of Structure, $\delta$ , by Method ( $\mu\text{in}$ )			FFT Excitation Frequency (Hz)	Crack 2 Displacement ( $\mu\text{in}$ )	Crack 3 Displacement ( $\mu\text{in}$ )	Crack 5 Displacement ( $\mu\text{in}$ )
		Integration of Velocities $G_{\text{max}}$	SDOF Method $f_n = 20 \text{ Hz}$	SDOF Method $f_n = 18\text{-}22 \text{ Hz}$				
Large Roller 3/18/2003 Event 1	0.041 (L) 0.046 (T) 0.069 (V)	240 (L) 250 (T) 250 (V)	940 (L) 860 (T) 1380 (V)	1010 (L) 900 (T) 1560 (V)	23	25	14	15
Large Roller 3/18/2003 Event 2	0.071 0.039 0.062	420 240 390	1450 760 1380	1710 900 1570	23	39	15	18
Large Roller 3/18/2003 Event 3	0.193 0.091 0.237	820 480 1140	2400 1620 3740	2950 1940 4280	23	110	40	32
Large Roller 3/18/2003 Event 4	0.229 0.147 0.314	940 740 1950	2850 2230 6640	3450 2650 7950	23	175	42	38
Large Roller 3/18/2003 Event 5	0.106 0.05 0.111	500 300 580	990 520 1040	1010 530 1120	32	25	20	22
Large Roller 3/18/2003 Event 6	0.367 0.138 0.456	1740 610 1960	3570 1020 3390	3840 1060 3440	32	450	70	100
Large Roller 3/18/2003 Event 7	0.083 0.027 0.069	420 140 330	1080 230 630	1080 240 660	32	40	18	21
Large Roller 3/18/2003 Event 8	0.354 0.113 0.373	1490 480 1620	2990 830 3400	3080 840 3560	32	350	55	85

**Table 5.3 Summary of directional measured peak particle velocity, computed displacements by integration of velocity and single degree of freedom methods, excitation frequency by FFT method, and measured crack 2, 3, and 5 displacements for all large roller events on 18 March 2003**

### **Long-term Triggering and Crack Response to Environmental Effects**

Figures 5.1 and 5.2 compare long-term crack response with long-term weather indicators for each crack. Figure 5.1 shows long-term indoor Crack 1 and Crack 3 displacement compared to indoor temperature and humidity, while Figure 5.2 shows outdoor Crack 2 and Crack 5 displacement compared with external temperature and humidity. Temperature, humidity, and crack displacement are plotted on the same time scale. Crack displacement along with indoor and outdoor temperature and humidity were measured every hour for the duration of the monitoring period. Large daily changes in temperature, particularly outdoors, are characteristic of Nevada's desert climate, as is the generally low and relatively slow changing humidity. These weather phenomena correlate well with large, sharp daily changes in displacement of exterior cracks 2 and 5. The interior of the house is air-conditioned, which controls temperature and humidity, which thus reduces weather fluctuations and crack displacements relative to those outside.

Employing the same methodology established for the Connecticut structure in Chapter 2, 24-hour rolling and overall averages were calculated for all four cracks. The average and maximum of each weather descriptor, the frontal, daily, and maximum weather (in this study, seasonal) effects, for the four cracks are presented in Table 5.4. These weather-induced effects on external Cracks 2 and 5 were noticeably larger than for the internal cracks. This disparity in magnitude between internal and external crack displacement is expected, as the interior of the house is temperature and humidity controlled and out of the influence of direct sunlight. The weather-induced crack displacement for all cracks as defined by the three aforementioned descriptors was at least a factor of ten larger than any vibration-induced displacement, and often much more. The external cracks are subjected to greater changes in weather effects as well as the intense heat of the desert sun, and these factors are recognized as 24-hour temperature changes of as much as 50 degrees Fahrenheit and humidity changes of as much as 40 percent.

Several notable weather events occurred during the collection of the long-term data. Significant increases in humidity and decreases in temperature, such as those seen



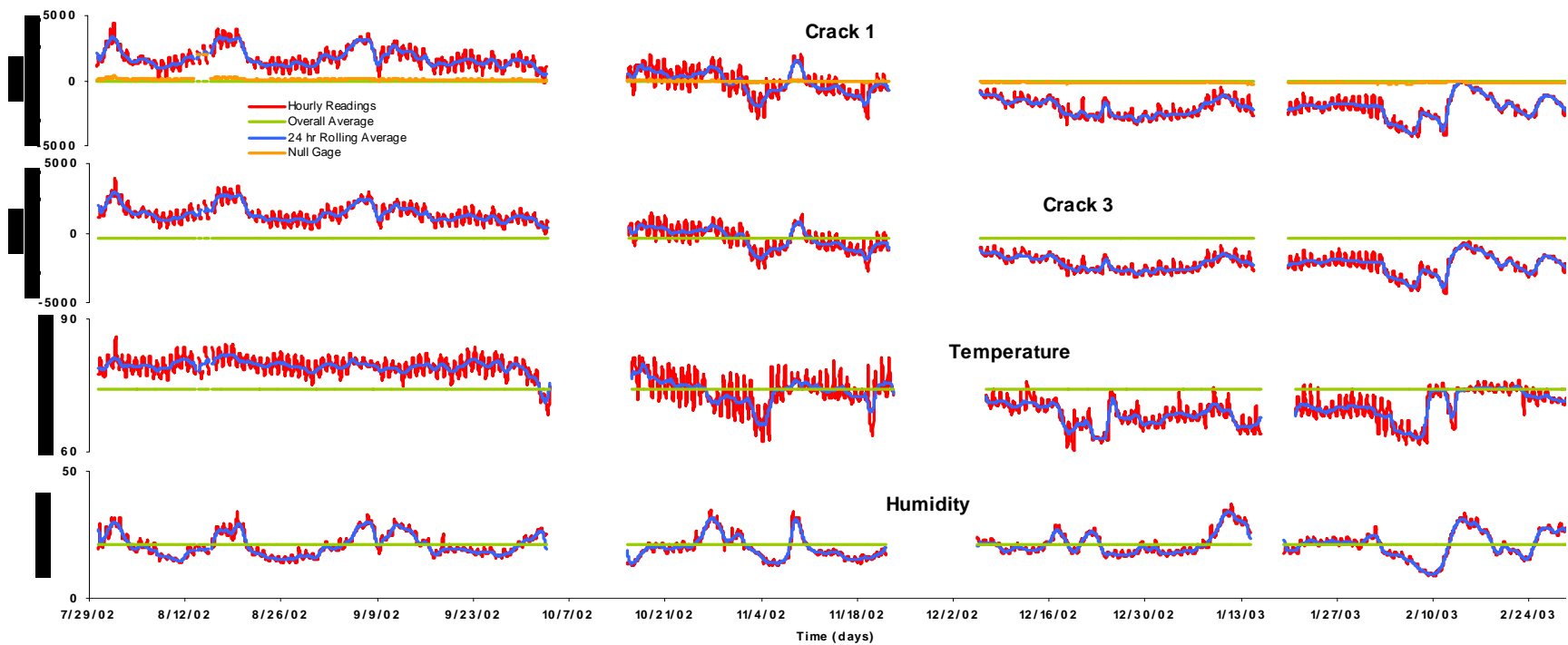


Figure 5.1 Long-term internal Crack 1 and Crack 3 displacement, indoor temperature and indoor humidity versus time

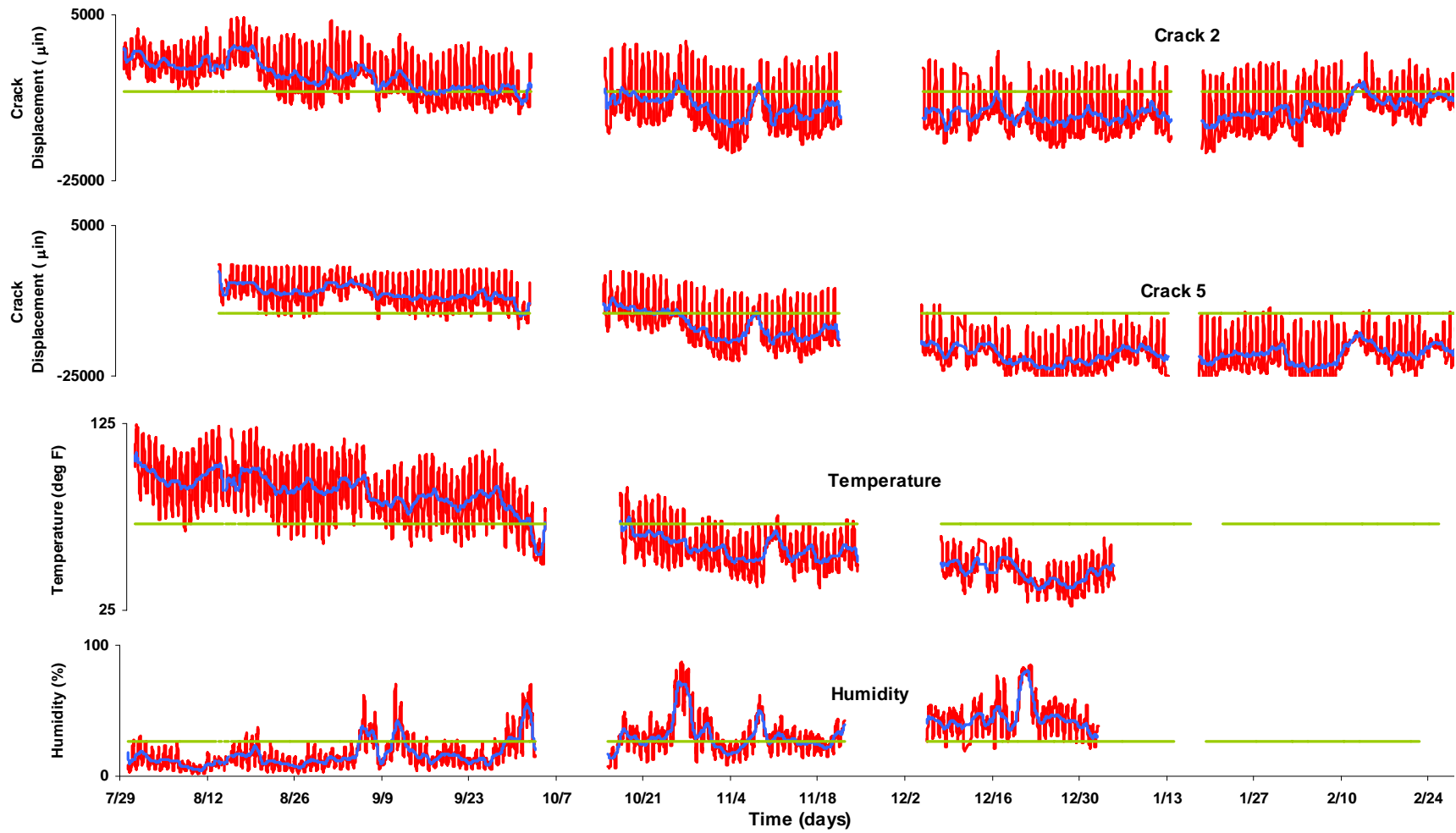


Figure 5.2 Long-term external Crack 2 and Crack 5 displacement, outdoor temperature and outdoor humidity versus time

	Temperature Change (DegF)	Humidity Change	Crack Displacement (μin)	Crack Displacement (μm)
<b>Crack Sensor 2- External South Wall</b>				
<b>Frontal Effect</b>				
Average deviation of 24-hour average from overall average	18.3	12.3	3300	84
Maximum deviation of 24-hour average from overall average	37.9	54.3	8800	224
<b>Daily effect</b>				
Average deviation of field measurement from 24-hour average	9.3	6.3	3200	81
Maximum deviation of field measurement from 24-hour average	45.9	38.2	10700	272
<b>Weather Effect</b>				
Average deviation of field measurement from overall average	19.5	13.8	4600	117
Maximum deviation of field measurement from overall average	71.2	61	13200	335
<b>Construction Effect</b>				
Maximum ground motion (PPV= 0.08 ips Trackhoe)	-	-	63	1.60
Maximum ground motion (PPV= 0.055 ips Trencher)	-	-	15	0.38
Maximum ground motion (PPV= 0.148 ips Small Roller)	-	-	48	1.22
Maximum ground motion (PPV= 0.456 ips Large Roller)	-	-	450	11.43

	Temperature Change (DegF)	Humidity Change	Crack Displacement (μin)	Crack Displacement (μm)
<b>Crack Sensor 3- Hallway Wall</b>				
<b>Frontal Effect</b>				
Average deviation of 24-hour average from overall average	4.6	3.8	1600	41
Maximum deviation of 24-hour average from overall average	11.5	13	3600	91
<b>Daily effect</b>				
Average deviation of field measurement from 24-hour average	1.3	0.9	270	7
Maximum deviation of field measurement from 24-hour average	9.5	23.2	1250	32
<b>Weather Effect</b>				
Average deviation of field measurement from overall average	4.5	4	1600	41
Maximum deviation of field measurement from overall average	14.2	17	4300	109
<b>Construction Effect</b>				
Maximum ground motion (PPV= 0.08 ips Trackhoe)	-	-	10	0.25
Maximum ground motion (PPV= 0.055 ips Trencher)	-	-	12	0.30
Maximum ground motion (PPV= 0.148 ips Small Roller)	-	-	16	0.41
Maximum ground motion (PPV= 0.456 ips Large Roller)	-	-	70	1.78

	Temperature Change (DegF)	Humidity Change	Crack Displacement (μin)	Crack Displacement (μm)
<b>Crack Sensor 5- External West Wall</b>				
<b>Frontal Effect</b>				
Average deviation of 24-hour average from overall average	18.3	12.3	5700	145
Maximum deviation of 24-hour average from overall average	37.9	54.3	13000	330
<b>Daily effect</b>				
Average deviation of field measurement from 24-hour average	9.3	6.3	2100	53
Maximum deviation of field measurement from 24-hour average	45.9	38.2	10000	254
<b>Weather Effect</b>				
Average deviation of field measurement from overall average	19.5	13.8	6100	155
Maximum deviation of field measurement from overall average	71.2	61	15500	394
<b>Construction Effect</b>				
Maximum ground motion (PPV= 0.08 ips Trackhoe)	-	-	26	0.66
Maximum ground motion (PPV= 0.055 ips Trencher)	-	-	22	0.56
Maximum ground motion (PPV= 0.148 ips Small Roller)	-	-	34	0.86
Maximum ground motion (PPV= 0.456 ips Large Roller)	-	-	100	2.54

**Table 5.4 Summary and comparison of Crack 2, 3, and 5 displacement with weather descriptors, vibratory activity and average and maximum temperature and humidity readings**

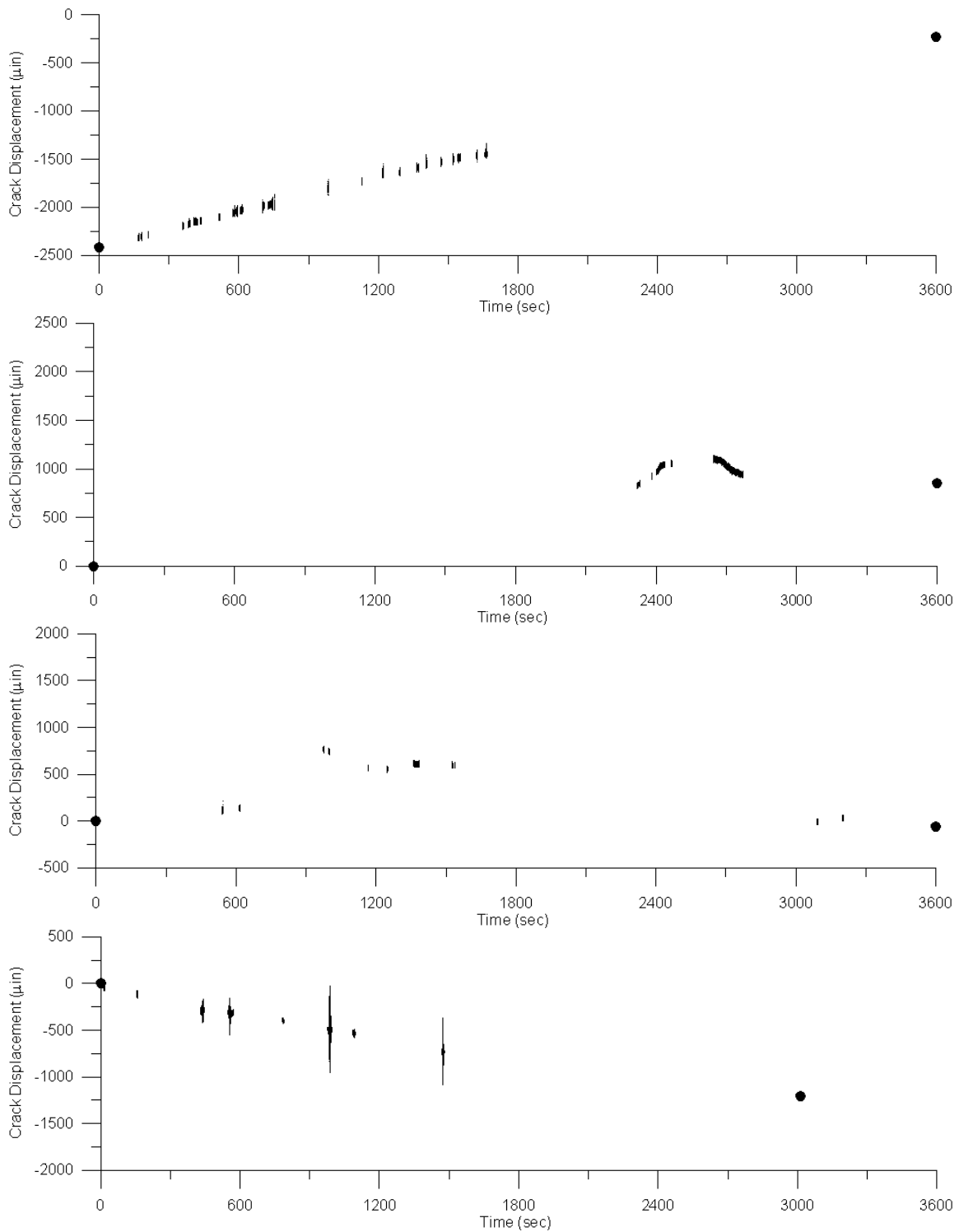
around the 7<sup>th</sup> of September, 27<sup>th</sup> of October and 21<sup>st</sup> of December suggest rainfall events occurred during these time periods. Weather stations in Las Vegas confirm that rainfall events did take place, and as the 24-hour rolling humidity average steadily increased, Cracks 1, 2, and 3 all reacted with correlating dramatic displacements. Crack 5, however, did not experience as relatively significant a reaction to this series of extreme weather phenomena. Table 5.5 summarizes all rain events that occurred between 15 June 2002 and 31 December 2002. 2003 data are not yet available from the National Weather Service.

Date	Rainfall (in)
7/17/2002	0.52
9/5/2002	0.03
9/6/2002	0.01
9/11/2002	0.27
10/2/2002	0.05
10/26/2002	0.1
10/27/2002	0.17
11/30/2002	0.12
12/16/2002	0.01
12/20/2002	0.03
12/21/2002	0.03

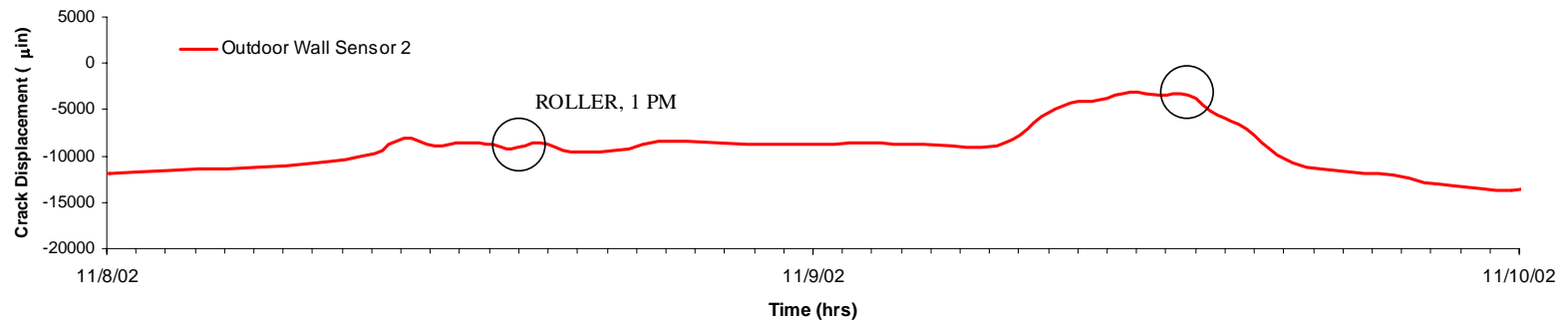
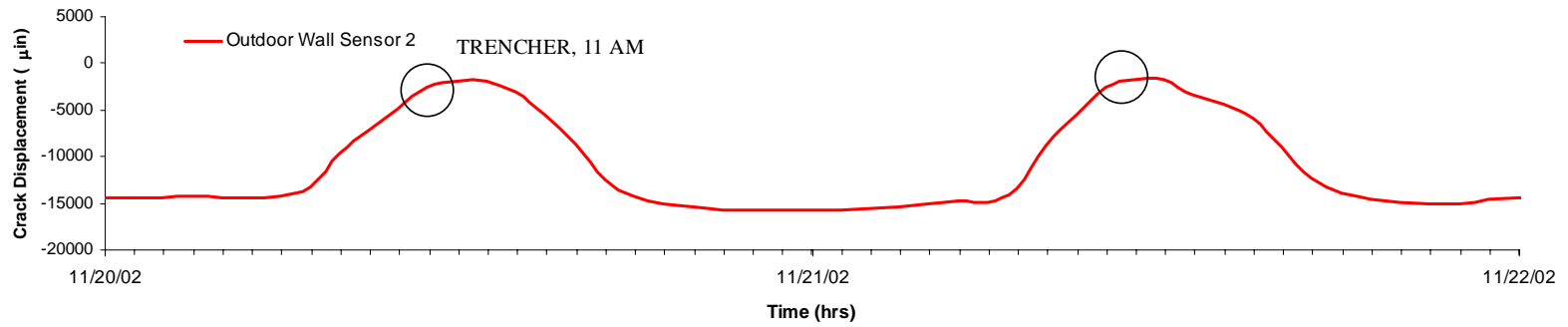
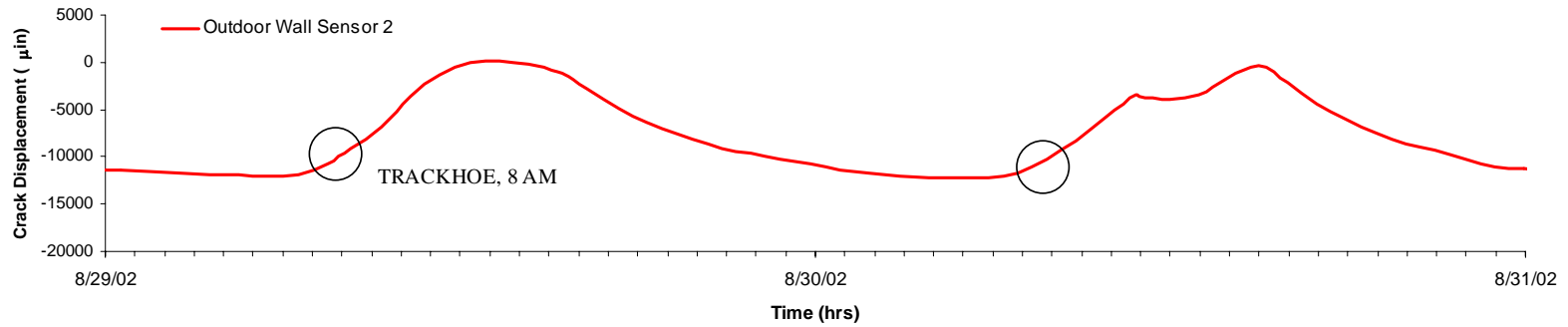
**Table 5.5 Summary of rainfall events during monitoring period in 2002. 2003 data not yet available. (National Weather Service)**

Figure 5.3 combines several long-term and continuous, triggered time history data collection schemes with Crack 2, to illustrate the dominant effect of weather relative to construction-induced vibration for (a) trackhoe excavation (29 August), (b) trencher excavation (22 November), and (c) small vibratory roller compaction (8 November), and large vibratory roller compaction (18 March). Each event period begins and ends with hourly readings (represented by large solid circles). With respect to the trackhoe plot (a), the largest crack displacement within this time period was 63 micro-inches. The difference between the initial crack width and its width one hour later from temperature change, however, was slightly under 2500 micro-inches. This change is equivalent to an accumulating 60 micro-inch increase in crack width *every two minutes*. More importantly, however, this comparison shows external crack displacement induced by weather phenomena to be well over a factor of ten greater than that of the largest crack displacement induced by the trackhoe at an overall peak particle velocity of 0.08 ips.

Figure 5.4 shows the time of day during which the activity in Figure 5.3 were collected. Trackhoe activity (a) was recorded around 8:30 AM, during which the time rate of change in temperature and humidity are near their peak. Trencher data were recorded later in the morning at 11:00 AM and roller data about 12:30 PM when the rate of change was much smaller as the insolation begins to decline. Weather effects typically reverse themselves for the day between the hours of 1 and 3 PM. These changes show the high sensitivity of the cracks on the stucco exterior. The pair of plots in Figures 5.3 and 5.4 show that even though the hourly rate of change in the weather effect is relatively small at that time, vibratory activity of both the trencher and small roller produce far smaller crack displacement than the changes in temperature and humidity. Furthermore, subsequent changes in weather response that same day can be large.



**Figure 5.3 Combined Crack 2 time histories and long-term data triggers showing dominant effect of weather versus vibration induced crack displacements over a one-hour period for trackhoe (top), trencher (middle-top), small vibratory roller (middle-bottom), and large vibratory roller (bottom)**



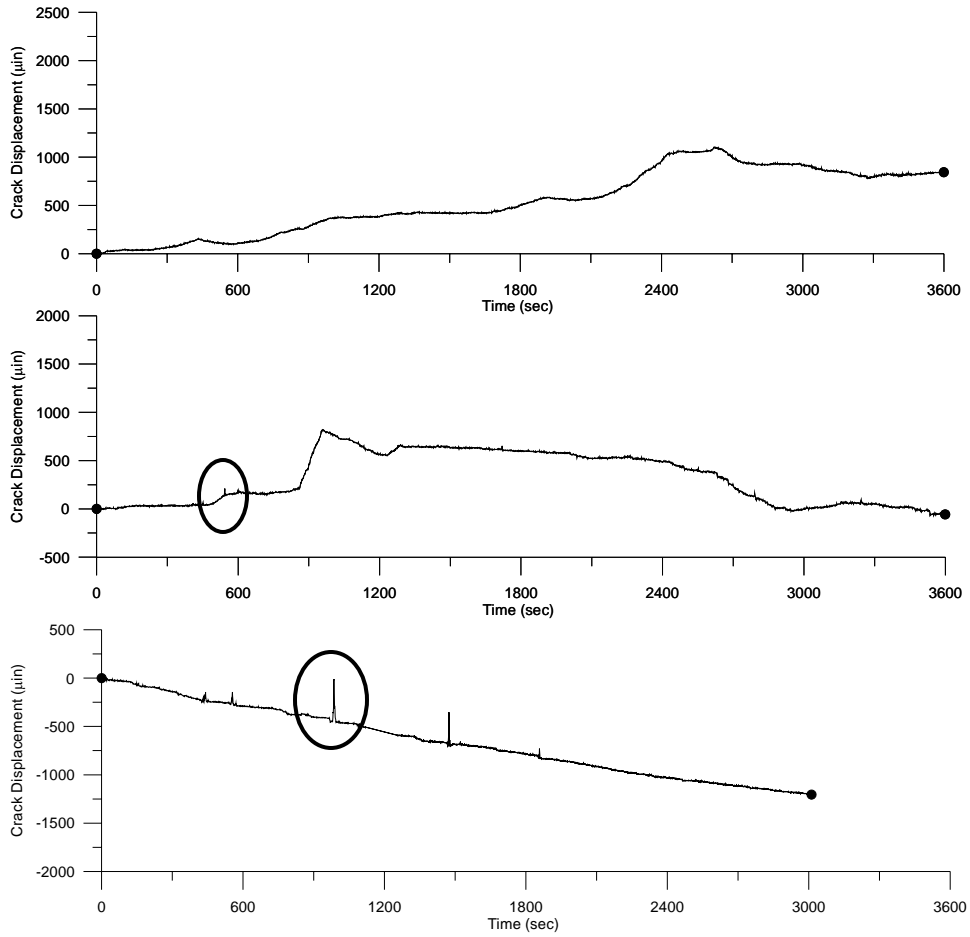
**Figure 5.4 Long-term Crack 2 data showing variability in acquisition periods for trackhoe excavation, trencher excavation, and small roller vibratory compaction**

To measure the regularity of crack displacement, changes in the width of Cracks 2 and 5 were recorded at a rate of 1 Hz, or 1 data point per second, without vibratory excitation. Figure 5.5 presents this data during the same one hour time period as the plots shown in Figure 5.3, under this modified triggering scheme. Two long-term data points are shown at times 0 and 3600 seconds, and the data between variations in crack response. While this and previous studies have shown that recording long-term and threshold triggered vibratory response does an effective job of conveying the dominant effect that weather phenomena have on cracks, these plots display the not insignificant non-vibratory response that may occur between hourly data points. For example, at approximately the 900-second mark of the small roller data in Figure 5.5, there is a significant deviation from the gradual change in displacement of Crack 2. Looking back to Figure 5.3 which presented all recorded transient events exceeding 0.04 ips during this hour of activity, however, shows that whatever phenomenon caused this deviation was not associated with any instantaneous ground motion large enough to trigger the individual event time history recording system. Even when the largest measured ground excitation events are large enough to appear in this data mode, such as those circled on the small and large vibratory roller plots in Figure 5.5, they are still vastly overwhelmed by the effect of non-vibratory response.

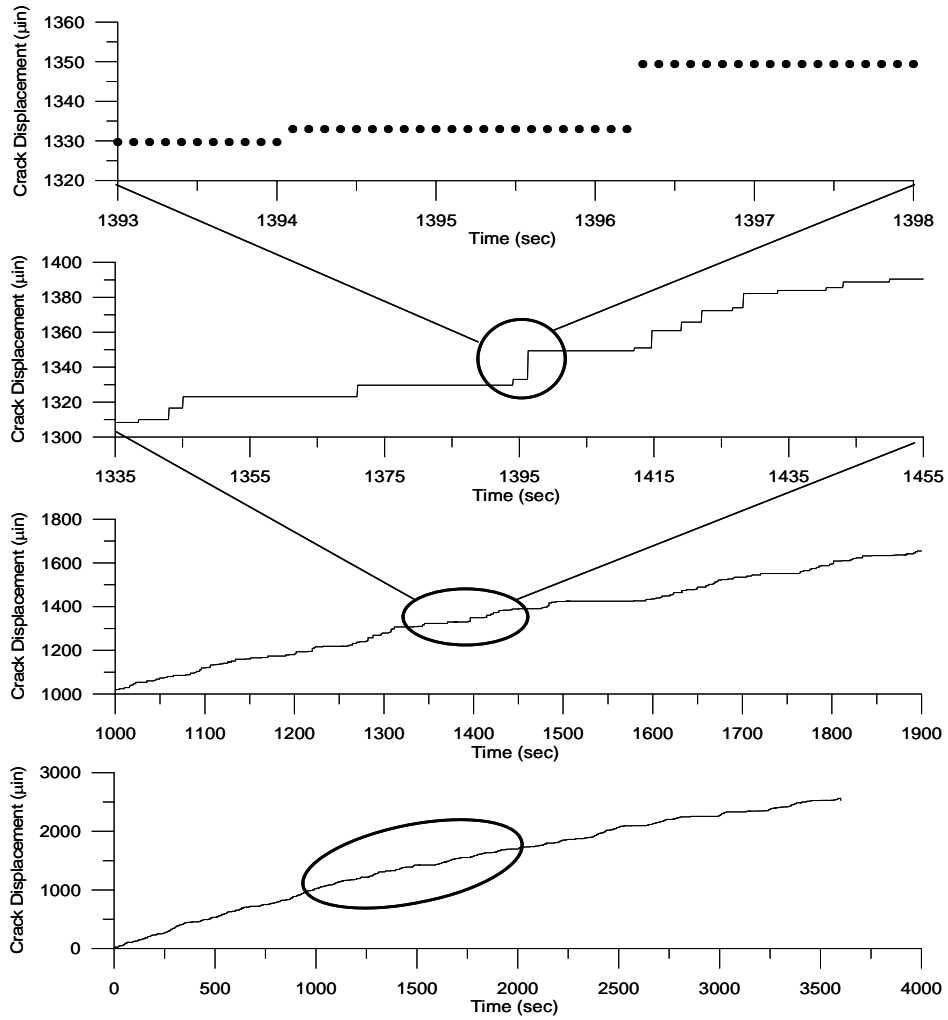
Cracks do not open and close continuously, but rather intermittently over time in a stick-slip fashion. Figure 5.6 (a) shows five seconds of external Crack 5 data recorded at a rate of 10 samples per second (10 Hz), at approximately 8:30 AM on a day when no construction activity significant enough to trigger time history data recording took place in the vicinity of the house. This stick-slip phenomenon may influence the interpretation of vibratory response if it occurs during a transient event. As shown in Figure 5.6 (b), there may be the appearance of “permanent” crack displacement in a recorded time history, or rapid and unexplained crack displacement in continuous data such as that presented earlier for Crack 2 response to the small roller in Figure 5.5. At first one might interpret Figure 5.6 to show that significant crack displacement seen at approximately the 1.2-second mark appears to have been the result of construction machinery induced ground motions. However, it would have occurred without activity in the vicinity of the house at the time of recording. This “stick-slip” effect may create a serious



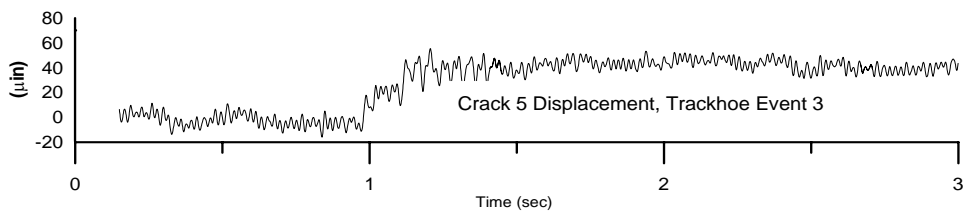
misconception about the nature of vibration crack response unless it is compared to the long-term response, which shows the true environmentally induced response.



**Figure 5.5 Combined long-term and one-hertz continuous data triggering non-vibratory Crack 2 response over the same one-hour time periods as Figure 5.3, for trencher (top), small vibratory roller (middle), and large vibratory roller (bottom). Example vibratory roller events recorded within this time period are circled.**



(a)



(b)

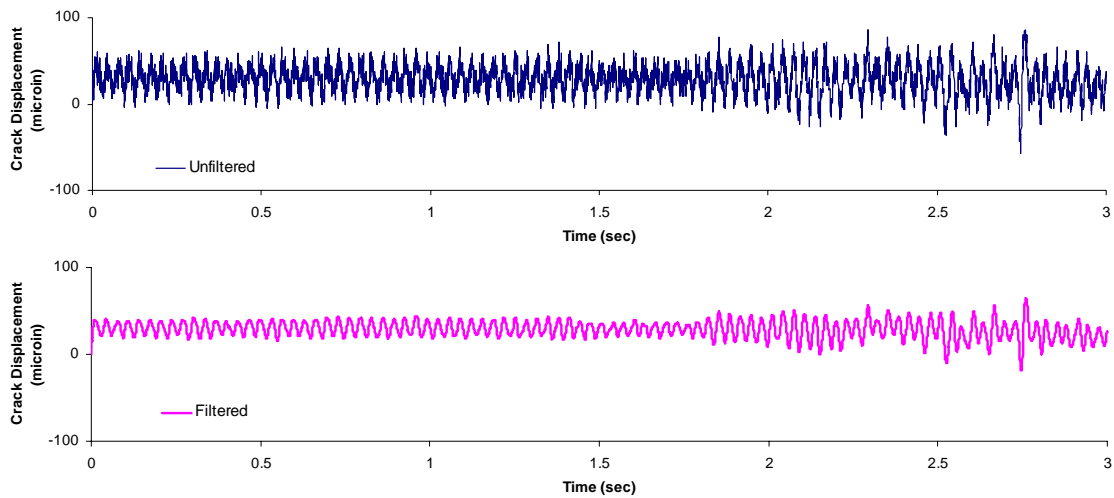
**Figure 5.6 Representation of proposed stick-slip crack displacement effect on external Crack 5 (a) from long-term data taken during a time period without construction activity and (b) trackhoe excavation Event 3 time history on 29 August 2002**

### **Individual Event Triggering and Transient Response**

Construction vibration induced ground motion and the resulting crack response were obtained whenever peak particle velocities exceeded 0.04 ips. They were produced by the various construction activities outlined in Chapter 4 during the widening and reconstruction of Ann Road. These activities have been divided into three primary categories, and again into individual, specific activities and events. The four principal categories are:

- General construction and miscellaneous response
- Trackhoe trenching activities involved with the installation of a 12-ft x10-ft reinforced concrete storm culvert,
- Trencher activities involved with the installation of an 8-inch PVC sanitary line,
- Vibratory compaction from trench backfill and sub-grade activities

A 10 micro-inch high-frequency electrical noise produced by the unusually large volume of machine and radio wave traffic in the vicinity of the data acquisition system and an occasionally low vibratory crack response presented a significant challenge in the discrimination of small measured crack response to construction vibrations by machines at typical distances. As a result a *Butterworth* low pass data-filter was employed to eliminate all crack displacement time history components with frequencies greater than 60 Hz. The filtering scheme was not necessary for ground motions. Filtering only improved the discern ability of transient response for external Cracks 2 and 5 and internal Crack 3 for trackhoe and trencher excavation activities; the vast majority of responses in internal Crack 1 were still too small to identify above the remaining noise produced at nearly the same frequency. Figure 5.7 is an example of an unfiltered time history for External Crack 5 responding to small vibratory roller excitation at approximately 46 feet, with its corresponding filtered time history plotted below. The crack displacement can be easily deciphered in this time history, but the figure clearly demonstrates the significant loss in amplitude associated with these necessary filtering activities. This particular event will be explored in detail later in the chapter, but for future reference, any post-filter crack displacement less than 10 micro-inches will be henceforth be considered merely a product of filtered noise, and negligible.



**Figure 5.7 Three seconds Unfiltered (top) versus filtered (bottom) Crack 2 displacement data for 3 seconds and peak of vibratory roller compaction event #3 on 8 November 2002**

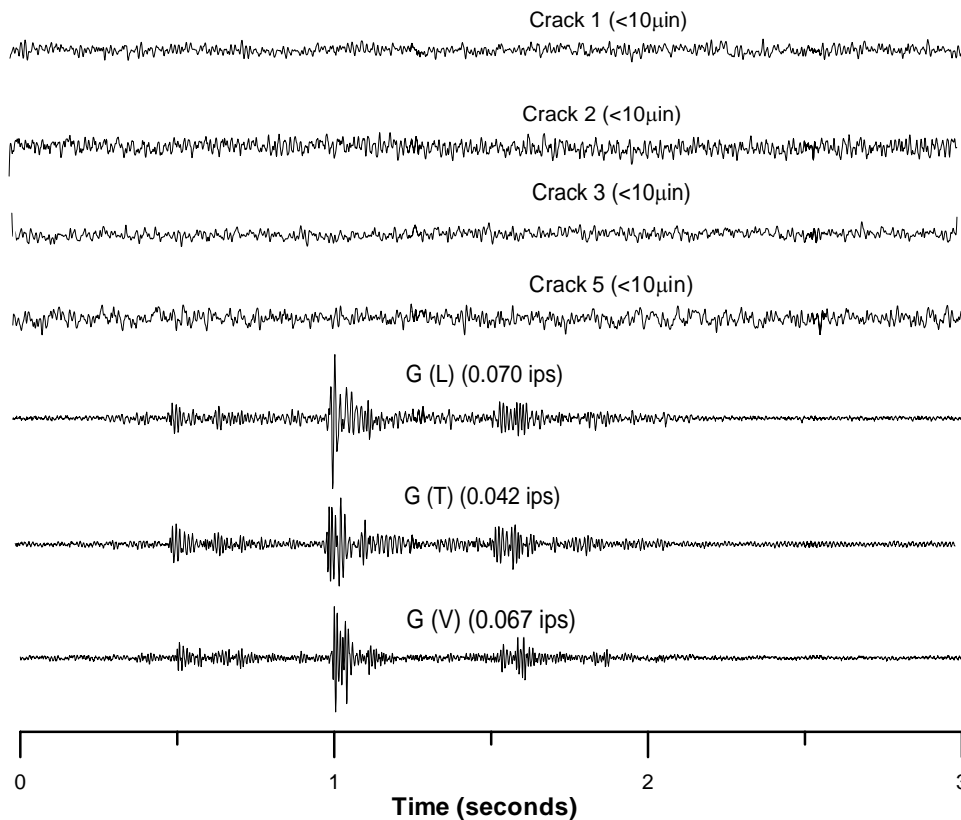
### Miscellaneous Activities

In the introduction to this chapter it was noted that one of the inherent challenges to monitoring construction vibrations versus blasting vibrations is the relatively continuous nature of construction activity necessitating unusual triggering schemes. The number, size and specifics of all machinery active in one location on a standard construction site is far too random to identify all sources without continuous on-site inspection. During the monitoring of construction on Ann Road, several construction machines were active near the test house that randomly triggered the system, but did not create ground motions or crack responses significant enough to present in detail. Graders, front-end loaders, and forklifts are examples of these machines.

During the excavation activities to be discussed later, a large number of spurious events shown in Figure 5.8 occurred. All had the same signature pattern: small/large/small amplitudes at approximately half-second intervals. The top four time histories are crack displacement time histories of cracks 1, 2, 3, and 5, which display no response. The bottom three represent longitudinal, transverse, and vertical ground velocity time histories of this event. The significant portion of this ground motion lasts a mere two tenths of a second and results in essentially no crack displacement from any of

the sensors. The exact cause of these frequent events is not entirely known, but could possibly be personnel walking over the geophone.

The greatest challenge these spurious events pose for vibration monitoring is their occurrence during the same event history as other monitored activities. This dual arrival makes it difficult to decipher what degree of ground motion and structural response are results of the desired activity, rather than these miscellaneous activities. Separation of these two activities requires the examination of dominant frequencies within the time histories and will be examined further later in this chapter.



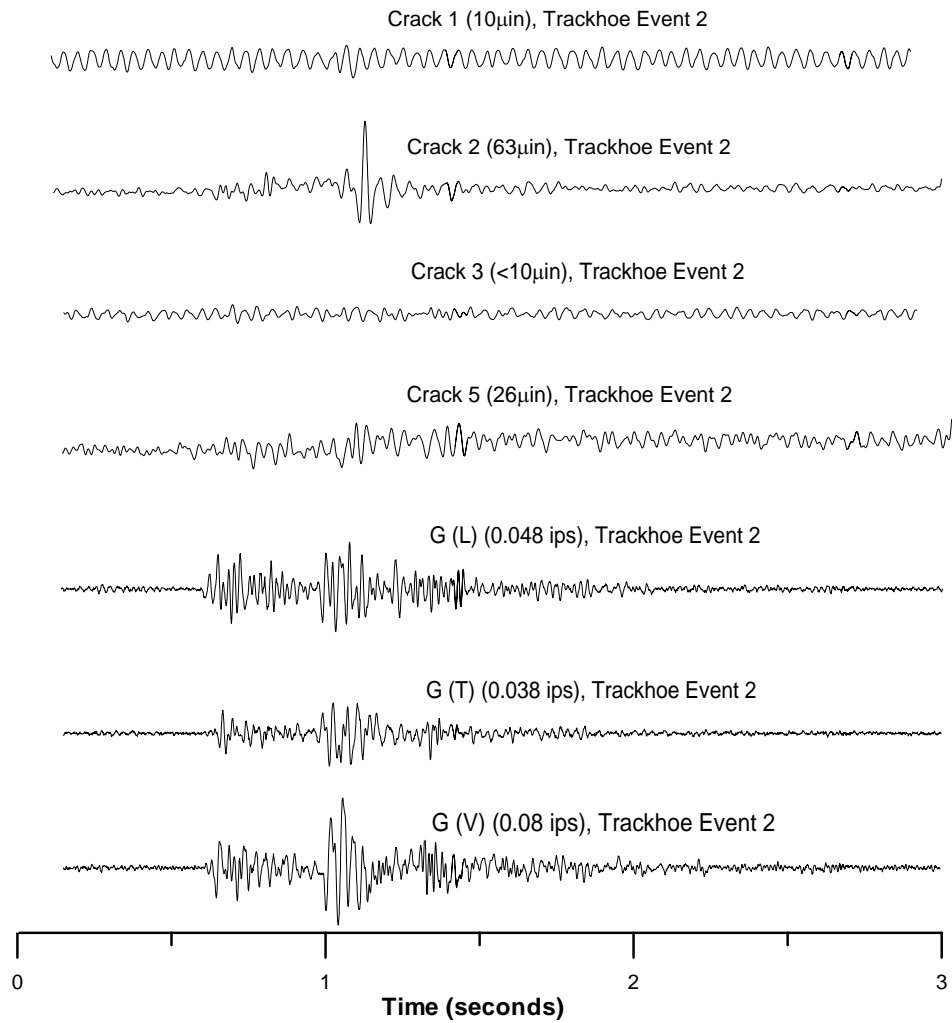
**Figure 5.8 Time histories of Crack 1, Crack 2, Crack 3 and Crack 5 from miscellaneous trigger event in August 2002 compared to longitudinal, transverse and vertical ground excitation**

### Trenching Activities

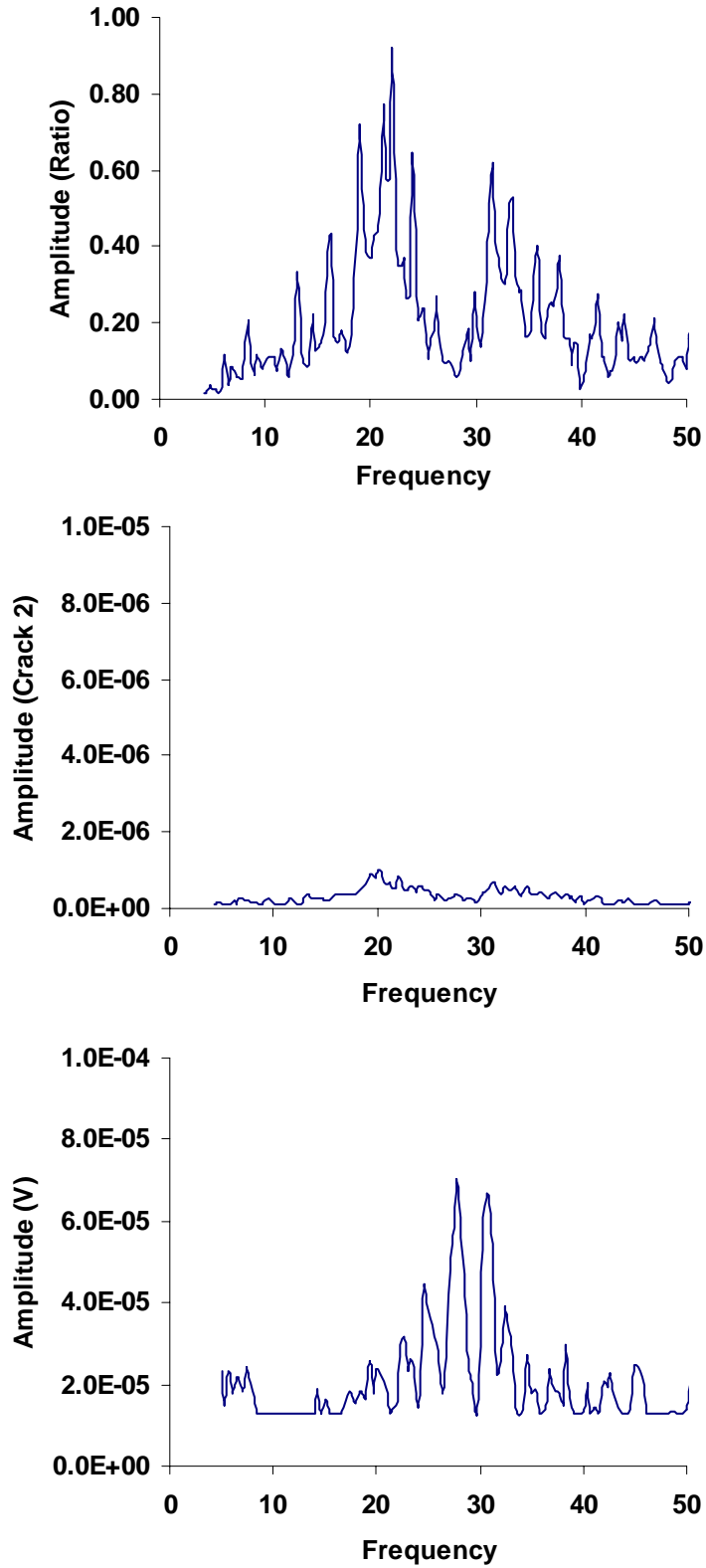
Figure 5.9 shows time histories of excitation ground motions and the associated internal and external crack response produced by excavation of a 12 foot wide trench with an Hitachi 1200 EX Super track-hoe (shown in Figure 4.4 (a)) at approximately 8:30 AM on August 29<sup>th</sup>, 2002. The centerline of the trench is oriented parallel to and is approximately 46 feet in front of the house. The top four graphs in Figures 5.9 show crack displacement time histories of cracks 1, 2, 3, and 5. The bottom three time histories show longitudinal, transverse, and vertical ground velocity time histories. This particular excitation produced a peak particle velocity of 0.08 ips in the vertical direction. Crack 2, the external south wall crack nearest the excavation, experienced the greatest displacement of 63 micro-inches zero-to-peak while Crack 5 experienced 26 micro-inches zero-to-peak. Internal Cracks 1 and 3 showed little or no discernable response after filtering of general instrument and electrical noise is performed. The attenuation relationship presented in Figure 4.9 shows this peak particle velocity matches that expected from the trackhoe.

The natural frequency of the wall is calculated employing the Frequency Fourier Transform (FFT) response spectra method described in Chapter 2. Crack 2, the outside south wall sensor, was selected because it is closest to the construction activities and the most responsive, and as a result has the highest signal to noise ratio. Figure 5.10 shows the longitudinal FFT spectra for the Crack (b), the longitudinal component of ground displacement (c), and the ratio (a), which show a dominant frequency of 18 to 22 Hz for this wall.

The SDOF response spectrum for Event 2 on 29 August is presented as Figure 5.11. A damping coefficient of 5% was assumed for all spectra based on average values from previous studies. The approximate dominant frequency of the Las Vegas wall is 20 Hz, therefore the estimated displacement of the structure relative to the ground motion produced by this event was 1510  $\mu$ in in the longitudinal direction. Single degree of freedom analyses were performed on all longitudinal, transverse and vertical ground motions produced by the excavation activities. These spectra are used to predict cracking potential in structures subjected to excitation ground motions and are described in detail in Chapter 2.

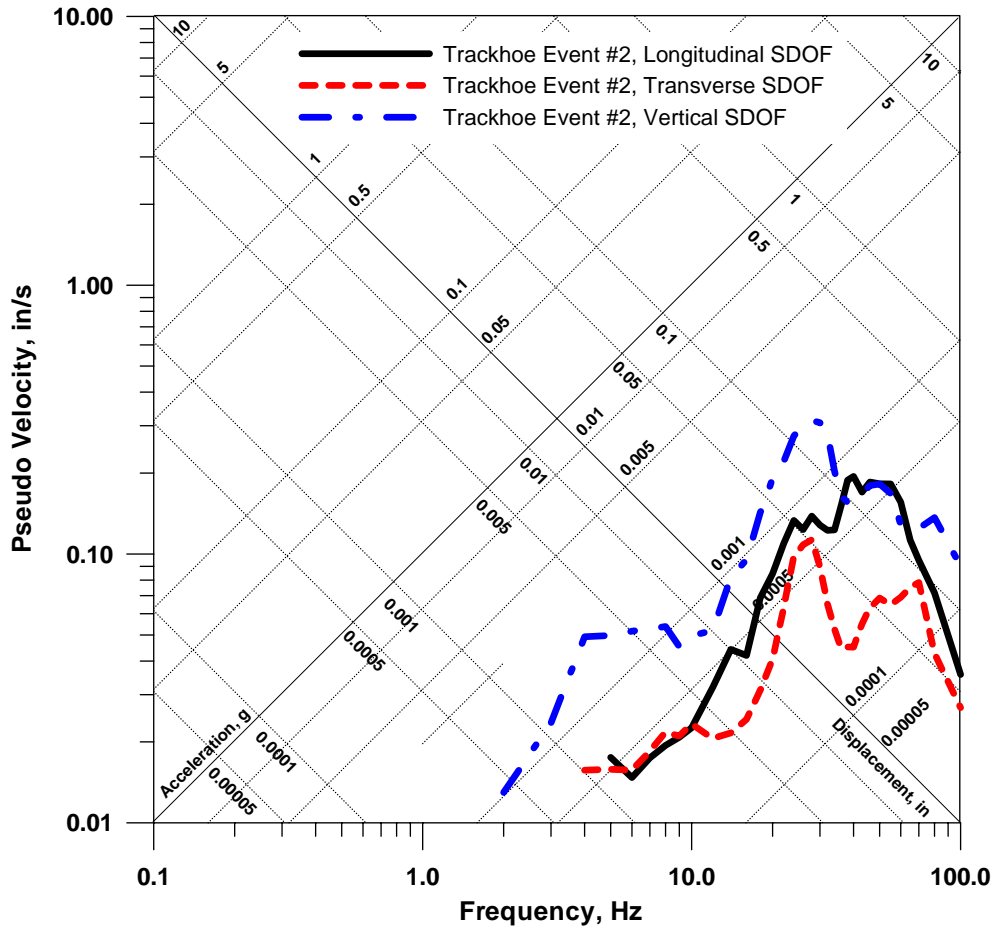


**Figure 5.9 Time histories of Crack 1, Crack 2, Crack 3 and Crack 5 from trackhoe excavation event 2 on 29 August 2002 compared to longitudinal, transverse and vertical ground excitation**



**Figure 5.10** Frequency Fourier Transform histogram of Crack 2 divided by vertical ground displacement ratio (top), measured Crack 2 displacement (middle), and calculated vertical ground displacement (bottom) for trackhoe event #2 on 29 August 2002

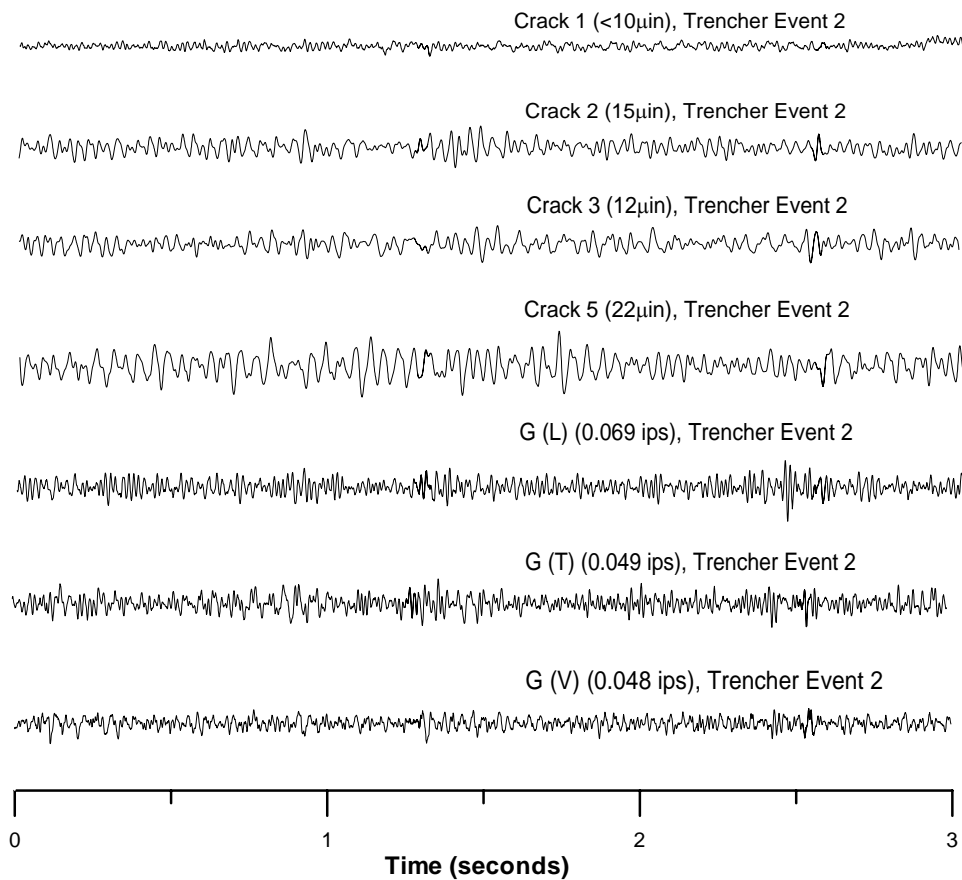




**Figure 5.11 Longitudinal, transverse, and vertical single degree of freedom response spectra for trackhoe event #2 on 29 August 2002, showing relative displacement of a 20 Hz structure**

Figure 5.12 shows time histories of excitation ground motions and the associated internal and external crack response associated with the excavation of a trench for an 8” PVC sanitary sewer line with the Tesmec TRS-1175 trencher shown in Figure 4.4 (b) at approximately 11:00 AM on November 21<sup>st</sup>, 2002. The centerline of this trench lies parallel to and approximately 32 feet in front of the house. The top four graphs in Figures 5.12 show crack displacement time histories of cracks 1, 2, 3, and 5. The bottom three time histories show longitudinal, transverse, and vertical ground velocity time histories. These trenching activities constituted the first “continuous” activity in the vicinity of the structure and did not produce particularly large ground motions. Figure 5.12 shows the greatest overall peak particle velocity resulting from the trencher activity of 0.069 ips. From the attenuation study presented in Chapter 4, trencher peak particle

velocities from 30 to 35 feet away should average between 0.01 and 0.03 ips, with occasional spikes similar to those seen throughout the velocity time histories in Figure 5.12. It is theorized that these spikes are a result of the trencher cutting its way through a deposit of caliche, which produces greater vibration. All of the time histories recorded for the trencher are continuous, but very choppy and unharmonious, which is to be expected from chain trenching excavations (Dowding, 1996). Crack displacements were smaller than those of the trackhoe and rollers. A maximum of  $17\mu\text{in}$  was measured from Crack 2,  $12\mu\text{in}$  from Crack 3 and  $23\mu\text{in}$  from Crack 5. This relatively small response is expected due to the relatively small ground motions.



**Figure 5.12 Time histories of Crack 1, Crack 2, Crack 3 and Crack 5 from trencher excavation event 2 on 21 November 2002 compared to longitudinal, transverse and vertical ground excitation**

### Vibratory Compaction

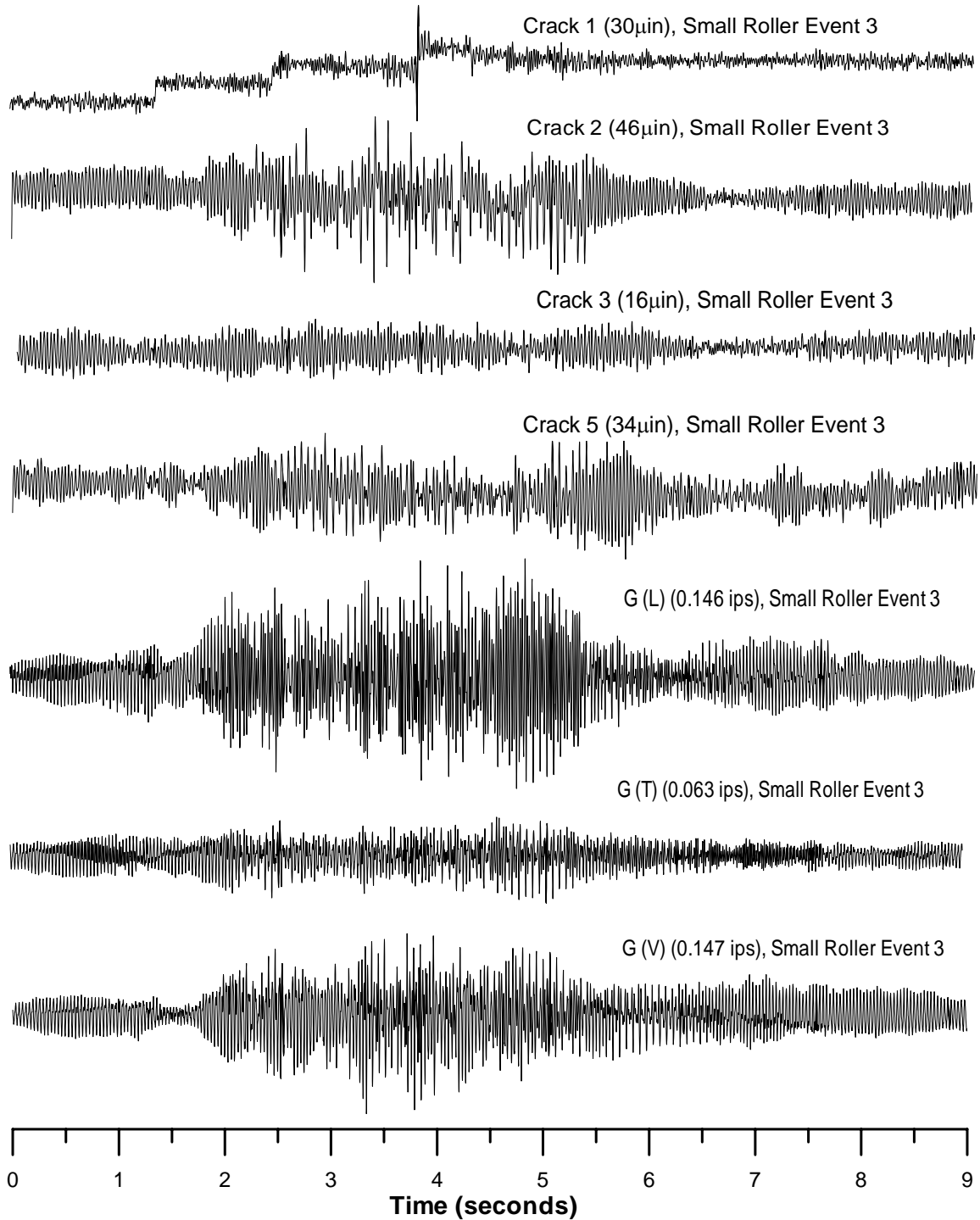
A variety of data were collected for vibratory roller activities in the vicinity of the test house. As reported in Chapter 4, two different rollers were employed for this project; a small roller to compact granular backfill in the culvert trench excavated by the trackhoe, and a larger roller employed to compact granular roadway sub-grade prior to application of the asphalt surface. Both rollers provided continuous time histories that last significantly longer than blasting or trackhoe activities, and are much more harmonic and smooth compared to the chain-excavation of the trencher. Therefore, roller events are not assumed to last 3 seconds, as are the trackhoe and trencher events, but extend as long as the peak particle velocity in any direction exceeds the “significant peak” threshold of 0.04 ips for the small roller and 0.08 ips for the large roller.

Figure 5.13 (a) presents response of internal Cracks 1 and 3, external Cracks 2 and 5 and excitation ground motion velocity time histories produced by vibratory compaction of the granular backfill within the 12-foot wide culvert trench. Compaction was performed with the Dynapac single-drum vibratory roller shown in Figure 4.5 (a). This activity took place during the early afternoon on November 8<sup>th</sup>, 2002. The top four time histories represent crack displacement time histories for the four cracks, and the bottom three show the corresponding excitation ground velocities. This event produced a peak particle velocity of 0.147 inches per second (vertical) and resulted in a peak overall crack displacement of 46 $\mu$ in from external Crack 2. External Crack 5 sustained a displacement of 34 $\mu$ in, Crack 3 displacement of 16 $\mu$ in, and Crack 1 experienced a stick-slip displacement of 30 $\mu$ in, described earlier.

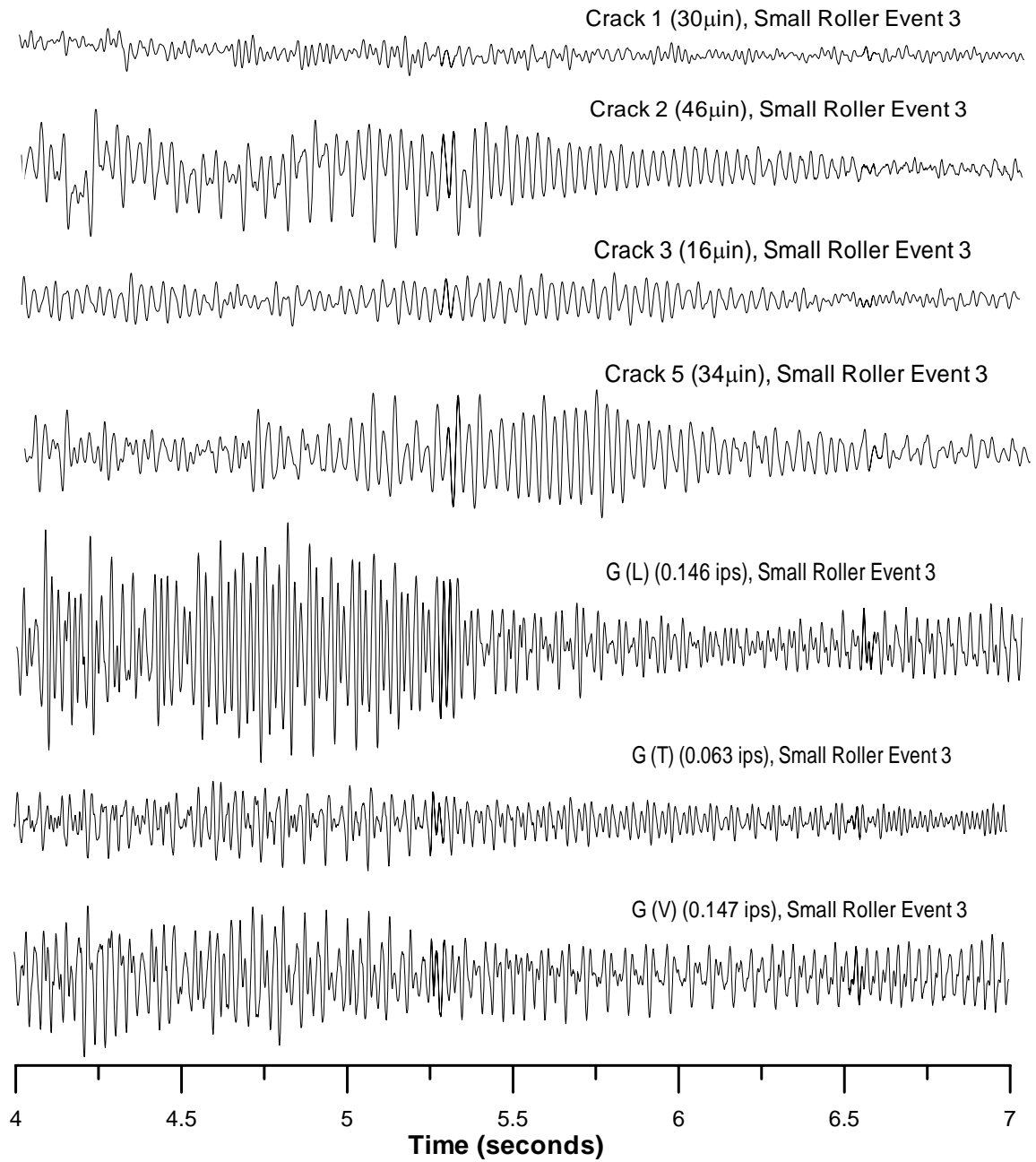
Many of the time histories measured during vibratory compaction monitoring show unusual changes and overlaps in particle frequencies when examined closely. The three seconds of all Event 3 time histories in Figure 5.13 (a) with the largest peak particle velocities and crack displacements are shown in Figure 5.13 (b). They show a distinct time variation in frequency and amplitude that may have resulted from approaching or run-up. At the 1.5-second mark, the time histories change from a relatively harmonious 0.025 to 0.05 ips longitudinal amplitude, 32 Hz signal to a 0.15 ips amplitude with overlying 32 and 48 Hz frequencies. During this time interval, displacements of external Cracks 2 and 5 also increase significantly from a relatively harmonic 10 $\mu$ in peak-to-peak,

to their maxima of  $46\mu\text{in}$  and  $34\mu\text{in}$ , respectively. At approximately the 7.5-second mark, or 1.5 seconds before the end of the time history, the amplitudes decline, the 48 Hz frequency drops out and the time history returns to its original state. The importance and origin of the run-up phenomena is discussed in Dowding (1996).

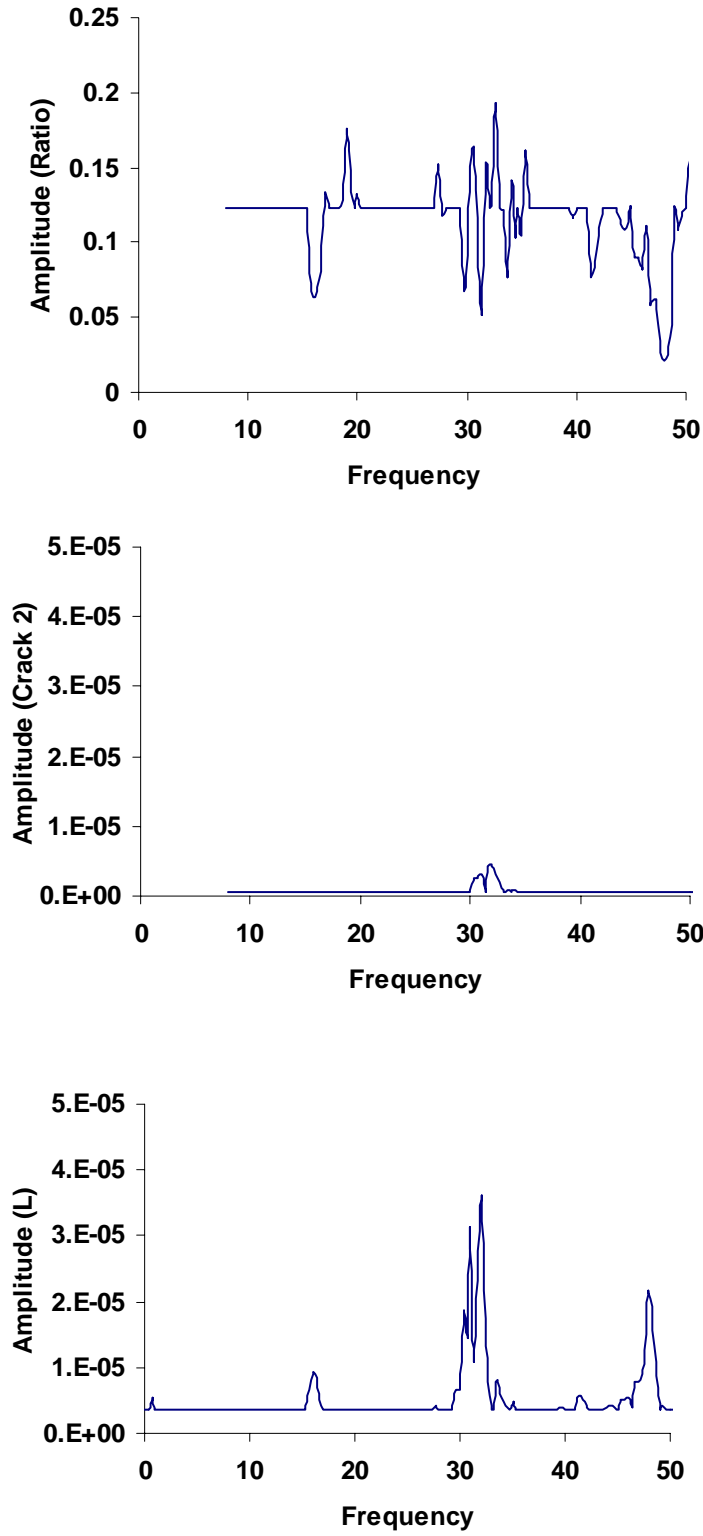
Figure 5.14 shows the fourier frequency spectra of Crack 2, longitudinal integrated ground velocity (displacement) and Crack 2 divided by long displacement for this event, and Figure 5.15 is the longitudinal, transverse and vertical SDOF spectra for this event. From the ground displacement FFT spectrum, it is obvious that, along with the dominant vibratory roller frequency of 32 Hz, there is also a significant frequency at 48 Hz. From the Crack 2 spectrum, however, there is no response from the crack at 40 Hz. The ratio spectrum of Figure 5.14, Crack 2 divided by the ground motion, confirms that at 48 Hz the spectrum is below the base line, meaning that the ground motion contains these dominant frequencies, but the crack is not responding.



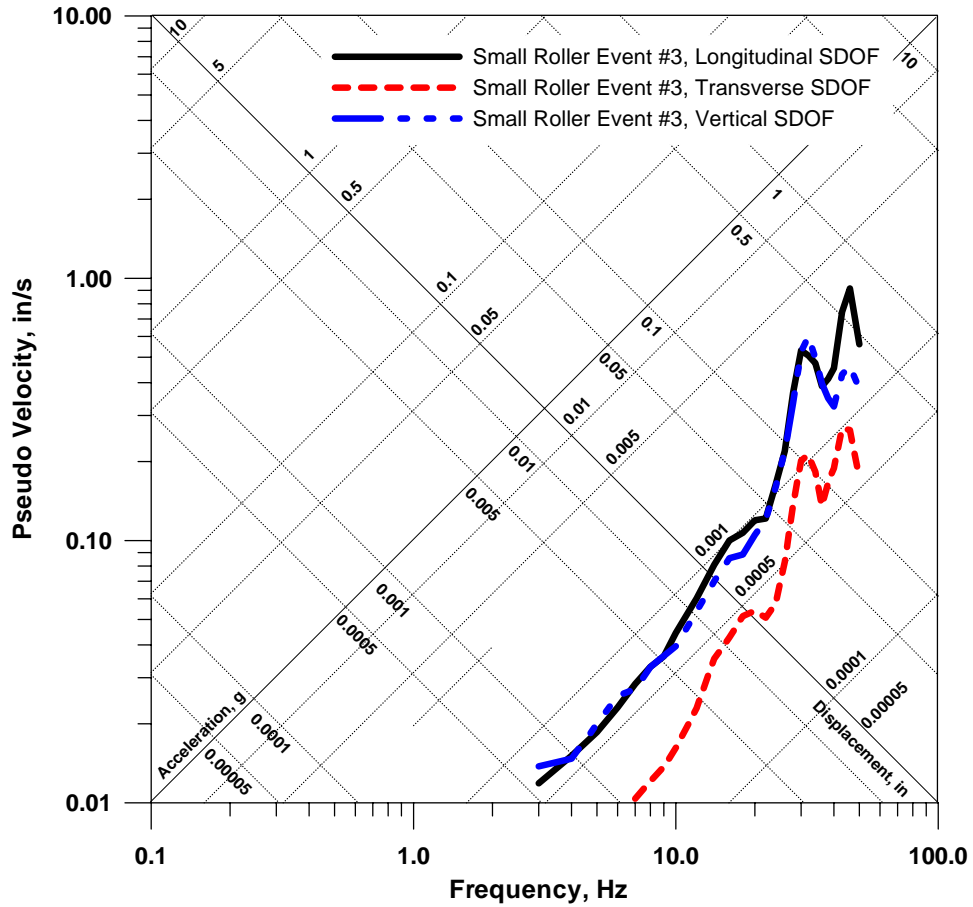
**Figure 5.13 (a) Time histories of Crack 1, Crack 2, Crack 3 and Crack 5 from granular trench backfill small single-drum roller compaction Event #3 on 8 November 2002 compared to longitudinal, transverse and vertical ground excitation ground motion**



**Figure 5.13 (b) 3-second magnification of granular trench backfill small single-drum vibratory roller event 3 time history showing Crack 1, Crack 2, Crack 3, Crack 5, and longitudinal, transverse and vertical ground motion waveforms**



**Figure 5.14 Frequency Fourier Transform histogram of Crack 2 displacement divided by longitudinal ground displacement ratio (top), measured Crack 2 displacement (middle), and calculated longitudinal ground displacement (bottom) for small vibratory roller event #3 on 8 November 2002**



**Figure 5.15 Longitudinal, transverse, and vertical single degree of freedom response spectra for small roller event #3 on 8 November 2002**

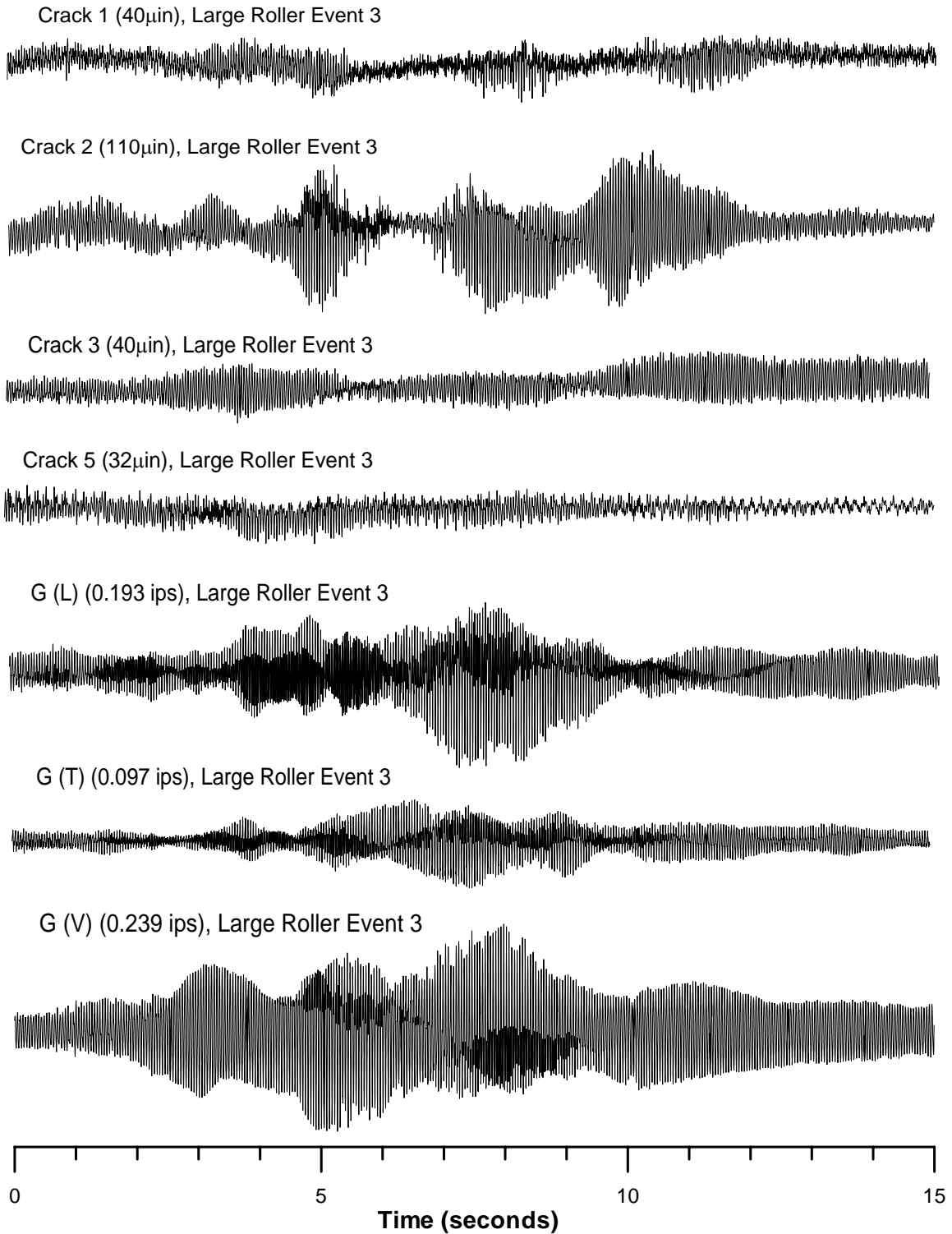
Figure 5.16 presents response of internal Cracks 1 and 3 displacement, external Cracks 2 and 5 displacement and excitation ground motion time histories for the vibratory compaction of stone sub-grade in front of the house. Compaction of the sub-grade was conducted with the Ingersall-Rand (IR) Pro-Pack series SD115 soil compactor shown in Figure 4.5 (b). The top four time histories represent crack displacement time histories for the four cracks, and the bottom three show the corresponding excitation ground velocities. A 3-second waveform magnification of Crack 1, Crack 2, Crack 3, Crack 5 and tri-axial ground motion with the highest peak particle velocities presented in Figure 5.16 (a) are shown in Figure 5.16 (b). This event, and all sub-grade compaction monitoring, occurred the afternoon of 18 March 2003.

The events recorded from the IR soil compactor were conducted at distances of 8, 31, and 55 feet parallel to the house, and varied in exposure time. The peak particle

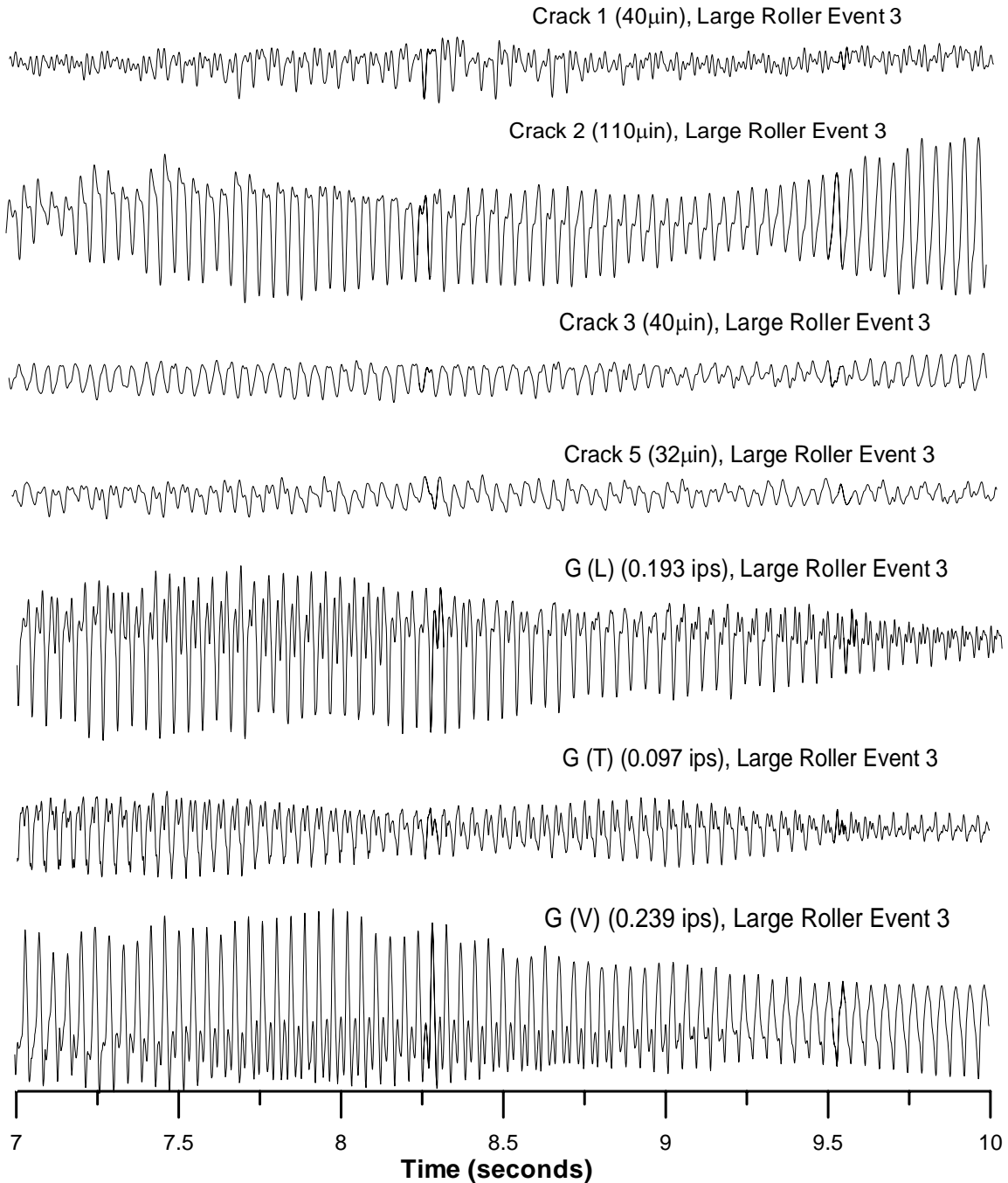


velocity for event 3 was 0.237 ips in the vertical direction, while the events varied in PPV from 0.039 to 0.456 ips. The largest PPV occurred in the longitudinal or vertical directions, and the lowest in the transverse direction. Crack displacements varied greatly, with Crack 2 responding the most. Crack 2 displacement for event 3 in Figure 5.16 (a) was 110 $\mu$ in, Crack 3 displacement was 40 $\mu$ in, and Crack 5 displacement 32 $\mu$ in. Crack 2 displacement for all the events varied from 25 $\mu$ in to 450 $\mu$ in. The largest response was produced by activity within 10 feet of the house and was the largest response produced by any construction activity.

These events consist of time histories recorded at two different frequencies (23.5 Hz and 32 Hz) and two “nominal amplitude”, or centrifugal force levels. The frequency of vibration is controlled by a dial located next to the operator, and varies from approximately 18 to 32 Hz. The nominal amplitude adjustment is made via a switch, also located next to the operator, and has only two settings. Adjusting this setting will change the amount of energy released into the ground, and therefore allow the contractor to control the level of vibration and peak particle velocities in the vicinity of the house. Figure 5.17 shows the SDOF response spectra for this event. From the time history and response spectra the same 48 Hz overlying high-frequency pulse can be seen during the higher amplitude ground motions that was first presented under the small roller. When the ground motions are reduced, the high-frequency pulse again disappears. These frequency effects, and the large roller FFT, will be presented in further detail at the end of the chapter.



**Figure 5.16 (a) Time histories of Crack 1, Crack 2, Crack 3 and Crack 5 from granular sub-grade large single-drum roller compaction Event #3 on 18 March 2003 compared to longitudinal, transverse and vertical ground excitation**



**Figure 5.16 (b) 3-second magnification of granular sub-grade large vibratory single-drum roller compaction event 3 time histories showing Crack 1, Crack 2, Crack 3, Crack 5, longitudinal, transverse and vertical ground motion waveforms**

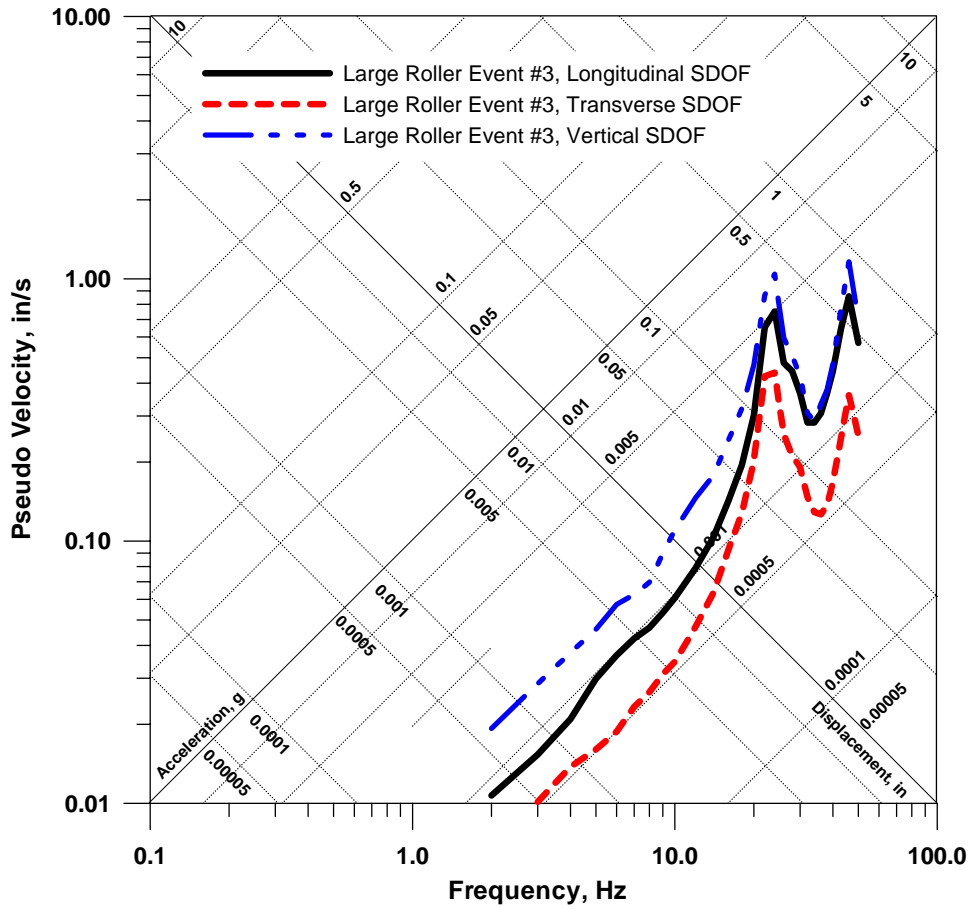


Figure 5.17 Longitudinal, transverse, and vertical single degree of freedom response spectra for large roller event #3 on 18 March 2003

### **Comparisons of Traditional Motion Controls with Measured Crack Displacements**

Since measurement of crack displacement is a new approach to assessing the effect of construction vibrations, it is important to understand how the traditional controls of ground and structural motions correlate to crack displacement. To investigate the correlation, maximum measured response of Cracks 2, 3, and 5 produced by trackhoe, trencher and vibratory roller construction vibrations are compared in Figures 5.18, 5.19, and 5.20 to traditional ground motion controls of peak particle velocity, ground displacements via integrated velocity, and computed structure/wall displacement. Figure 5.18 shows trackhoe activity, 5.19 shows trencher activity, and 5.20 shows vibratory roller compaction from both machines. Displacement calculations for integration of velocities and single degree of freedom follow the methods outlined in Chapter 2. Crack 1 did not show enough consistent response outside the static noise level ( $\approx 10\mu\text{in}$ ) after filtration to warrant correlation consideration. The geodynamic controls considered for correlation are those also traditionally employed in blasting studies.

As described in Chapter 2, analyzing SDOF response spectra of blast and construction induced ground excitations relative displacements of structures can be estimated at various dominant frequencies. Two approaches to comparing SDOF relative displacements to measured crack displacements are employed. The first approach is to find the relative displacement associated with the dominant frequency of the structure or wall, in this case approximately 20 Hz. Due to the fact that the dominant frequency is not universally 20 Hz, but tends to float about that value, the second approach is to take an average SDOF relative displacement between 18 and 22 Hz.

Correlation of crack response with ground motion is a function of crack location, planar direction and the machine creating the vibration. External Crack 2 and internal Crack 3 lie in the plane of longitudinal motion and External Crack 5 lies in the plane of transverse motion. Crack 2 was the most responsive to the trackhoe and vibratory roller motion, while Crack 5 was the most responsive to the trencher. Crack 3 was consistently the least responsive of all Cracks subject to correlation. Table 5.6 summarizes all correlation and regression coefficient data for peak particle velocity, integrated ground displacement and single degree of freedom correlations to Cracks 2, 3, and 5.

Trackhoe Correlations Crack #	R <sup>2</sup> by Peak Particle Velocity (in/sec)	R <sup>2</sup> by Relative Displacement of Structure Method			
		Integration of Velocities G <sub>max</sub>	SDOF Method f <sub>n</sub> = 20 Hz	SDOF Method f <sub>n</sub> = 18-22 Hz	
Crack 2	(L)	0.000	0.221	0.233	0.323
	(T)	0.033	0.233	0.497	0.698
	(V)	0.827	0.698	0.532	0.590
Crack 3	(L)	0.624	0.072	0.124	0.163
	(T)	0.005	0.124	0.247	0.257
	(V)	0.287	0.257	0.126	0.096
Crack 5	(L)	0.071	0.068	0.451	0.511
	(T)	0.078	0.451	0.834	0.793
	(V)	0.663	0.793	0.575	0.552

Trencher Correlations Crack #	R <sup>2</sup> by Peak Particle Velocity (in/sec)	R <sup>2</sup> by Relative Displacement of Structure Method			
		Integration of Velocities G <sub>max</sub>	SDOF Method f <sub>n</sub> = 20 Hz	SDOF Method f <sub>n</sub> = 18-22 Hz	
Crack 2	(L)	0.947	Negative	0.942	0.000
	(T)	Negative	0.964	0.063	0.090
	(V)	0.519	0.605	0.605	Negative
Crack 3	(L)	0.947	Negative	0.942	0.000
	(T)	Negative	0.964	0.063	0.090
	(V)	0.519	0.605	0.605	Negative
Crack 5	(L)	0.990	Negative	0.703	Negative
	(T)	Negative	0.980	0.000	0.000
	(V)	0.206	0.885	0.885	Negative

Roller Correlations Crack #	R <sup>2</sup> by Peak Particle Velocity (in/sec)	R <sup>2</sup> by Relative Displacement of Structure Method			
		Integration of Velocities G <sub>max</sub>	SDOF Method f <sub>n</sub> = 20 Hz	SDOF Method f <sub>n</sub> = 18-22 Hz	
Crack 2	(L)	0.920	0.960	0.820	0.720
	(T)	0.680	0.560	0.180	0.140
	(V)	0.910	0.770	0.770	0.310
Crack 3	(L)	0.940	0.960	0.930	0.880
	(T)	0.830	0.750	0.370	0.320
	(V)	0.970	0.890	0.890	0.480
Crack 5	(L)	0.920	0.950	0.720	0.610
	(T)	0.620	0.470	0.110	0.080
	(V)	0.870	0.690	0.690	0.210

**Table 5.6 Summary of R<sup>2</sup> linear regression correlation coefficients of measured Crack 2, 3, and 5 displacement with directional peak particle velocity and computed relative displacement methods for all trackhoe, trencher and vibratory roller events**

Figure 5.18 (a), (b), (c), and (d) show the trencher producing such small ground motions that correlations are nearly impossible with regard to the traditional controls. This lack of relationship is most likely the result of very low response from all the cracks. Response was generally so low as to be indistinguishable from the electronic noise. Displacements of Crack 3 are never significantly above the 10min noise level.

Correlations of Crack 2, 3, and 5 response to longitudinal, transverse, and vertical peak particle velocities, ground displacements, and SDOF relative displacements from trackhoe activities are presented as Figure 5.19 (a), (b), (c), and (d), but are not particularly strong. With the exception of Crack 3, all the best correlations (with respect to correlation coefficient,  $R^2$  values) lie in the vertical direction. This unusual lack of correlation does not conform to results from typical blasting studies. Traditionally, crack response to blasting vibrations correlate best to ground and structural motions in the plane in which they lie. This does not necessarily imply that they respond with the largest crack displacements to these motions, but that there is typically a noticeable correlation between the two. This lack of correlation may also be the result of the low response, as was the case discussed above with the trencher.

Correlations of crack response to peak particle velocities, ground displacements and single degree of freedom calculations produced by trench backfill and roadway sub-grade vibratory roller compaction are presented in Figure 5.20 (a), (b), (c), and (d). Crack response was large enough to produce a wide range of response to variable excitation and thus meaningful correlations. All cracks responded best to peak particle velocities and displacements in the longitudinal direction, Crack 2 responded best to SDOF response in the transverse direction, and Cracks 3 and 5 best SDOF in the long direction. Vibratory compaction produced the largest overall and widest range of peak particle velocities, plus the most harmonious frequencies. The attenuation study presented in Chapter 4 reveals that peak particle velocities between 0.1 and 0.2 are expected when the small roller is operated at its 32 HZ low frequency at a distance of 40 feet.

Several of the time histories from the large roller compaction represent a “worst-case” scenario. For four of the events, the roller passed within 10 feet of the house, an atypically small separation between the roller and the house. These events made it

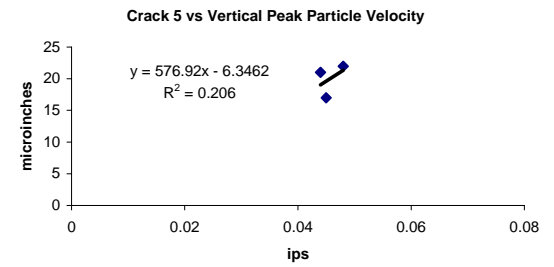
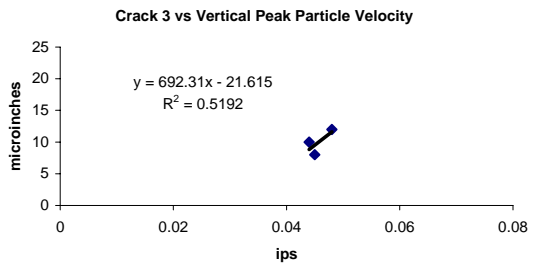
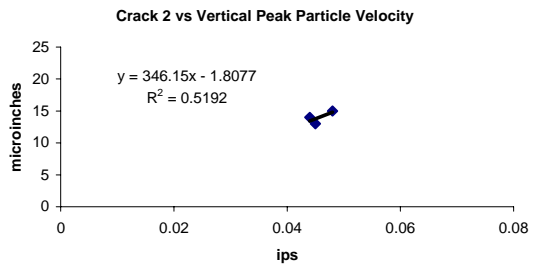
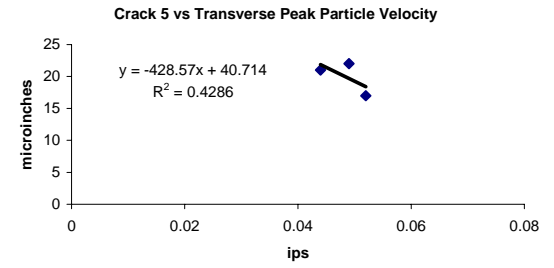
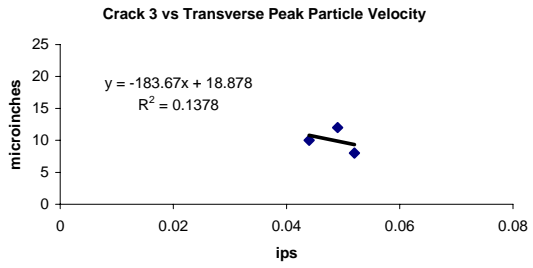
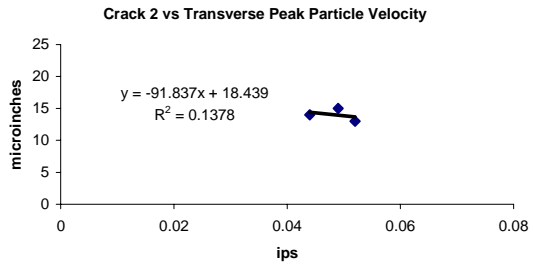
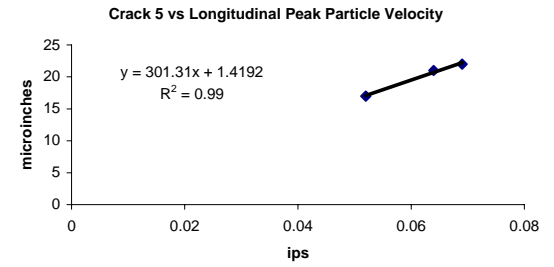
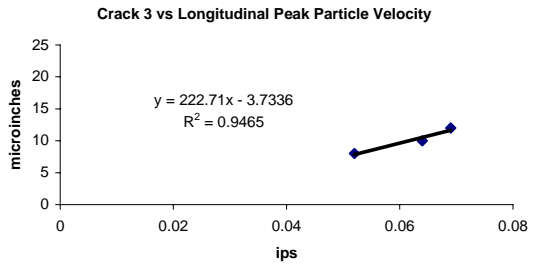
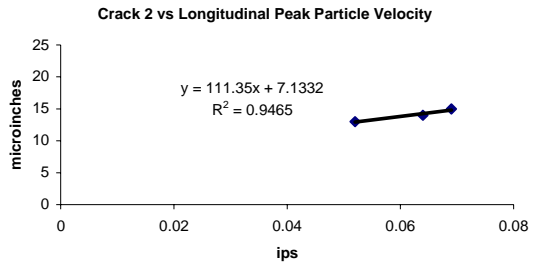
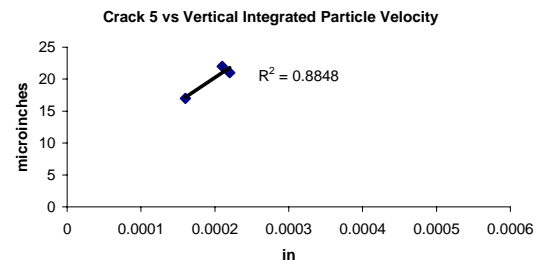
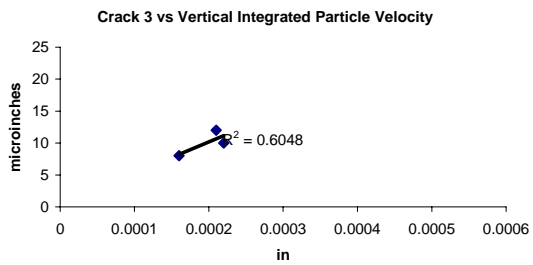
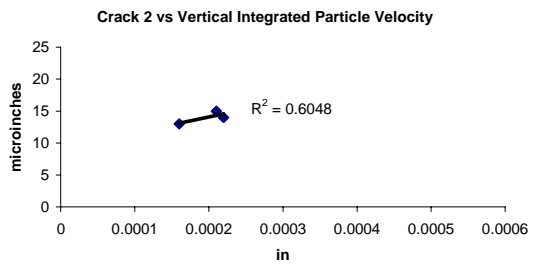
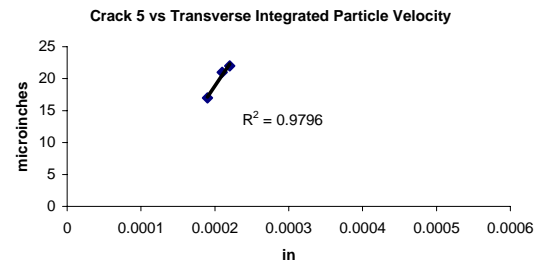
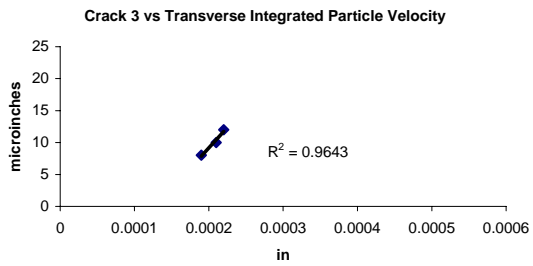
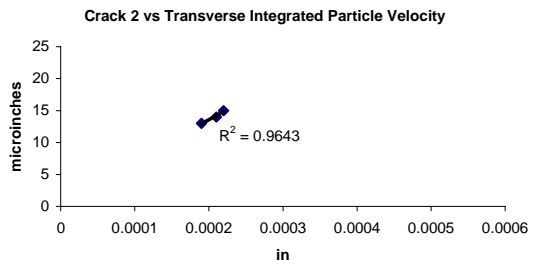
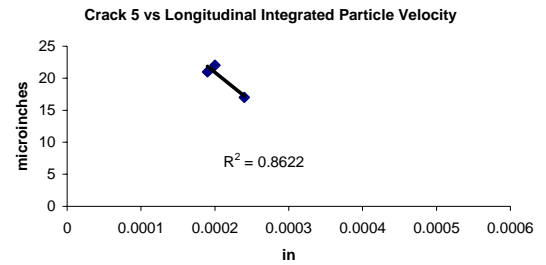
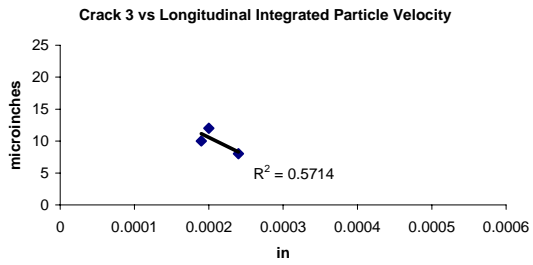
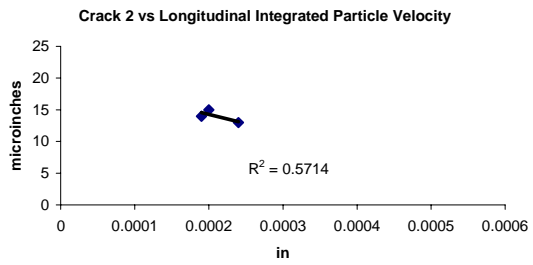


Figure 5.18 (a) Microinch crack 2, 3, and 5 displacement versus directional peak particle velocity correlations for trencher excavation events





**Figure 5.18 (b) Microinch crack 2, 3, and 5 displacement versus directional integrated particle velocity correlations for trencher excavation events**

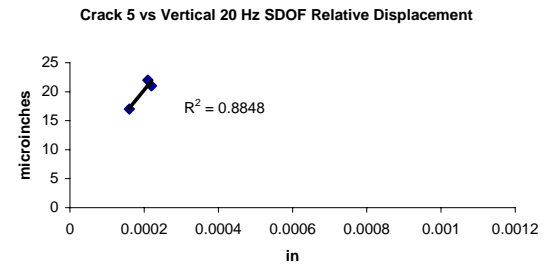
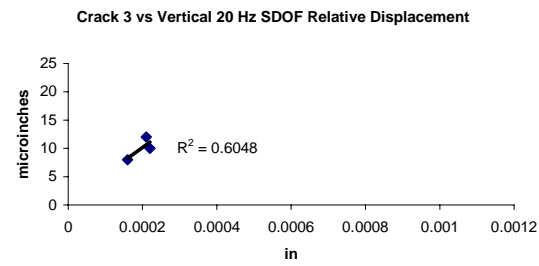
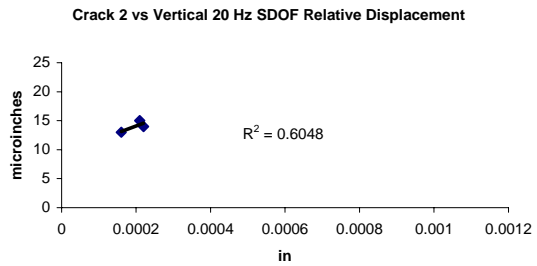
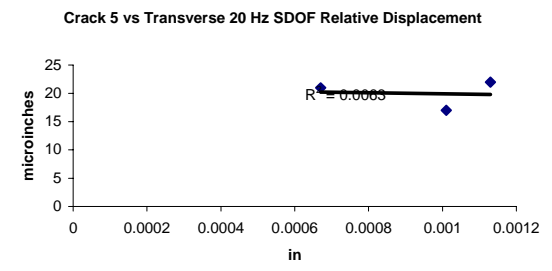
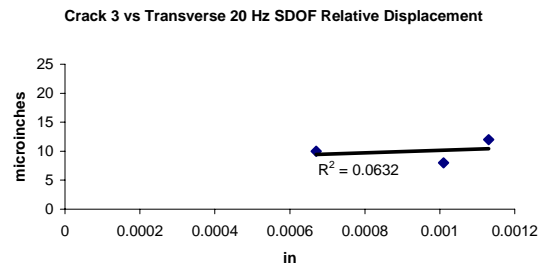
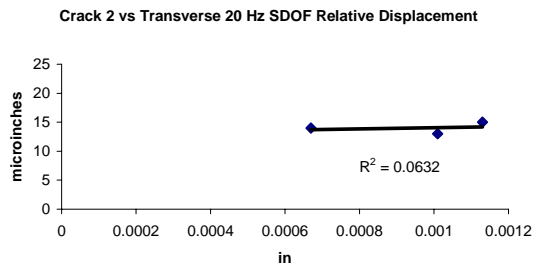
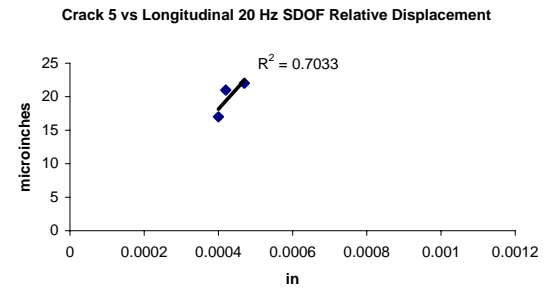
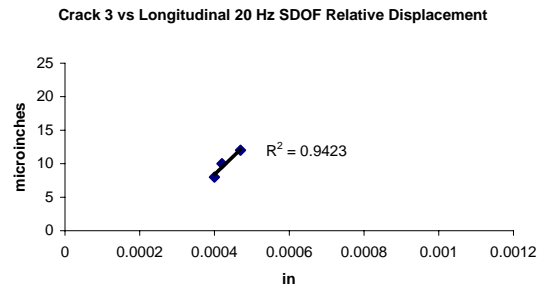
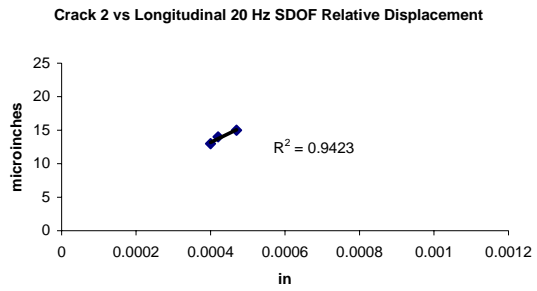


Figure 5.18 (c) Microinch crack 2, 3, and 5 displacement versus directional 20 Hz SDOF relative displacement correlations for trencher excavation events

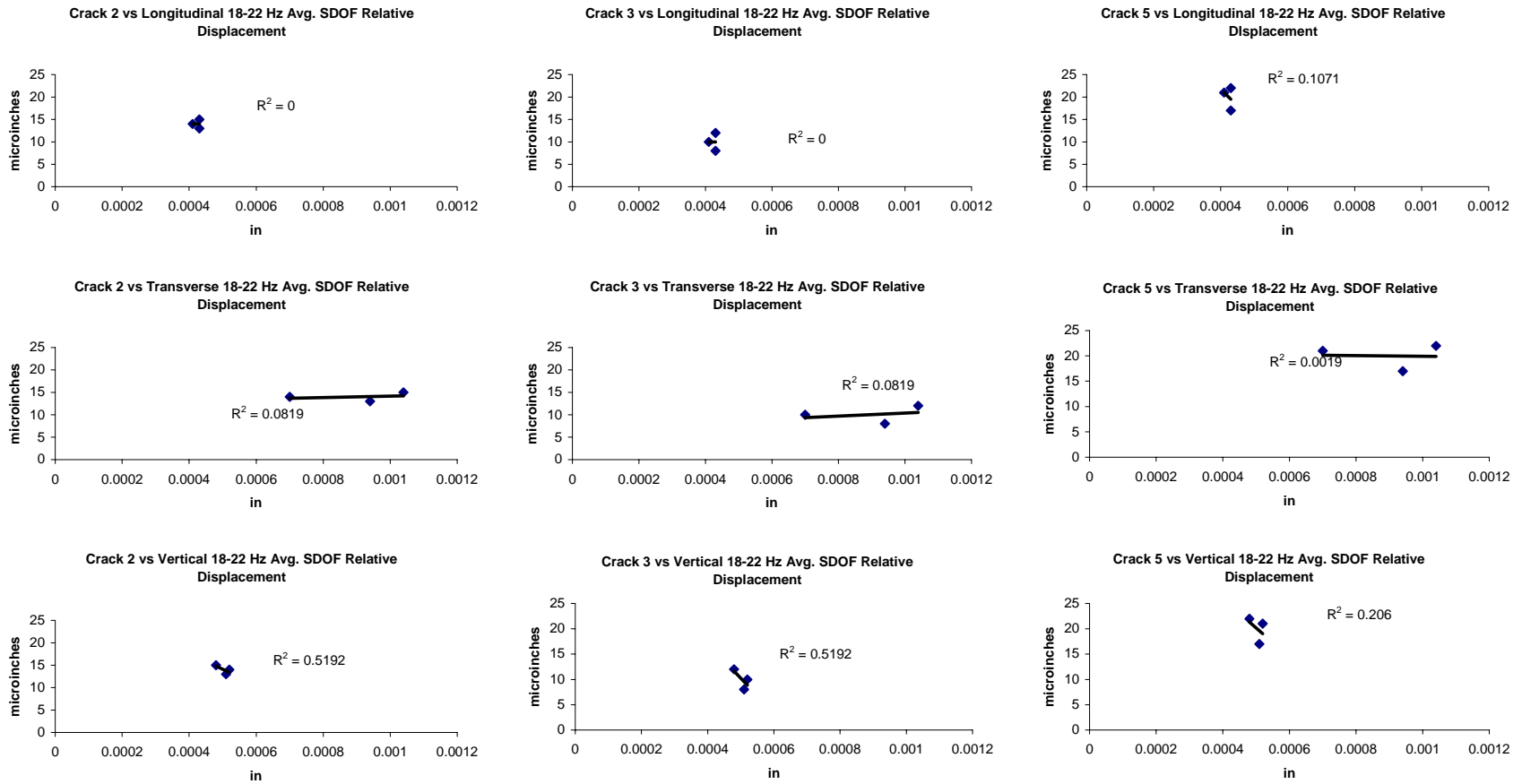


Figure 5.18 (d) Microinch crack 2, 3, and 5 displacement versus directional 18-22 Hz average SDOF relative displacement correlations for trencher excavation events

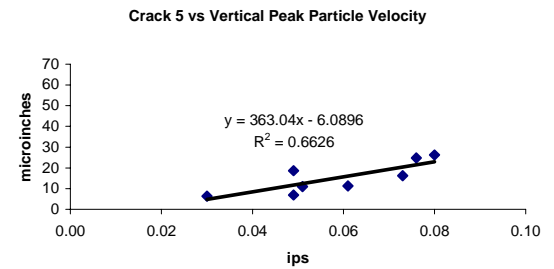
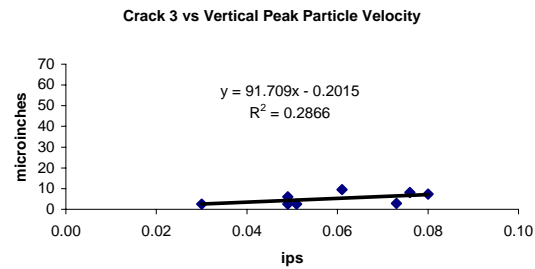
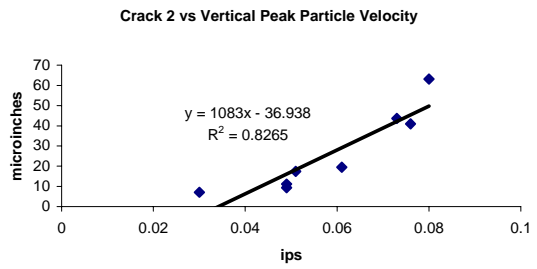
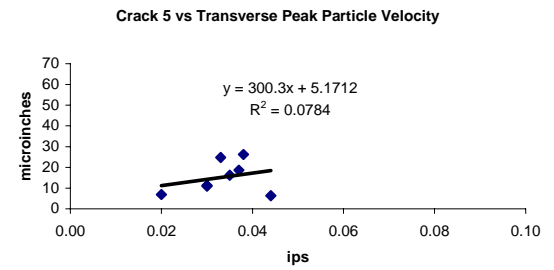
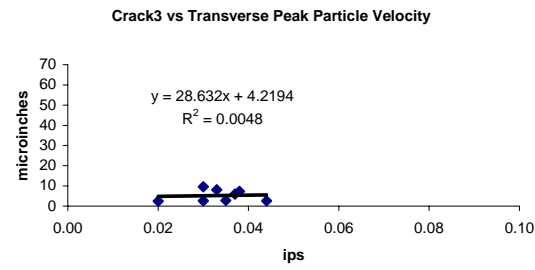
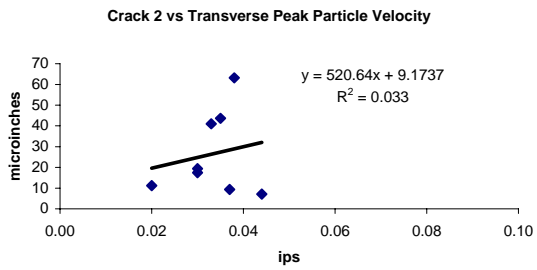
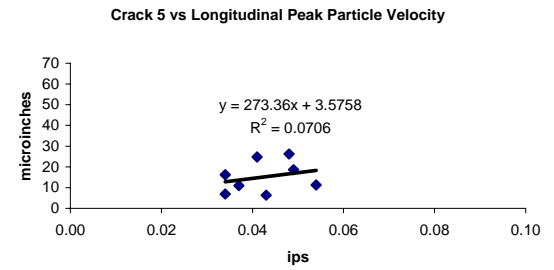
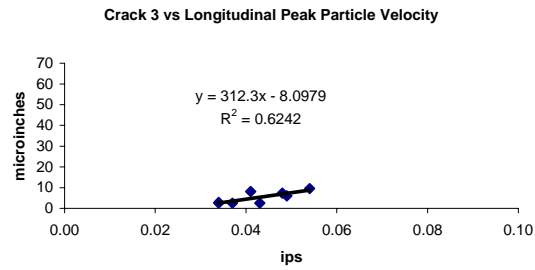
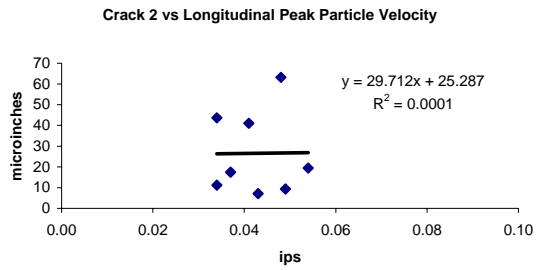


Figure 5.19 (a) Microinch crack 2, 3, and 5 displacement versus directional peak particle velocity correlations for trackhoe excavation events

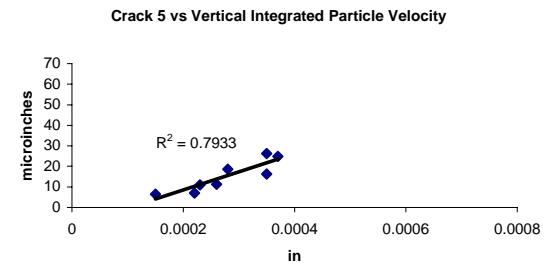
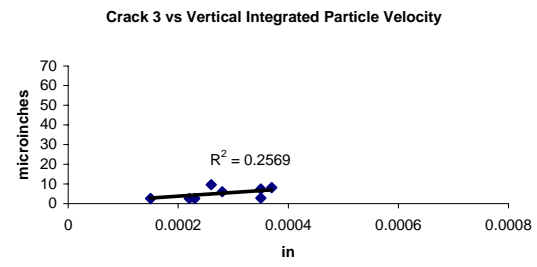
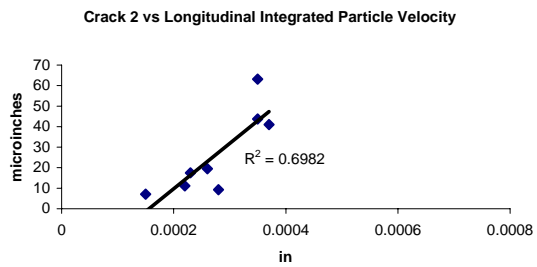
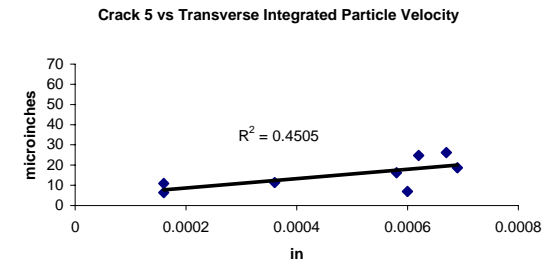
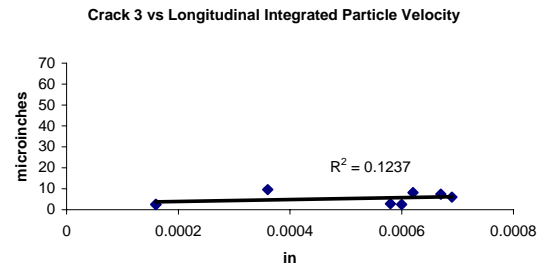
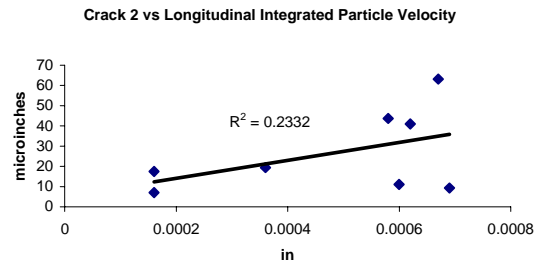
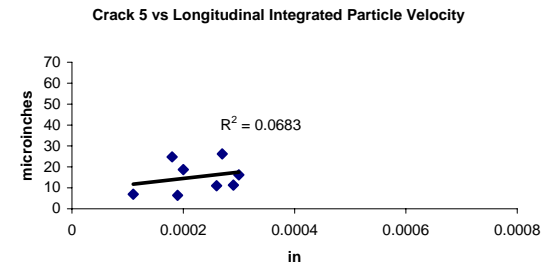
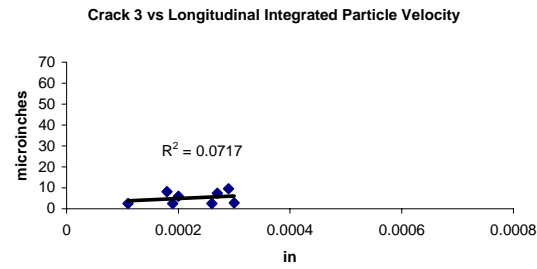
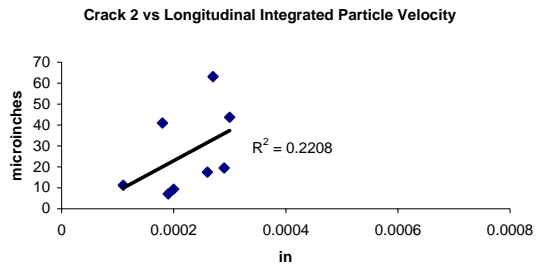


Figure 5.19 (b) Microinch crack 2, 3, and 5 displacement versus directional integrated particle velocity correlations for trackhoe excavation events

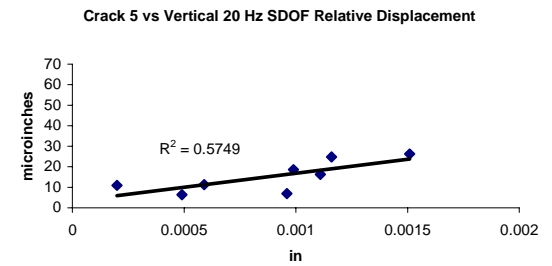
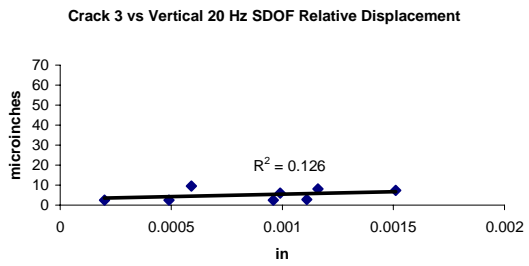
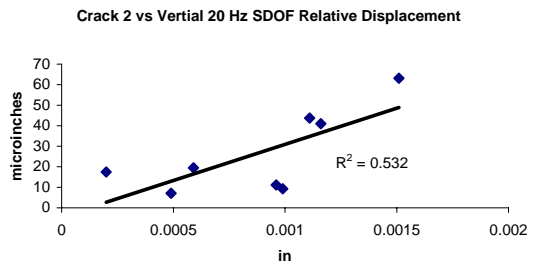
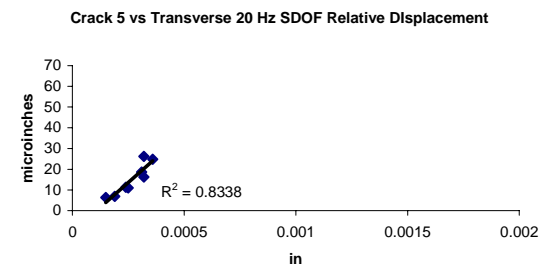
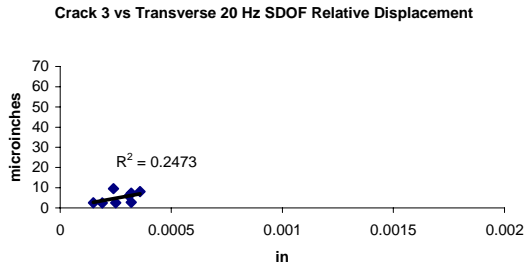
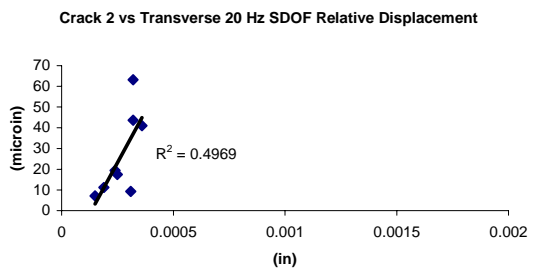
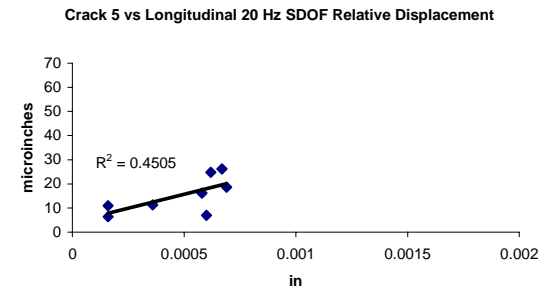
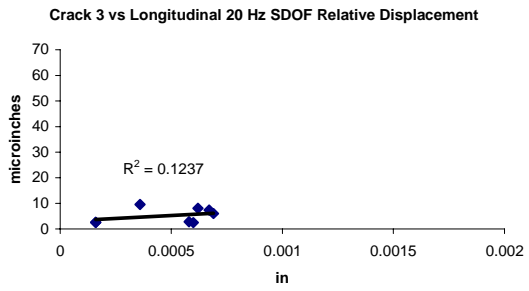
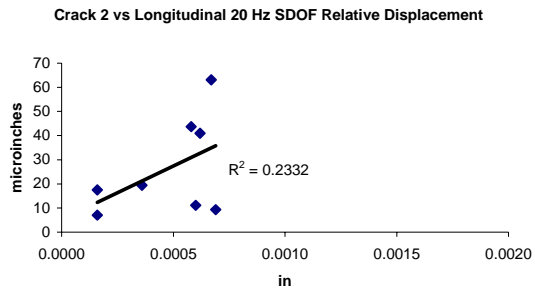


Figure 5.19 (c) Microinch crack 2, 3, and 5 displacement versus directional 20 Hz SDOF relative displacement correlations for trackhoe excavation events

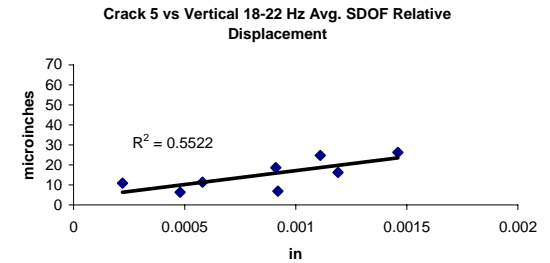
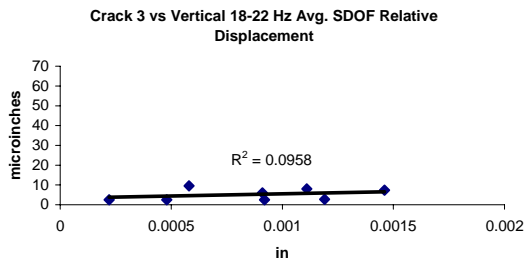
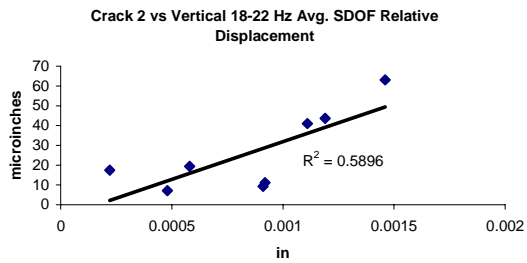
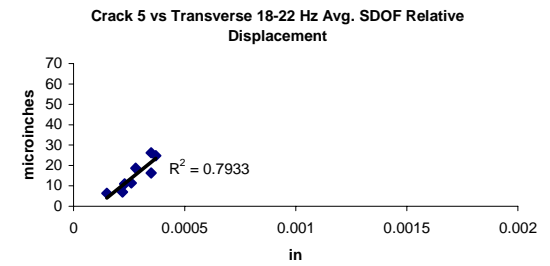
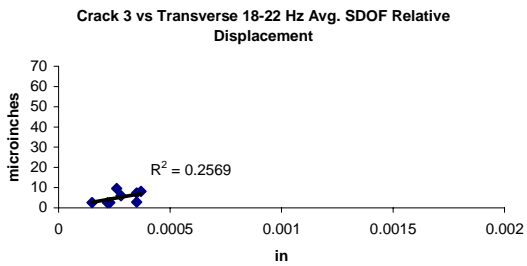
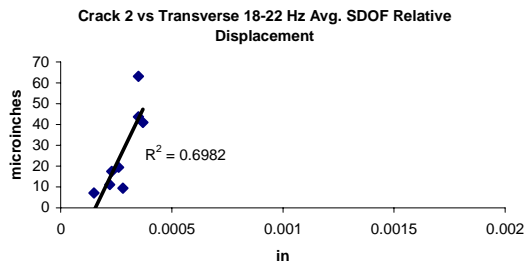
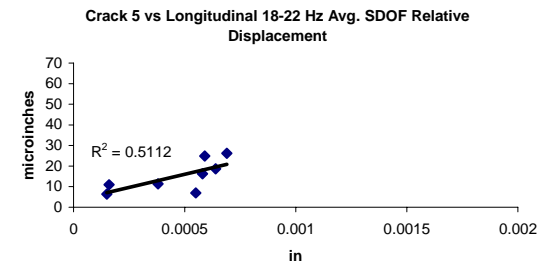
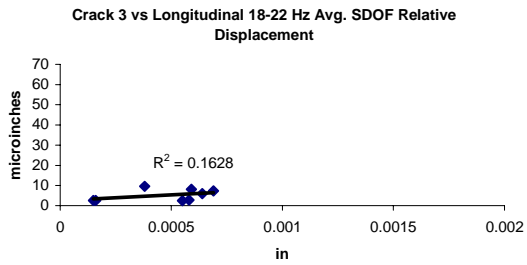
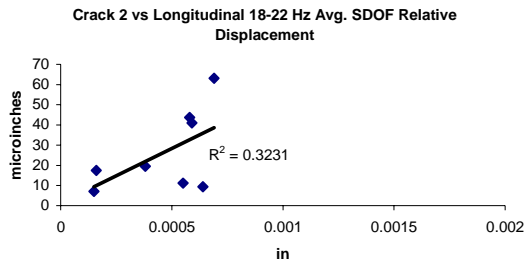


Figure 5.19 (d) Microinch crack 2, 3, and 5 displacement versus directional 18-22 Hz average SDOF relative displacement correlations for trackhoe excavation events

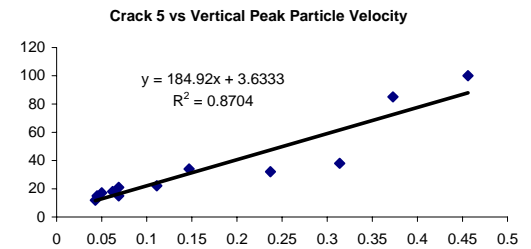
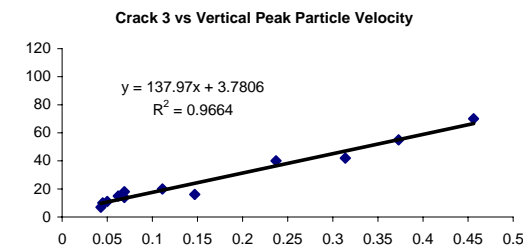
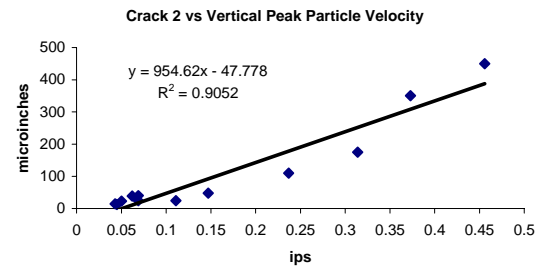
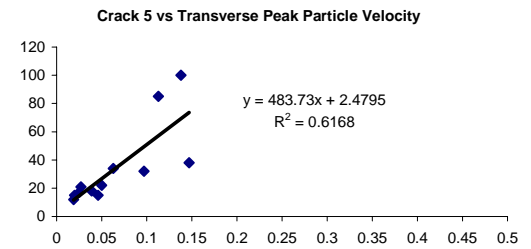
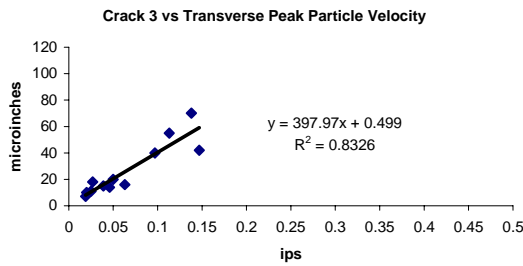
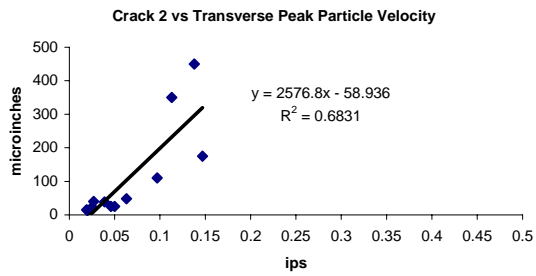
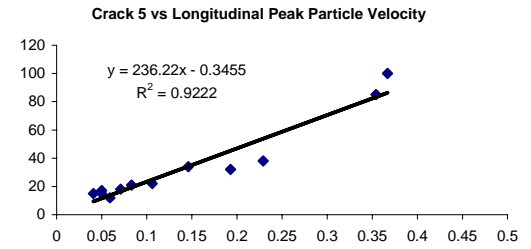
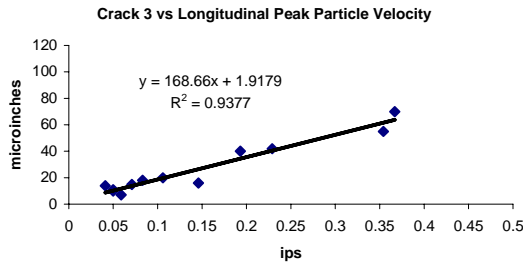
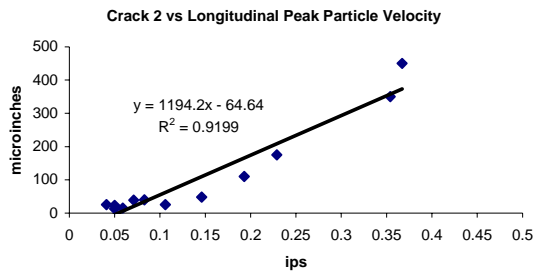


Figure 5.20 (a) Microinch crack 2, 3, and 5 displacement versus directional peak particle velocity correlations for vibratory roller compaction events



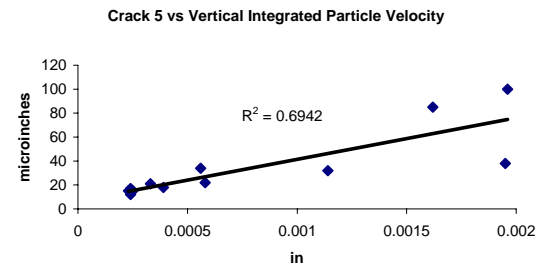
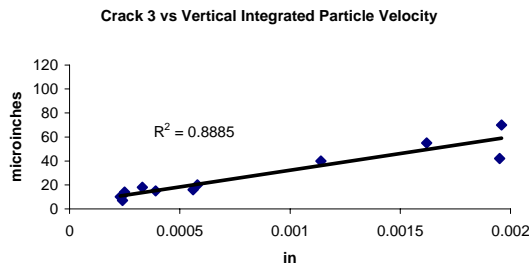
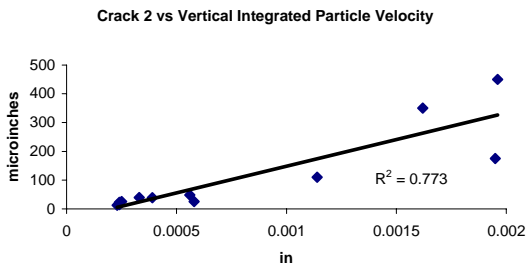
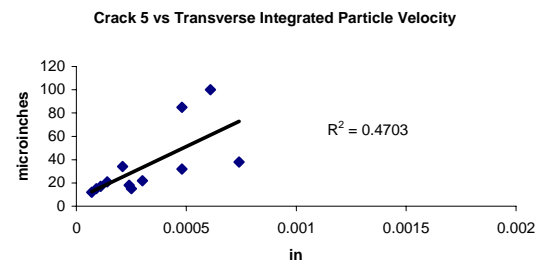
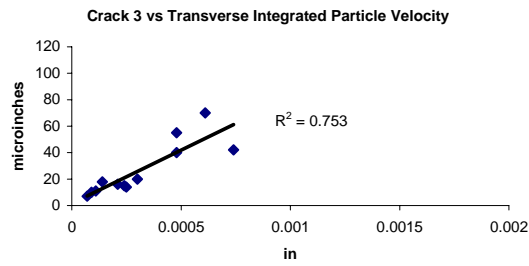
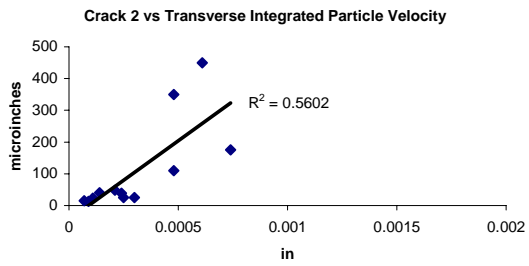
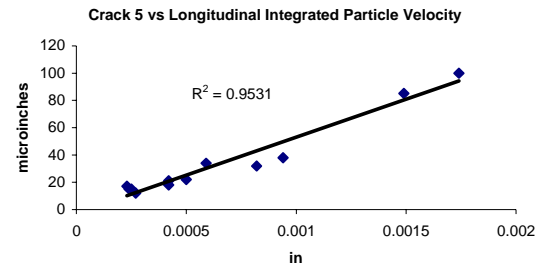
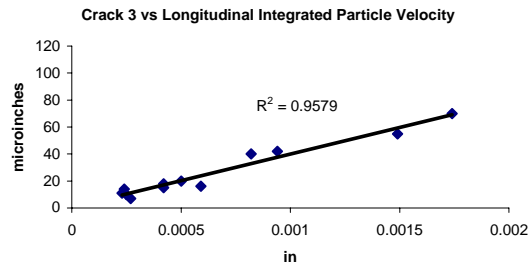
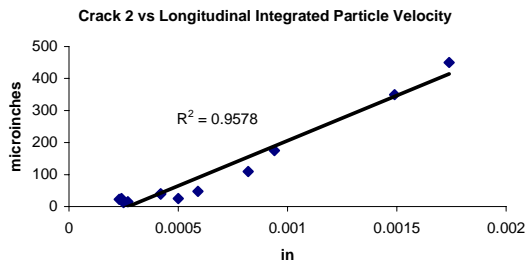


Figure 5.20 (b) Microinch crack 2, 3, and 5 displacement versus directional integrated particle velocity correlations for vibratory roller compaction events

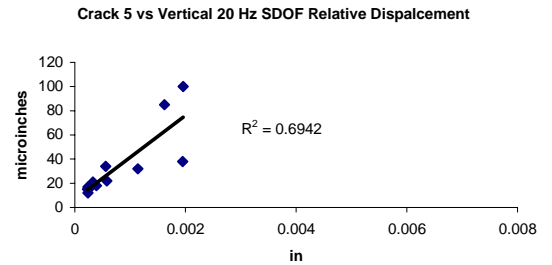
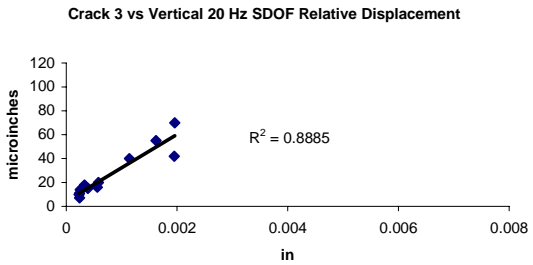
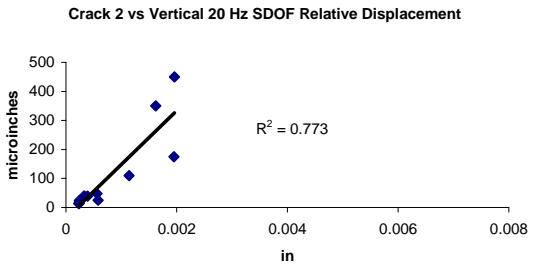
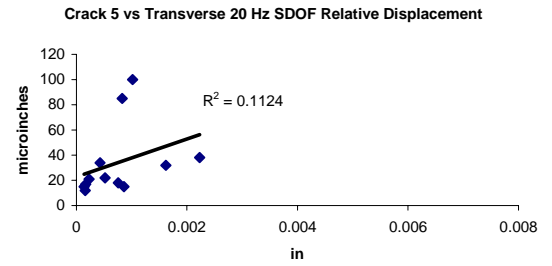
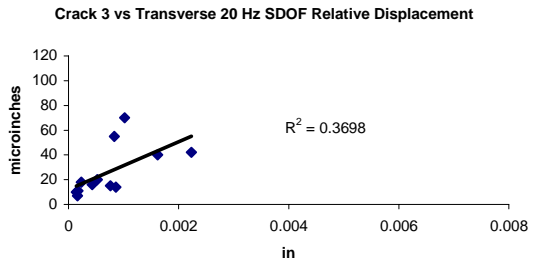
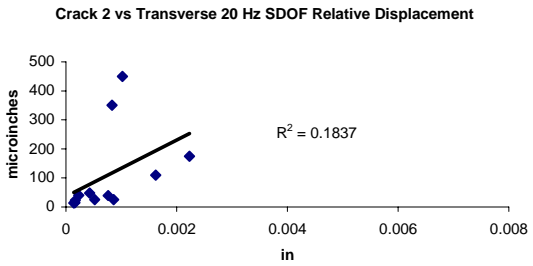
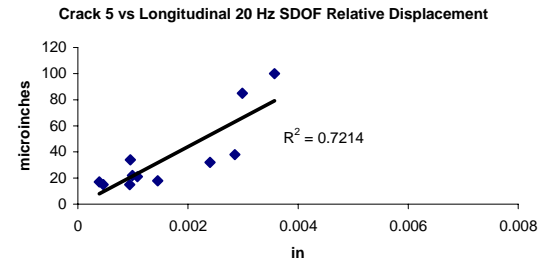
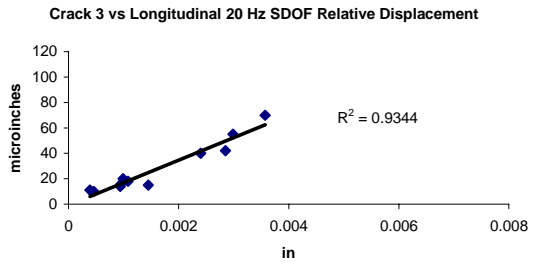
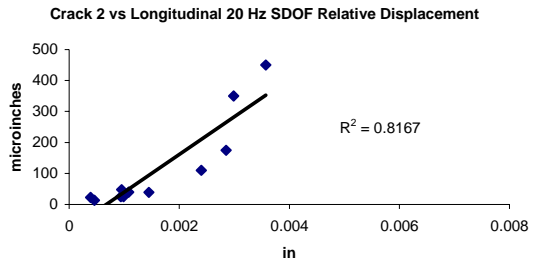


Figure 5.20 (c) Microinch crack 2, 3, and 5 displacement versus directional 20 Hz SDOF relative displacement correlations for vibratory roller compaction events

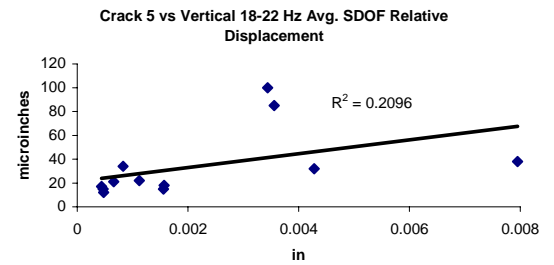
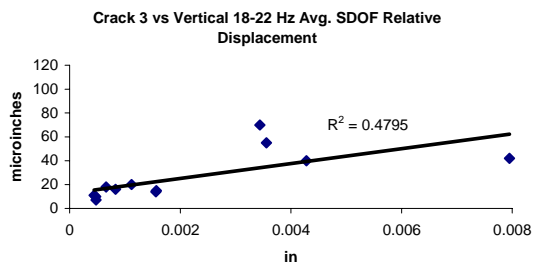
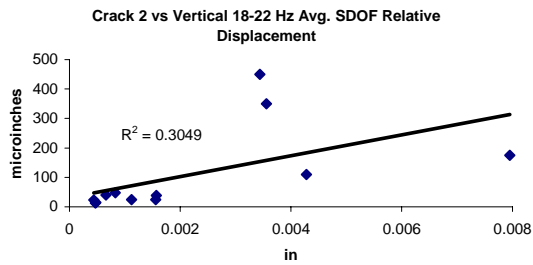
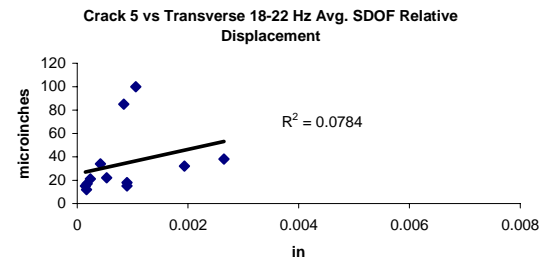
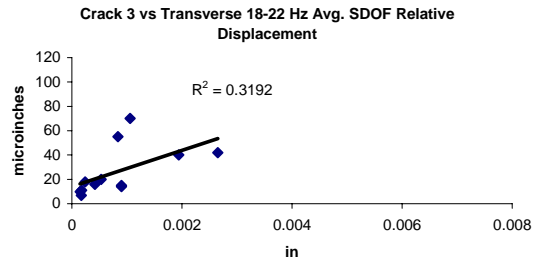
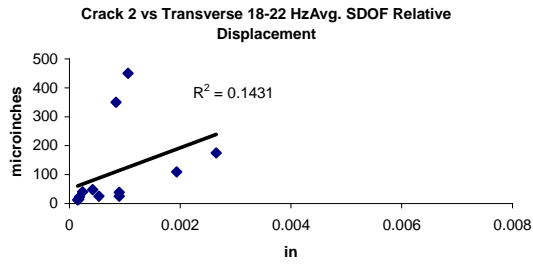
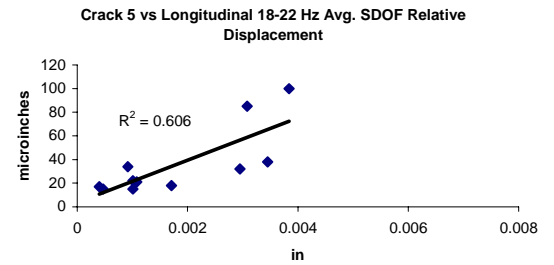
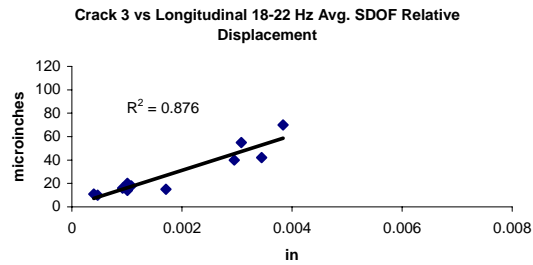
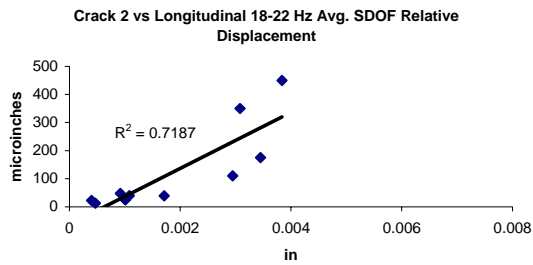


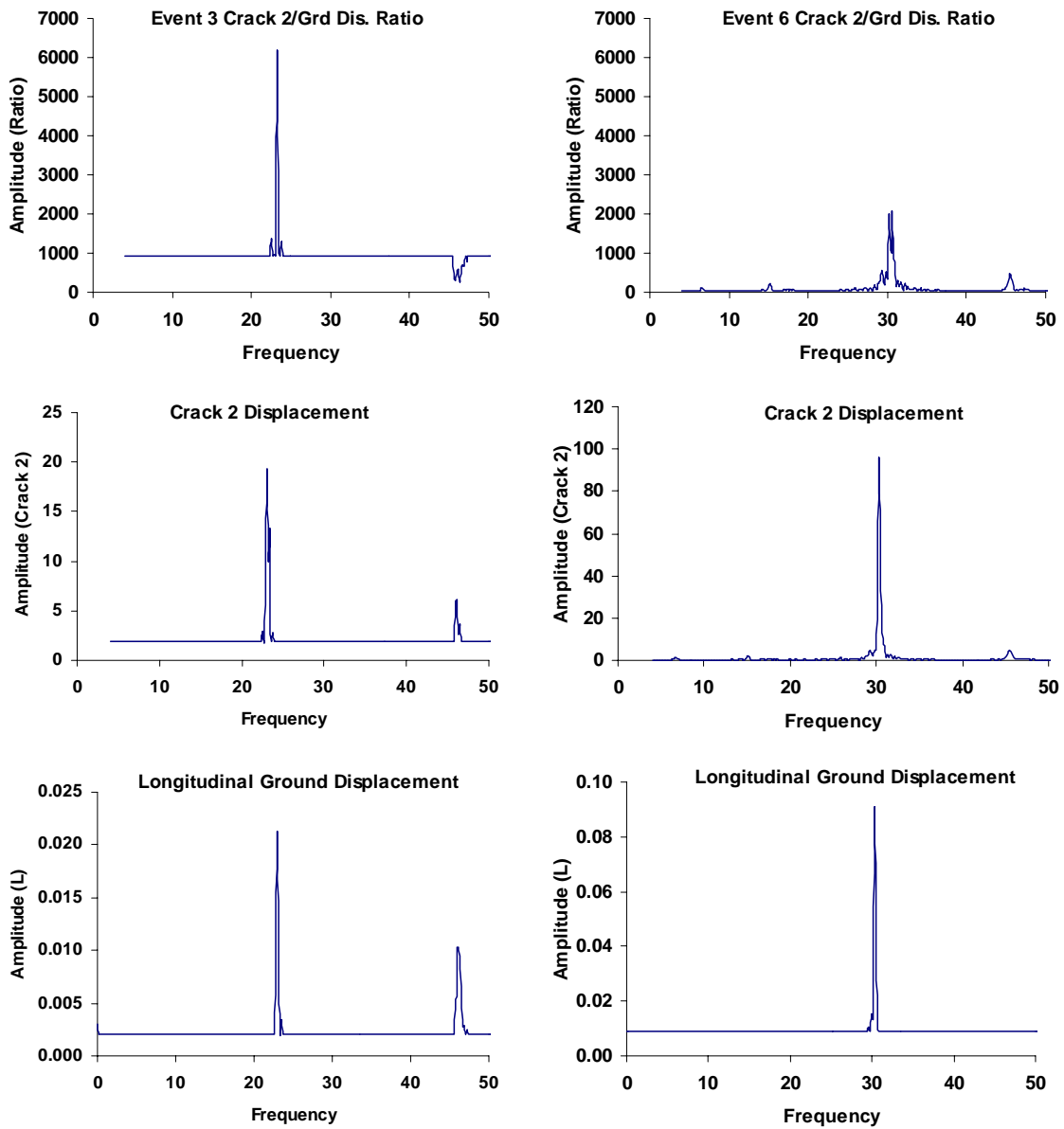
Figure 5.20 (d) Microinch crack 2, 3, and 5 displacement versus directional 18-22 Hz average SDOF relative displacement correlations for vibratory roller compaction events

possible to examine and correlate a broad and complete range of peak particle velocities, much like traditional blasting studies. These correlations over a wide range of excitation provide the best insight into the construction vibration sensitivity of the cracks.

An important issue presented by vibratory compaction is the effect of excitation frequency on crack displacement, an issue discussed in Chapter 3 for blasting. 15-25 Hz is a common natural frequency for a structure wall, but only the large roller produced frequencies within this normal range. Events 1 through 4 were conducted at a frequency of approximately 23 Hz, while events 5 through 8 were at 32Hz. Regardless of the frequency associated with these eight roller events, Crack 2 response correlates best with peak particle velocity, seen in Figure 5.20 (a), regardless of excitation frequency.

Figure 5.21 shows longitudinal ground displacement, Crack 2 displacement, and Crack 2 divided by longitudinal ground displacement FFT spectra for large roller events 3 and 6, both recorded at the same distance ( $\approx 8$ ft). Despite the fact the wall should have only one response frequency, it responds most at the active vibration frequency whether it is 23 or 32 Hz. It is possible that there are two modes of response, one near 20 Hz and the other near 30 Hz. This possibility is supported by review of the FFT response associated with trackhoe event 2 in Figure 5.10, which shows amplified wall response at both 20 and 30 Hz.

To be discussed in Chapter 6, the local nature of close proximity construction vibrations may promote the component excitation and response over whole structure excitation and response. This local nature of excitation may contribute to the observation of possible dual roller response modes, such as those described above. Figure 5.22 (a) compares the single degree of freedom response spectra for large roller events 2 and 7 (low amplitude excitation), while Figure 5.22 (b) compares events 3 and 6 (high amplitude excitation).



**Figure 5.21** Frequency Fourier Transform histogram of Crack 2 displacement divided by longitudinal ground displacement ratio (top), measured Crack 2 displacement (middle), and calculated longitudinal ground displacement (bottom) for large vibratory roller events #3 (left) and #6 (right) on 8 November 2002

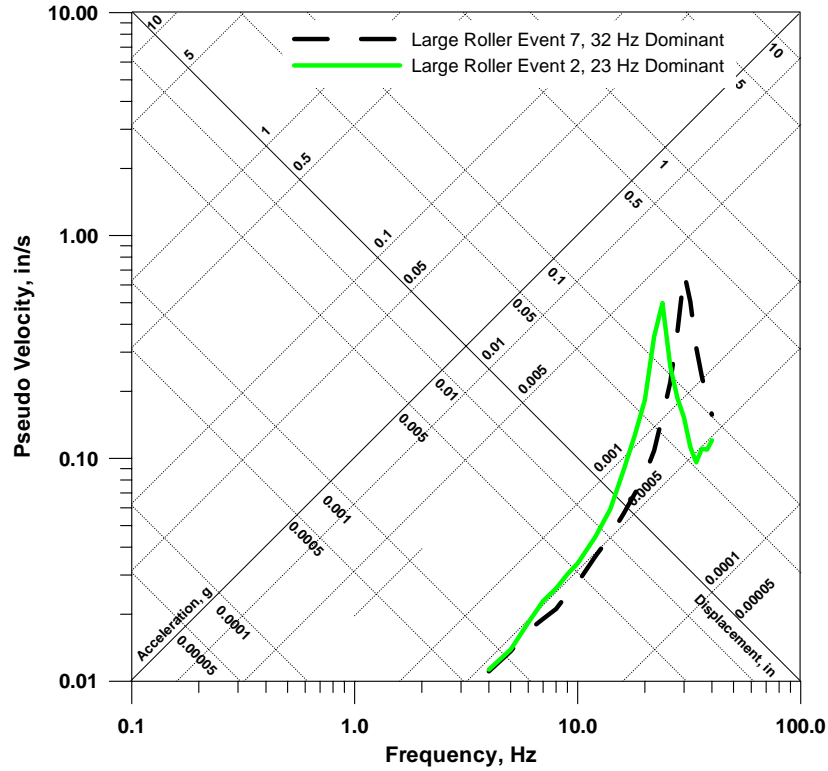


Figure 5.22 (a) Longitudinal single degree of freedom response spectra for large roller events #2 and #7 (low amplitude)

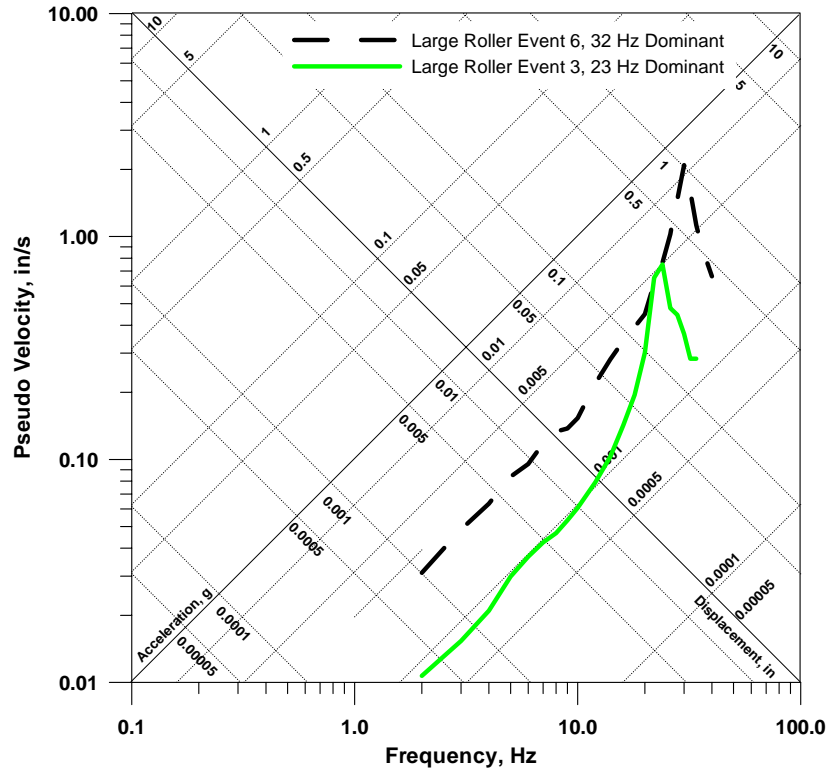


Figure 5.22 (b) Longitudinal single degree of freedom response spectra for large roller events #3 and #6 (high amplitude)

## *Chapter 6*

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### *Comparison of Crack Displacements Resulting from On-Site Blasting Vibrations and On-Site Construction Vibrations*

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#### *Introduction*

This chapter compares the effects of blasting in the Connecticut aggregate quarry in Chapters 2 and 3 with the construction activities in North Las Vegas, Nevada from Chapters 4 and 5. Comparisons are made with descriptors that are employed across disciplinary boundaries to define the effects of excitation ground motions near structures and homes. These descriptors are based upon the concepts of geo and structural dynamics as applied to situations involving blasting, construction or earthquake induced ground motions in order to predict structural response.

The descriptors of ground excitation waveforms that have been shown to have a significant impact on overlying structures are amplitude, number of principal pulses, frequency (or period) and wavelength. Non-waveform specific, but equally important factors, are absolute distance from the energy source and dynamic properties of the structure itself, such as damping and natural frequency (Dowding, 1996). Each of these properties and factors is associated with a particular analysis tool which will be employed for the purposes of comparison. The effect of time histories is compared by examining the peak amplitude (particle velocity), the number of “significant peaks”, and the length of the “significant waveform” within the time history. Frequency effects are shown

through calculation of single degree of freedom (SDOF) response spectra, and wavelengths are calculated from peak waveform frequencies and a general knowledge of the surrounding soil/rock medium. Plane-wave theory is employed to show non-uniform arrival at structures and explore the effects of absolute distance. Finally, structural “sensitivity” will be established by determining a relationship between crack displacements and weather-induced displacements.

### **Time Histories**

Five different construction activities will be evaluated: quarry blasting, trackhoe and trencher excavation, and two types of vibratory compaction. The differences between excitation vibrations caused by blasting and construction, as well as those between the construction equipment itself are all significant. The time histories chosen for analysis are those that produced substantial, if not the highest, ground motions. Time histories (introduced in previous chapters) to be discussed are:

- Blast Event 15, PPV=0.345 ips
- Trackhoe Event #2, PPV=0.08 ips
- Trencher Event #2, PPV=0.069 ips
- Small Roller Event #3, PPV=0.147 ips
- Large Roller Event #3, PPV=0.237 ips

Table 6.1 summarizes all the information pertinent to these events and the analysis to follow. Figure 6.1 compares time histories of crack displacement for Connecticut Cracks 1 and 3 and Crack 2 in Las Vegas resulting from blasting and the four construction induced vibrations respectively. Figures 6.2, 6.3, and 6.4 compare the longitudinal, transverse, and vertical components of ground motion. Crack 2 response was chosen for comparison, as it is consistently the most responsive of all the Las Vegas cracks.

Principal characteristics of the construction time histories shown in Figures 6.2-4 are summarized in Table 6.1. These are peak particle velocity, number of significant peaks, length of significant excitation, dominant frequency and wavelength. “Significant peak” is defined as any zero-to-peak value in a velocity time history that exceeds 75% of the peak particle velocity. “Significant excitation” is defined as the time during which velocity peaks exceed 50% of the control, or peak particle velocity (Dowding, 1996).

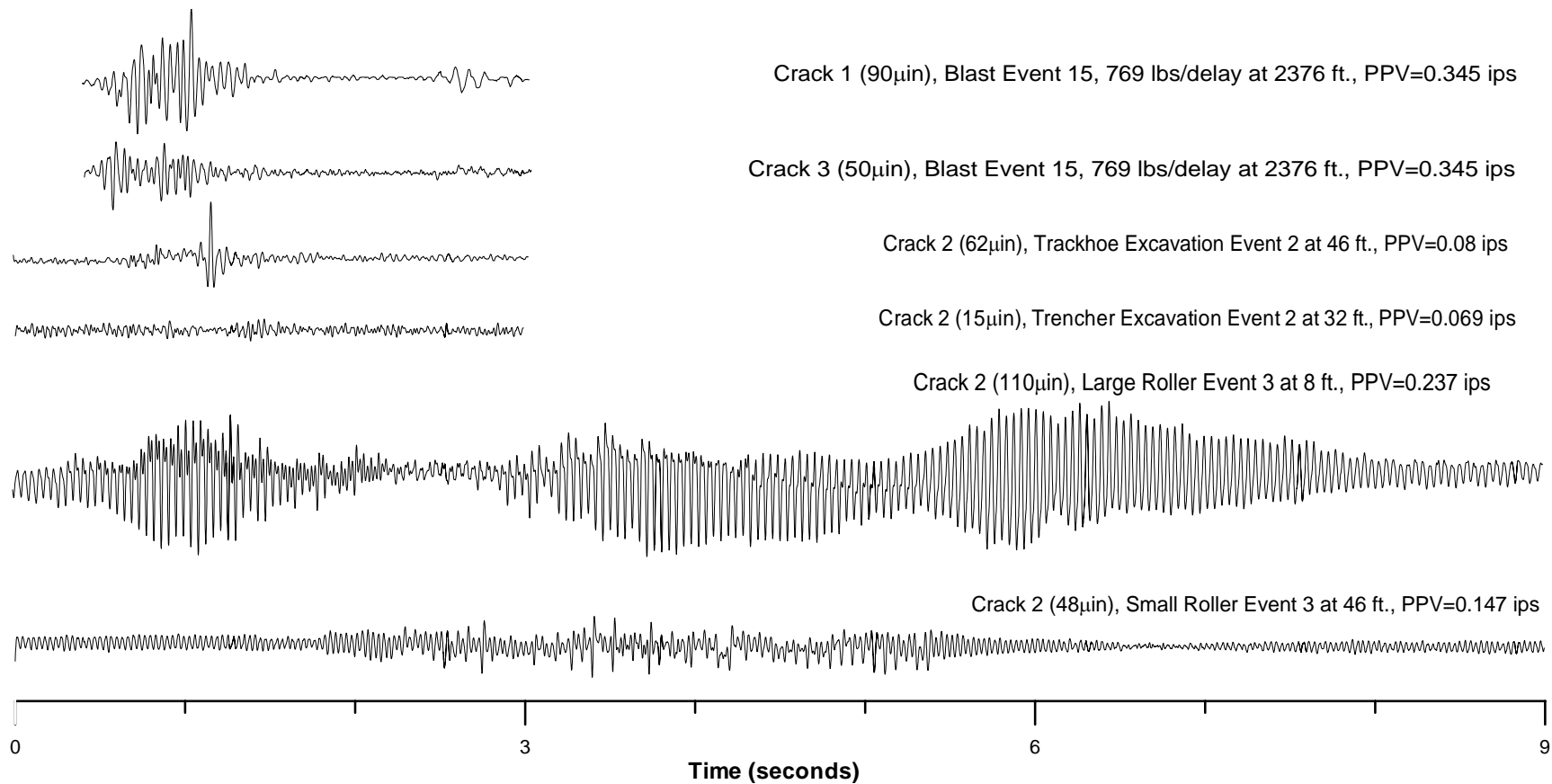


Furthermore, pulses and waveforms are only quantified by the positive or negative, zero-to-peak side of the time history that contains the overall peak. For instance, blast 15 had 7 significant pulses and a 0.62 second excitation. Trackhoe event 2 had 2 significant pulses with a significant excitation of 0.14 seconds. Trencher event 2 produced only 1 significant peak, with a very short 0.08 second excitation. The two roller events caused the most sustained vibration. Small roller event 3 contained 10 significant pulses with a 2.7 second excitation, and large roller event 3 contained 35 significant pulses over 4.3 seconds of excitation.

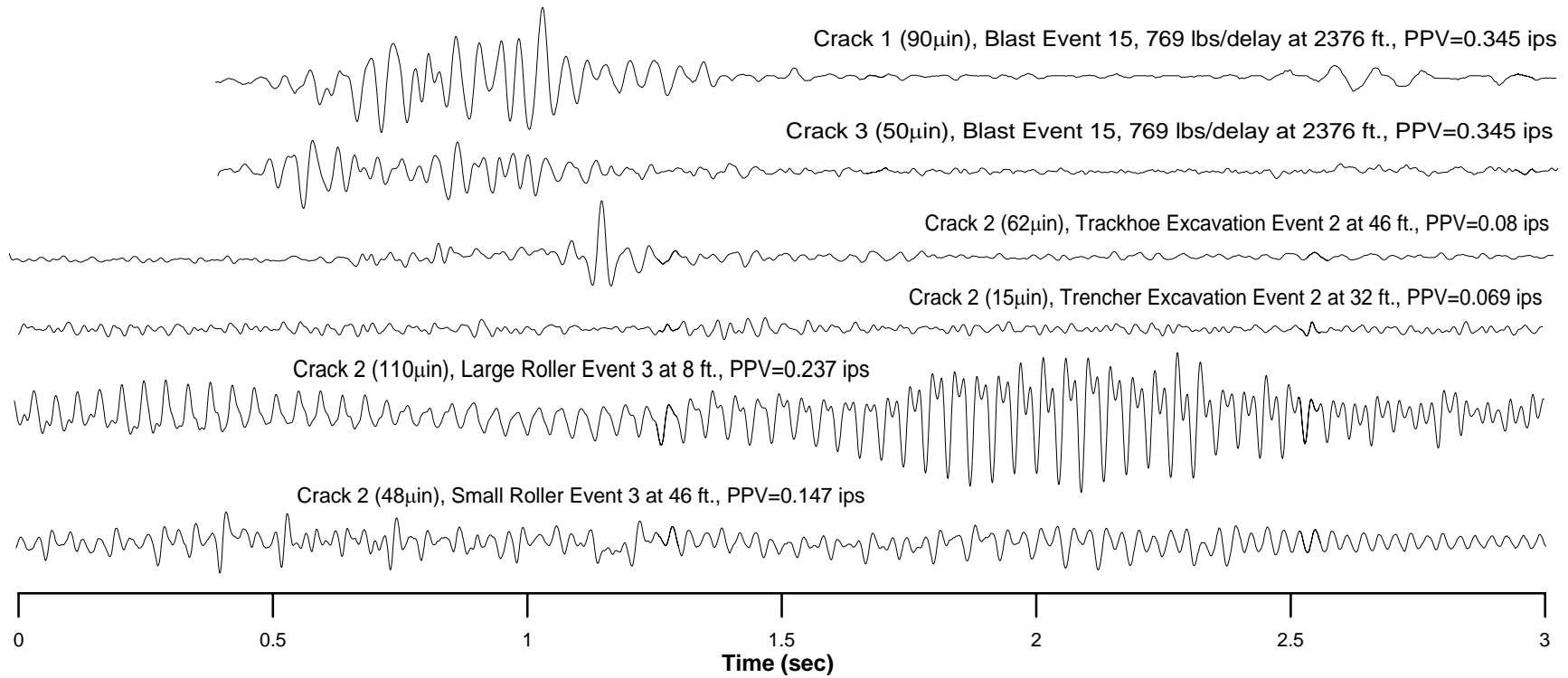
While the parameters in Table 6.1 provide a means of quantitatively describing what is visually evident in Figures 6.2-6.4, they ultimately cannot be quantitatively combined to easily predict differences in response. The response spectrum can integrate all of these parameters because it employs the full waveform of the excitation. Thus the next two sections focus on a method to normalize the excitation, employment of the single degree of freedom response spectrum, and calculations and visualizations of wavelength to help demonstrate the differences in the structural response produced by these different forms of excitation.

Event	Peak Particle Velocity (ips)	75% Peak (ips)	# of Significant Peaks	50% Peak (ips)	Length of Significant Excitation (sec)	Peak SDOF Frequency (Hz)	Dominant FFT Frequency (Hz)	Wavelength (ft)	Crack Displacement '(#, Location)' (μin)
Blast 15	0.345	0.259	7	0.173	0.65	20	23	435	90 (3, Conn)
Trackhoe #2	0.080	0.060	2	0.040	0.14	25	22	182	63 (2, LV)
Trencher #2	0.069	0.052	1	0.035	0.08	70	54	74	15 (2, LV)
Small Roller #3	0.147	0.110	10	0.074	2.7	30	32	125	48 (2, LV)
Large Roller #3	0.237	0.178	23	0.119	4.50	24	24	167	110 (2, LV)

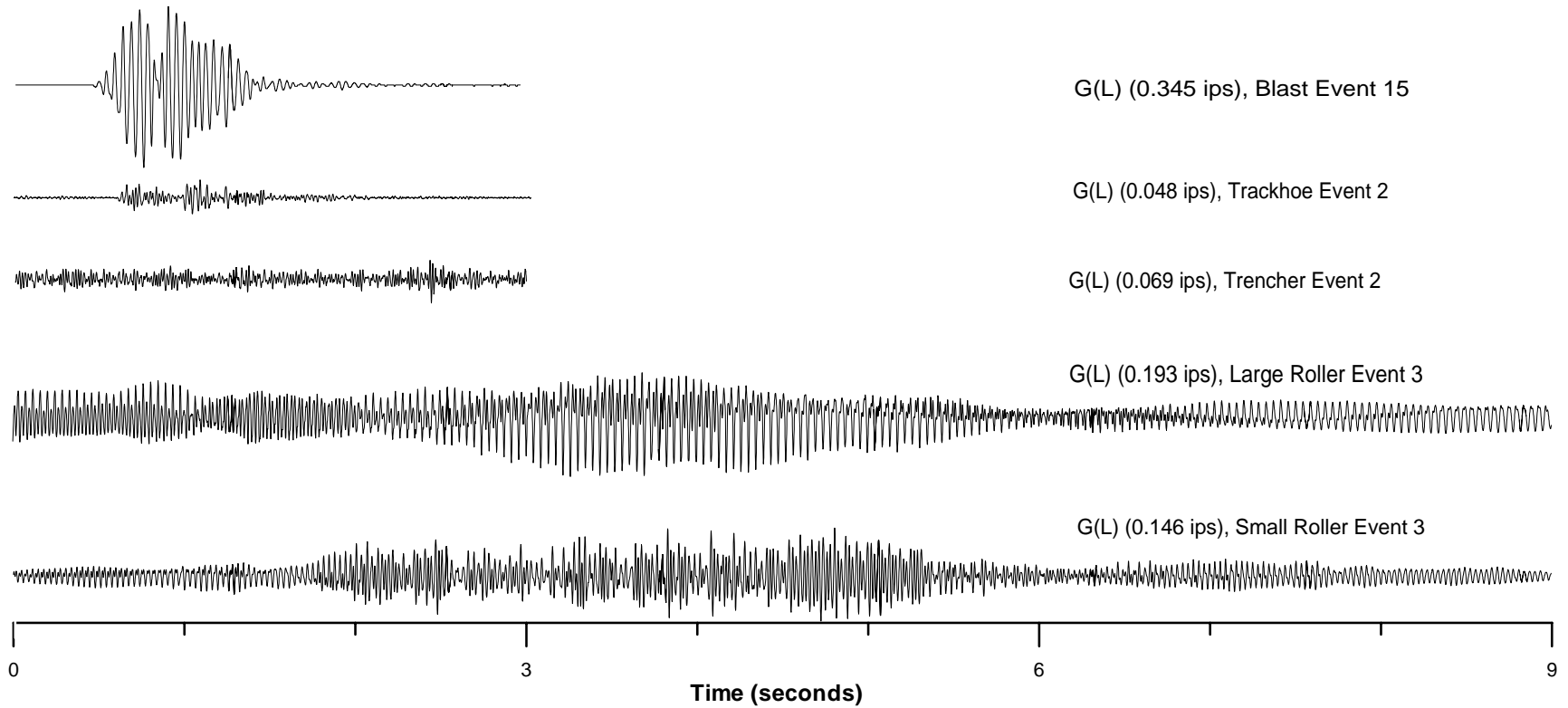
**Table 6.1 Summary of information pertaining to time histories and waveforms, frequency response, and crack displacements**



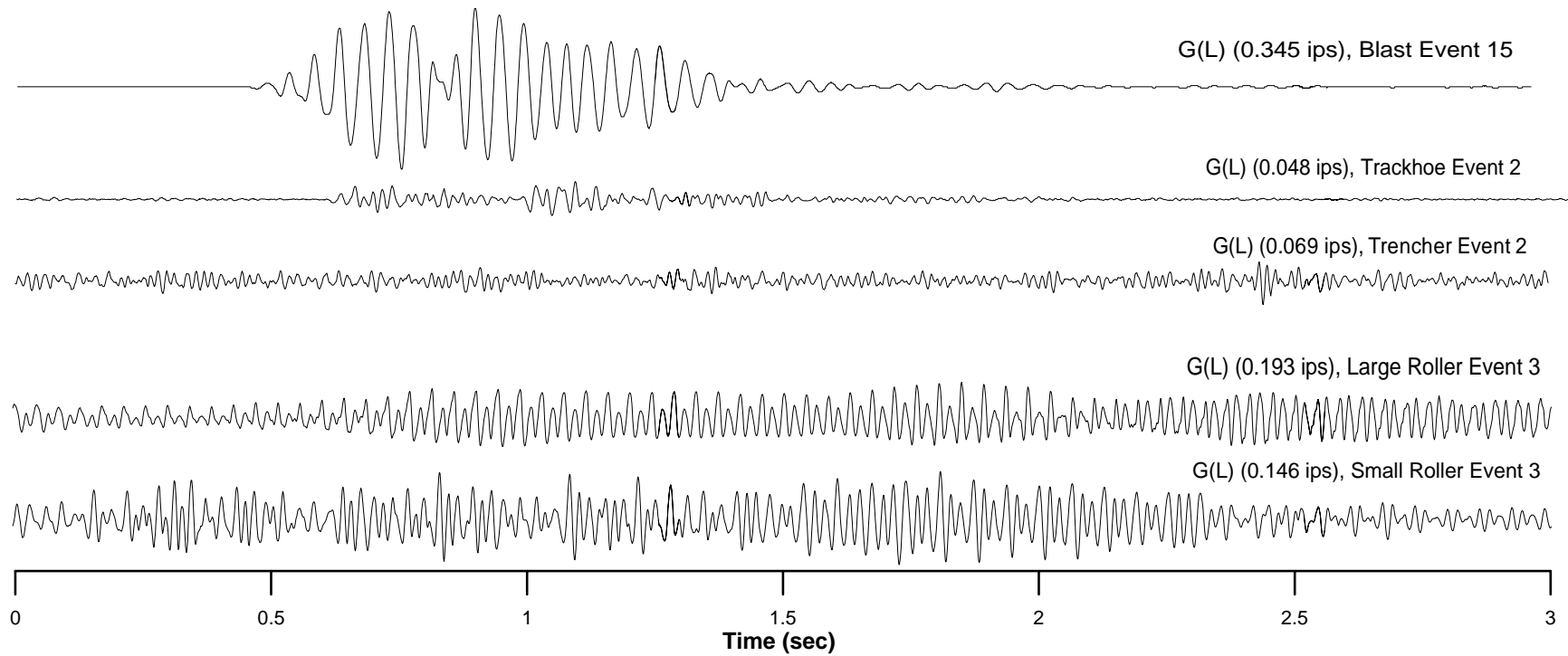
**Figure 6.1 (a) Global time histories of Connecticut Crack 1 and 3 displacements from blast event 15 and Las Vegas Crack 2 displacement from trackhoe event 2, trencher event 2, small vibratory roller event 3 and large vibratory roller event 3**



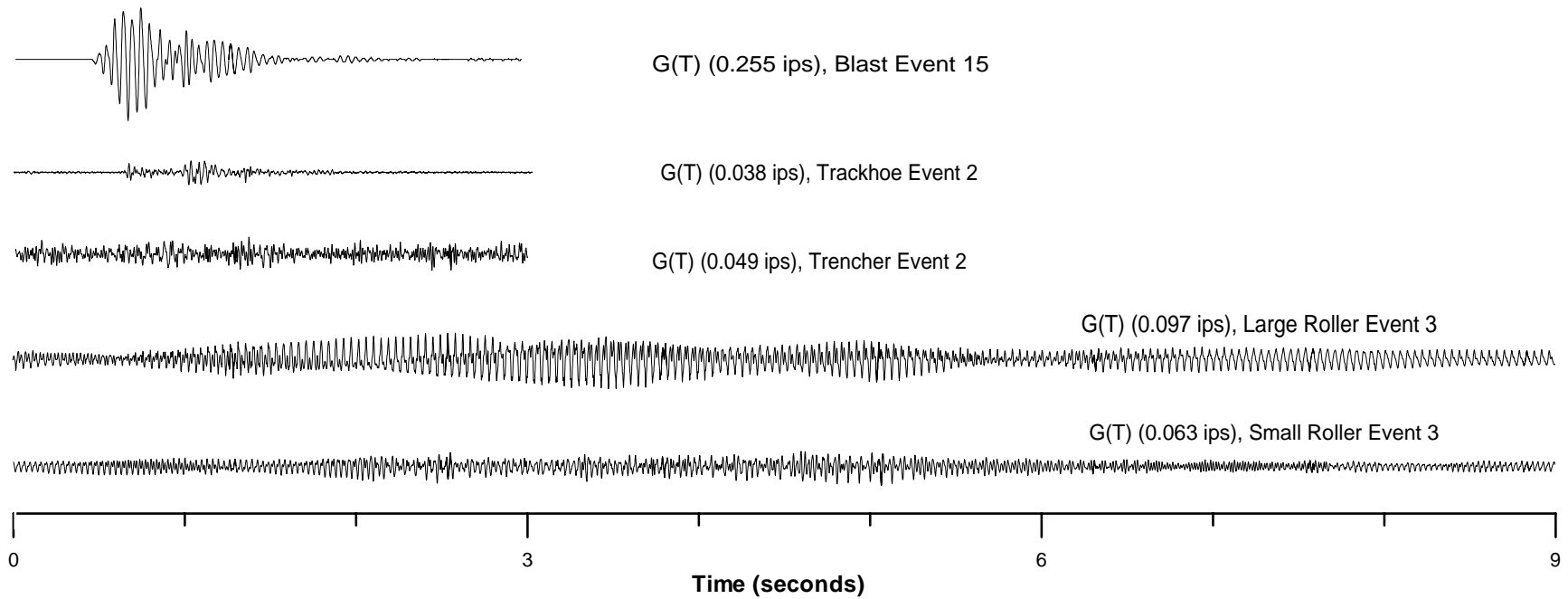
**Figure 6.1 (b) 3-second time history magnification of Connecticut Crack 1 and 3 displacements from blast event 15 and Las Vegas Crack 2 displacement from trackhoe event 2, trencher event 2, small vibratory roller event 3 and large vibratory roller event 3, showing detail of waveforms**



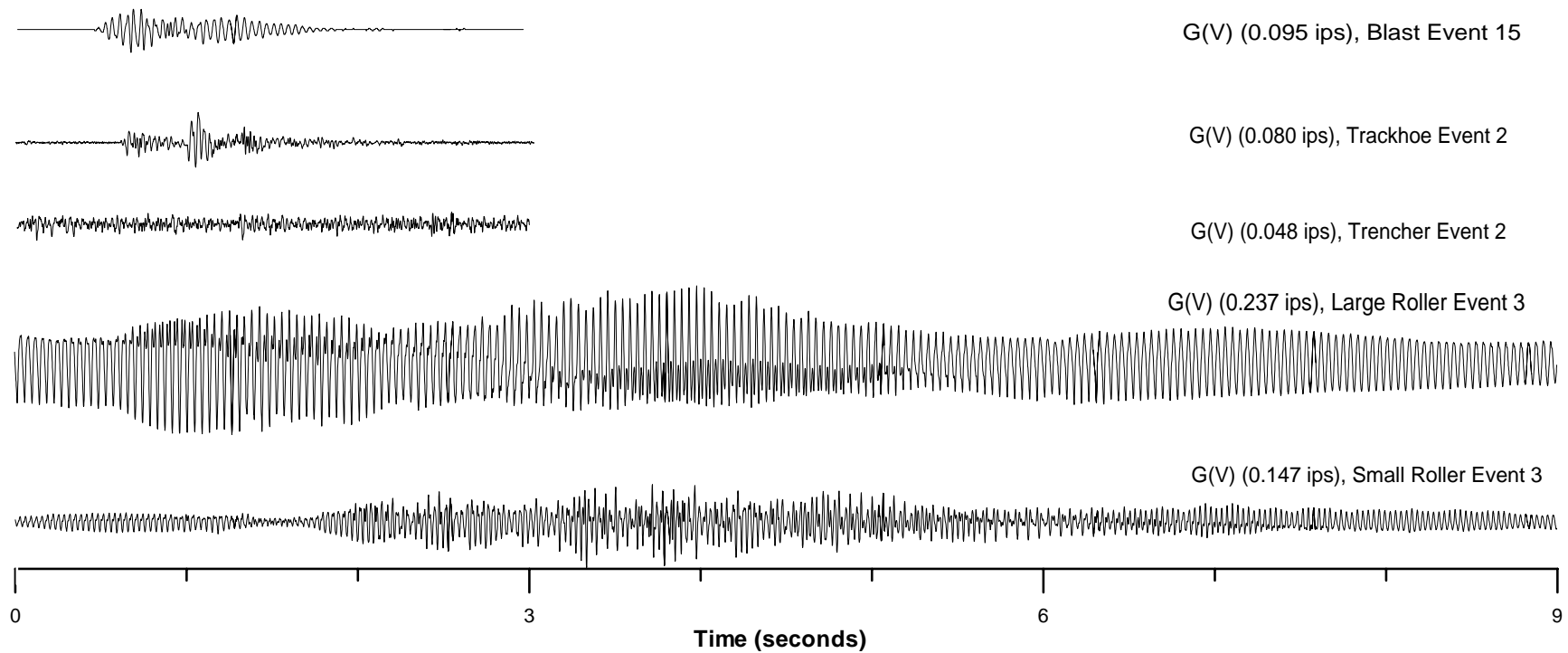
**Figure 6.2 (a) Global time histories of longitudinal particle velocity from blast event 15 in Connecticut, and trackhoe event 2, trencher event 2, small vibratory roller event 3 and large vibratory roller event 3 in Las Vegas**



**Figure 6.2 (b) 3-second time history magnification of longitudinal particle velocity from blast event 15 in Connecticut, and trackhoe event 2, trencher event 2, small vibratory roller event 3 and large vibratory roller event 3 in Las Vegas, showing detail of waveforms**



**Figure 6.3 Time histories of transverse particle velocity from blast event 15 in Connecticut, and trackhoe event 2, trencher event 2, small vibratory roller event 3 and large vibratory roller event 3 in Las Vegas**



**Figure 6.4 Time histories of vertical particle velocity from blast event 15 in Connecticut, and trackhoe event 2, trencher event 2, small vibratory roller event 3 and large vibratory roller event 3 in Las Vegas**

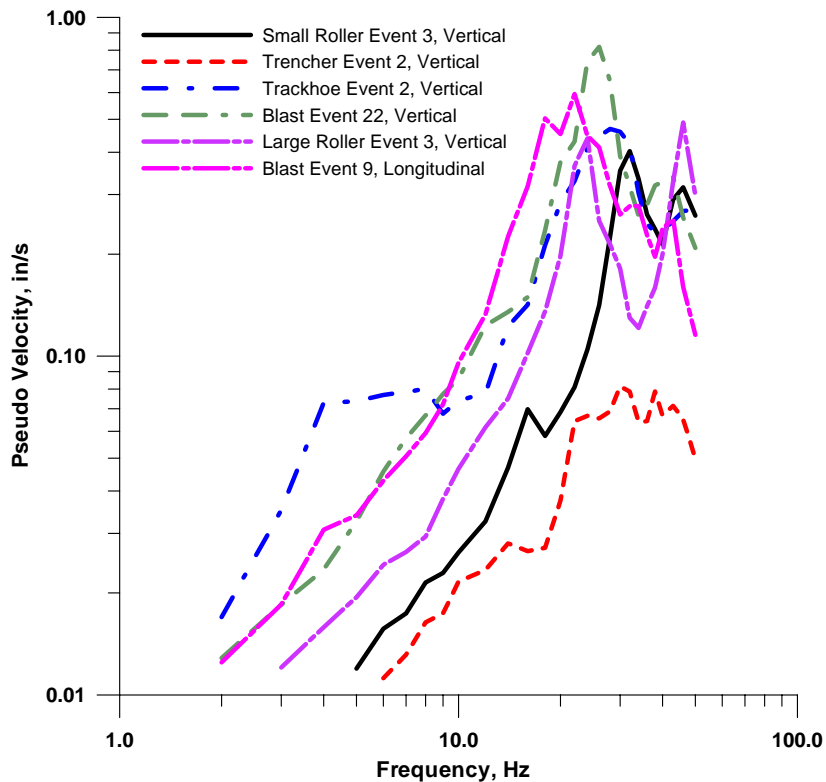


### *Single Degree of Freedom Response Spectra*

Since response spectrum techniques encompass the full waveform, they can be employed to visualize expected differences in response from the widely varying excitation and displacements. The critical response factor in predicting and quantifying the cracking potential of construction vibrations is discussed in terms of differential movements that occur between structural components. The importance and influence of ground vibration and natural structural frequencies on these differential movements has been studied and well documented. Construction and blasting vibrations achieve maximum amplitude response with dominant frequencies above the natural frequency of the responding superstructure and near and above that of the walls.

Events studied in Chapters 2 and 5 occurred at varying distances from their respective structures, thus some means of normalization is needed. Comparing the crack response to machinery excitation 40 feet away to that from the large roller 8 feet away presents misleading information.

A two-step normalization process was followed. First, all construction equipment vibration was normalized by considering only events that occurred at a typical distance of 30 feet ( $\approx 10\text{m}$ ). Normalization to a 30 ft stand-off distance eliminates the difficulty of comparing varying peak particle velocities from different equipment produced as a function of differences in stand-off distance. This normalization produces similar peak particle velocities (0.1 to 0.12 ips) at 30 ft in the trackhoe and vibratory rollers, while the trencher produces far smaller peak particle velocities (0.03 ips). Comparison of construction vibrations with those produced by blasting through the normalization process is more challenging. Blasting occurred at distances in excess of 2000 ft, so it was decided as the second step in the normalization process to employ only blast events that produced peak particle velocities (0.14 and 0.13 ips) similar to those produced by the trackhoe and rollers at 30 ft. Figure 6.5 presents the single degree of freedom response spectra for all 30 ft. normalized construction events, plus the two blasting events producing similar peak particle velocities. These spectra will be discussed in the following section.



**Figure 6.5 Vertical single degree of freedom response spectrum for blast event 22 in Connecticut (PPV=0.13 ips) and trackhoe event 2, trencher event 2, small roller event 3 and large roller event 3 in Las Vegas (PPV normalized to 30 feet), and Longitudinal SDOF spectrum for blast event 9 in Connecticut (PPV=0.14)**

### Homogenous Excitation

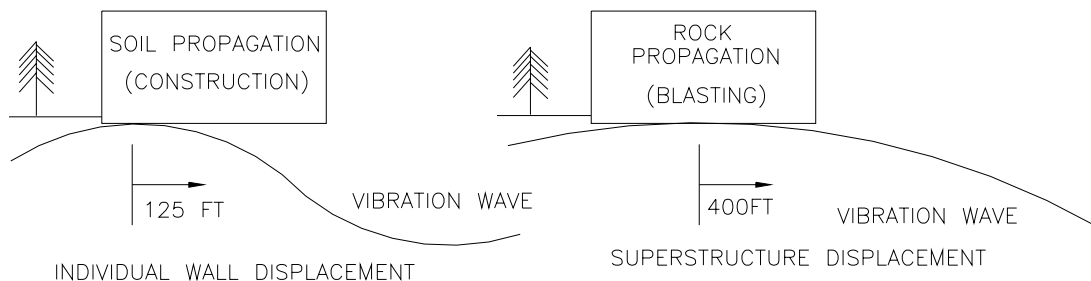
The wavelength and proximity of the excitation source control the degree to which the structure is excited homogeneously or uniformly. Comparison of blast and construction vibration effects provides an opportunity to illustrate the effects of uniform versus localized excitation. Vibration wavelengths are a function of excitation frequency and propagation velocity as shown by Equation 5.1

$$\lambda = c * (1/ f)$$

where  $\lambda$  is wavelength,  $c$  is propagation velocity and  $f$  is frequency. In addition, vibrations from construction normally involve soils, whose relatively high compliance compared to rock leads to shorter wavelengths. The following discussion illustrates the importance of these concepts.

Two types of structural motion, or displacement, are responsible for micro-inch crack displacements: homogenous superstructure displacement and individual wall component displacement. Superstructure displacements traditionally occur at frequencies between 5 and 12 Hz, while individual walls respond at 12 to 20 Hz. Wavelengths have significant influence over the degree of homogenous superstructure versus individual wall component displacement occurring as a result of construction or blasting vibrations, making them an important comparison. For rock blasting at the Connecticut site, the propagation velocity is approximately 10,000 ft/sec. Vibration frequencies maximize between 20 and 30 Hz. The wavelength for blast event 15 is approximately 400-450 ft. In Las Vegas soils, the propagation velocity is significantly lower, about 4000 ft/sec. Vibration frequencies for the construction equipment at the Las Vegas site were approximately 28 Hz for the trackhoe, 35 Hz for the trencher, 32 Hz for the small roller, and 23 Hz for the large roller (in Event 3). These wavelengths vary between 100-200 ft.

Figure 6.6 compares vibration wavelengths to scale with the long axis of the Connecticut and Las Vegas houses. Wavelengths are 7 to 8 times longer than the long axis of the house for rock motions in Connecticut, thus the vibrations generate homogenous superstructure motion at anywhere from 5 to 12 Hz. At the Las Vegas site, the approximate wavelengths of the construction vibrations are only 2 times longer than the long axis of the house, and produce individual wall component displacement at higher (18-22 Hz) frequencies. From the single degree of freedom spectra presented earlier in Figure 6.5, it is obvious that these natural frequency response variations produce significantly differing relative displacement structural response.



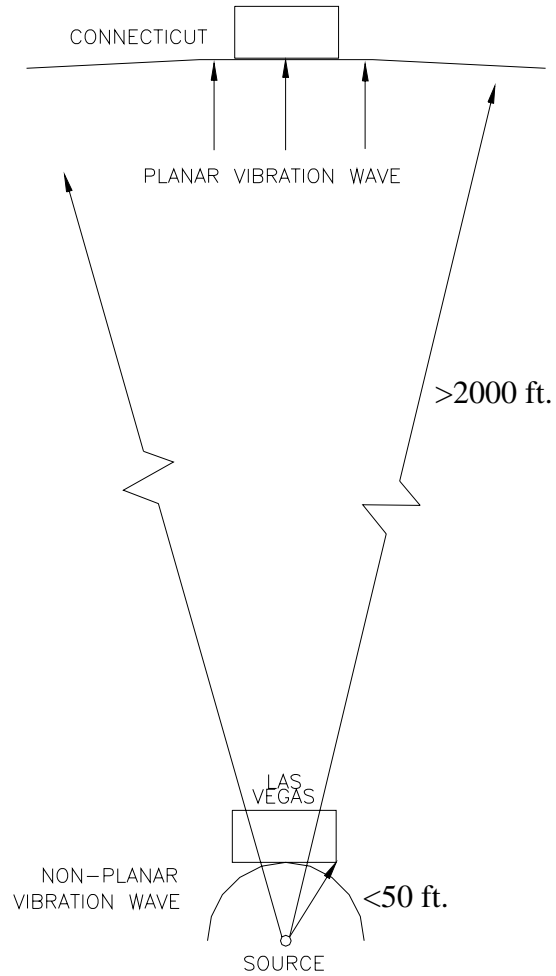
**Figure 6.6 Visual comparison of wavelengths ( $\lambda$ ) producing individual wall component displacement with soil propagation ( $\lambda=125$  ft) and homogenous superstructure displacement with rock propagation ( $\lambda=400$  ft)**

Absolute distance from the energy source producing blasting or construction vibrations also affects the uniformity or homogeneity of the excitation. Figure 6.7 shows both the Connecticut house, at approximately 2500 ft. from the source of blasting at the quarry, and the Las Vegas house, between 10 and 50 ft. from the activities of the construction equipment. As the energy propagates outward from the source (all assumed to create a circular wavefront in plan view), the radius of curvature of the front increases with distance and the particle motions within the rock and soil become increasingly parallel. When this particle motion becomes parallel, the wave can be approximated as a plane wave. At distances greater than 100 ft. from the source for a 60 ft. wide house, the wavefront is said to behave as a plane, or without curvature (Dowding, 1996). As stated before, the Connecticut structure lies well outside this 100 ft. range, meaning the waves coming from the quarry are most certainly acting as a plane-wave. The Las Vegas structure, however, is closer than 100 ft from the source of construction vibrations. Therefore, different portions of the structure can be expected to react to the same ground motions within a time history at differing times.

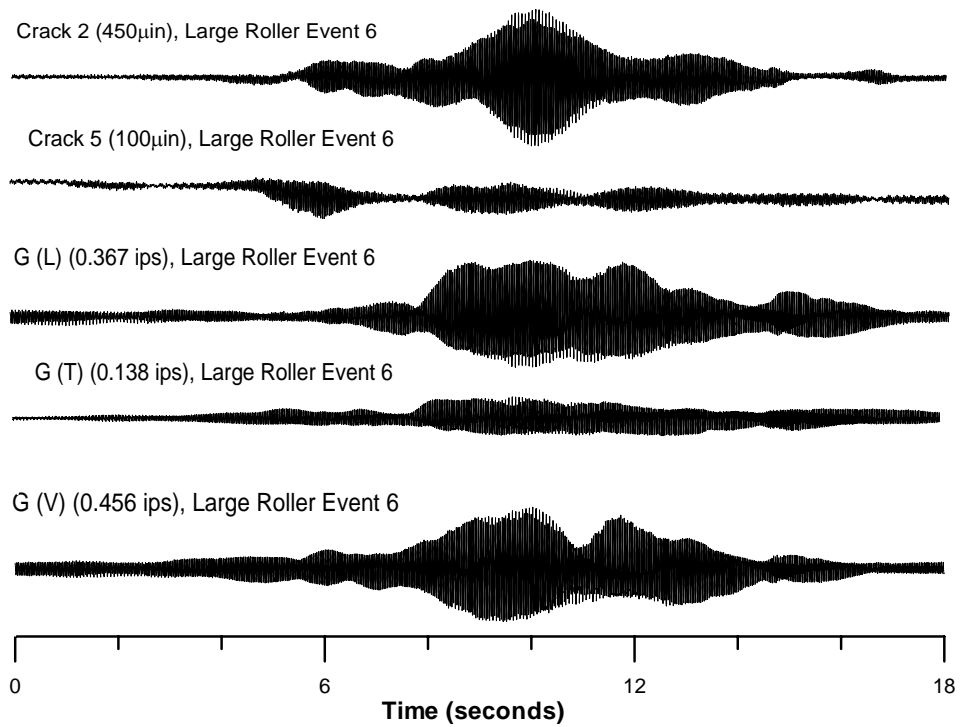
Differing times of maximum response illustrate the local nature of excitation with unusually close construction vibration. In addition, these timing differences can make data collection of construction-induced crack displacements with a single geophone a significant challenge. In Connecticut, where the wave moves as a plane, the vibration of the entire superstructure can be assumed to occur at the exact same time (to the millisecond) as the vibration of the geophone. At the Las Vegas site, however, there is a significant time lag between the maximum vibration of the geophone and the maximum response of Crack 5, situated on the western exterior wall approximately 30 feet from the location of the geophone.

Figure 6.8 shows Crack 2 and 5 time histories compared with longitudinal, transverse, and vertical ground velocity time histories (bottom) resulting from large roller event 6. It can be seen that, while the peak displacement of Crack 2 occurs at roughly the same time as the peak ground motions, the peak response of Crack 5 occurs four to five seconds before the peak ground motions at the geophone. Figure 6.9 shows that, if the roller is moving west to east (as it was during this event), that when the peak motions are

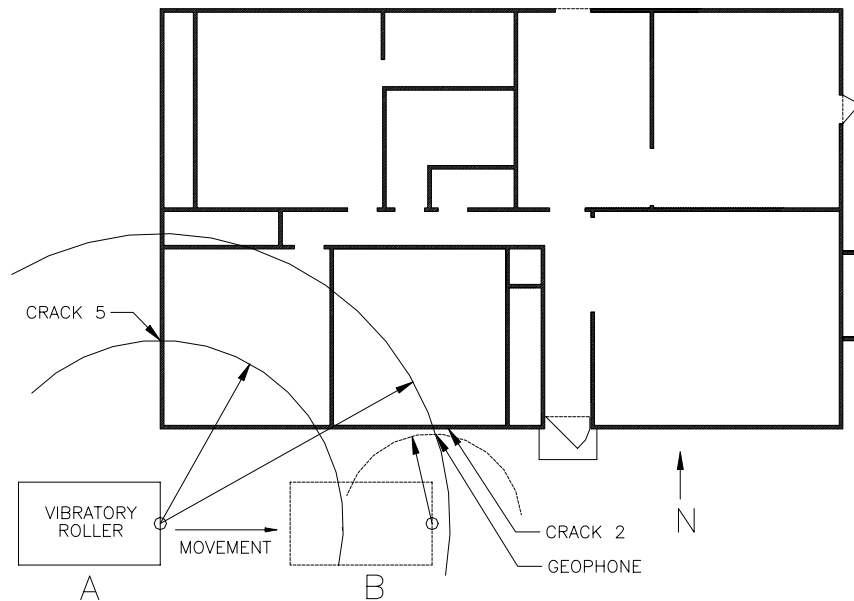
occurring at the wall containing Crack 5, they will be significantly attenuated by the time they reach the geophone 30 feet away.



**Figure 6.7 Plane wave theory of radial geodynamic wave motion impacting structures in Connecticut and Las Vegas**



**Figure 6.8 Crack 2 and 5 displacement, longitudinal, transverse and vertical time histories of large roller event 6 showing time lag in Crack 5 versus Crack 2 response**



**Figure 6.9 Visualization of time lag existing between Crack 5 response and Crack 2 response during large roller event 6, resulting from radial plane wave motion**

### Crack Sensitivity

The final descriptor that will have a significant effect on crack displacements is the sensitivity of the wall and structure to any effect, be it produced by transient vibration or long-term weather. Response of structural components depends on many different loading effects such as foundation movements, in-home climate control, changes in weather patterns, etc. Sensitivity depends on wall material composition, framing details, support details, etc. To compare sensitivities independent of vibration source, the long-term data from Chapters 2 and 5 are examined to choose extreme weather events resulting in significant humidity changes for the determination of crack response. Sensitivity was calculated as the change in 24-hour rolling average crack displacement ( $\Delta$  Crack Displacement) divided by the corresponding change in 24-hour rolling average of humidity ( $\Delta$  Humidity). Each of these ratios was then plotted as Figure 6.10 to determine an average. Not all these points were produced by the same humidity event, as some rain events last longer than others, some produce less rainfall with a greater humidity change, etc. Thus normalizing these factors out is particularly difficult. Table 6.2 lists the events and responses employed to produce these comparisons.

In Figure 6.10 separate crack sensitivity functions are calculated by assuming, in the short-term, zero change in humidity will produce zero crack displacement. Each crack is thus associated with a specific sensitivity. Figure 6.10 (a) shows Connecticut Crack 1 sensitivity is  $400\mu\text{in}/\%$ , and Crack 3 is  $115\mu\text{in}/\%$ . Figure 6.10 (b) shows Las Vegas Crack 1 sensitivity is  $150\mu\text{in}/\%$ , Las Vegas Crack 3 is  $130\mu\text{in}/\%$ , Las Vegas Crack 2 is  $165\mu\text{in}/\%$ , and Las Vegas Crack 5 is  $150\mu\text{in}/\%$ . As expected, Connecticut Crack 1 is by far the most sensitive to changes in humidity, followed by the external Las Vegas cracks, and finally the Las Vegas ceiling and Las Vegas and Connecticut drywall cracks.

Establishing criteria for vibration induced crack sensitivity and finding a method of correlating sensitivities in Connecticut to those in Las Vegas presents several challenges. It has been established earlier that the structures studied in Connecticut and Las Vegas are constructed of different materials with different support systems. Furthermore, blasting and construction vibrations produce significantly different excitation motions in terms of homogeneity of excitation as well as time history.

Comparing crack displacements resulting from homogenous structural motion of a two-story structure with crack displacements from localized structural motion of a single-story slab-on-grade structure would require a sophisticated and elaborate normalization procedure beyond the scope of this project. To make matters more challenging, as shown above each crack responds with its own sensitivity to environmental and (by implication) vibratory effects. This difference must be normalized as well. Ideally, monitoring blasting and construction vibration-induced crack displacements in the same structure would effectively normalize these factors.

	Date	Humidity Change (%)	Crack 1 Change ( $\mu\text{in}$ )	Crack 3 Change ( $\mu\text{in}$ )	Crack 1 Change ( $\mu\text{in}$ )	Crack 2 Change ( $\mu\text{in}$ )	Crack 3 Change ( $\mu\text{in}$ )	Crack 5 Change ( $\mu\text{in}$ )
Connecticut								
	6/27/2002	11	5000	1000				
	6/29/2002	13	7500	1500				
	7/26/2002	25	10000	2500				
Las Vegas								
	9/12/2002	18				2900		
	10/28/2002	40				6500		5700
	10/30/2002	30						5000
	11/9/2002	22				4100		2200
	9/10/2002	7.69			1100		950	
	10/31/2002	8.31			1320		1150	
	12/20/2002	2.65			470		365	

**Table 6.2 Events and crack responses employed in the calculation of crack sensitivity to environmental (humidity) change**



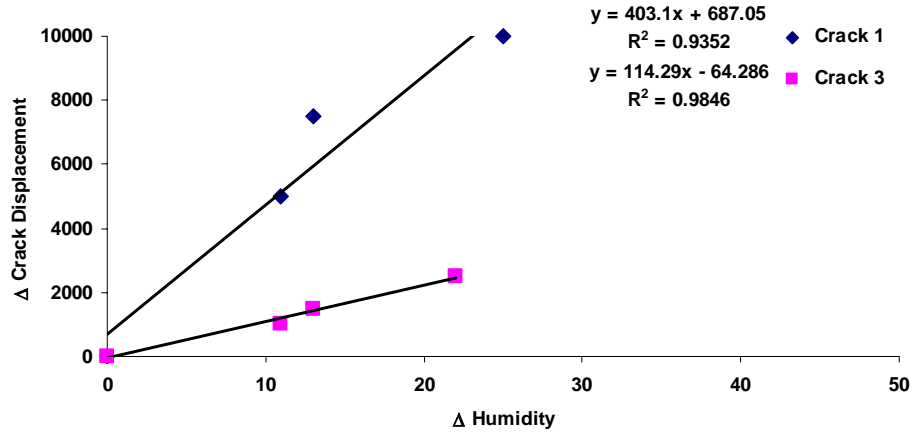


Figure 6.10 (a) Connecticut Cracks 1 and 3 sensitivity to humidity change

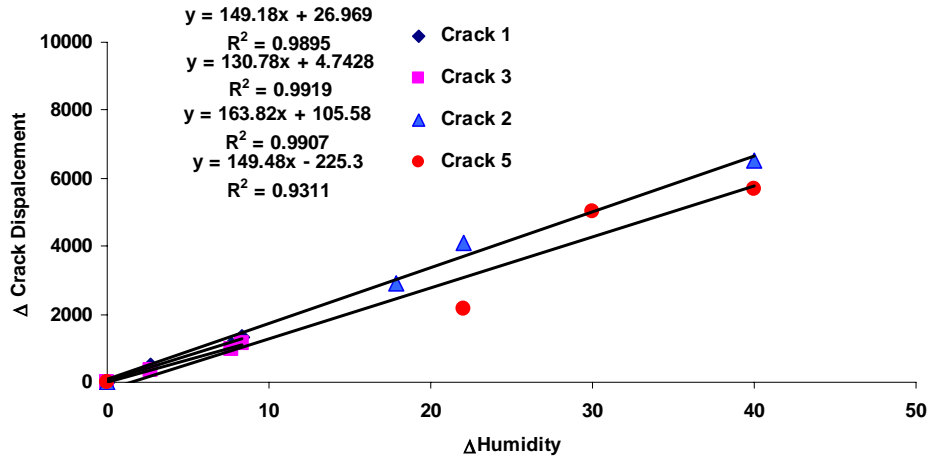


Figure 6.10 (b) Las Vegas Cracks 1, 2, 3, and 5 sensitivity to humidity change

## *Chapter 7*

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### *Conclusions*

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This thesis summarizes micro-inch crack response to blast-induced ground motions from the Stiles Road Quarry in Southbury, Connecticut, construction-induced ground motions along West Ann Road in Las Vegas, Nevada, and environmental phenomena at both sites. Both of these structures were instrumented and their response studied as part of the development of an Autonomous Crack Measurement (ACM) system sponsored by the Infrastructure Technology Institute at Northwestern University through a block grant from the United States Department of Transportation.

Responses of the Connecticut structure were measured with velocity transducers instrumented in the traditional manner of those in the study sponsored by the United States Department of the Interior Office of Surface Mining (OSM). Ground motions were measured in three orthogonal axes in front of the house. Structural response was measured with three upper structure velocity transducers, three lower structure velocity transducers, two mid-wall velocity transducers, an air pressure transducer, and for the final month of study, a mid-ceiling velocity transducer. One wall and two ceiling cosmetic drywall cracks were fit with eddie current “Kaman” sensors to measure micro-inch displacement response to environmental and blast-induced ground vibrations produced by an aggregate quarry approximately 2500 feet away.

Ground motions at the Las Vegas structure were measured with an in-ground tri-axial geophone customary to all ACM sites. Micro-inch displacements were measured across two interior drywall cracks and two exterior stucco cracks with LVDT displacement gages. No velocity response was measured in this structure. Construction adjacent to the house (all within 50 feet) involved excavation for the installation of a 10'x12' reinforced concrete box culvert by trackhoe, excavation of an 8-inch water service line trench by chain trencher, and vibratory compaction of trench backfill and granular sub-grade for the reconstruction of West Ann Road.

The synthesis of measurements and calculations from the response of the Connecticut site structure led to the following conclusions:

- Long-term environmental and weather crack displacement (temperature and humidity) in this structure is 40 to 90 times greater than the crack displacement caused by either the largest measured blast-induced ground motion (0.345 ips) or airblast (132 dB).
- Crack 1 responded as much, and correlated better to, large air over-pressures (+130 dB) as it did to large ground motions (+0.30 ips). Bedroom wall Crack 3 responded insignificantly to airblasts.
- Response of ceiling Crack 1 correlated best to measurements that involve absolute mid-ceiling displacements.
- Upper and lower structure response must be monitored on single-story walls constructed of the same material. If multi-story walls are instrumented, they must be of the same construction. Failure to isolate differing construction types results in erroneous relative displacement calculations.
- To compare structure and crack response, structural response should be measured along the same wall, or at least in the same portion of the house, as the crack itself.

Examination of the effects of blast design on crack displacements at the Connecticut site led to the following conclusions:

- Frequency of blast-induced ground motions had a significant effect on crack displacements.

- Crack 1 (apartment ceiling) response to unusually high airblast overpressure was as large as that from significant ground motions.
- Total number of holes, delay timing, or total shot time individually had less impact on crack displacement than frequency or the presence of unusually high airblasts.
- Borehole geometry and layout had a relatively minor, albeit existent, effect on crack displacement.

The synthesis of measurements and calculations of response from the Las Vegas, Nevada site structure led to the following conclusions:

- Trackhoe, trencher, and vibratory roller construction in the vicinity of the structure (<50 ft) did not create significant (<0.5 ips) ground motions.
- Long-term environmental and weather-induced crack displacement was 30 to 150 times greater than the crack displacement caused by the largest measured construction-induced ground motion (0.435 ips).
- Cracks in Las Vegas site structure appear to displace in a stick-slip fashion, rather than evenly over time. Further examination of this phenomenon is needed.
- One hour of typical weather-induced Crack 2 displacement was twice as large as that produced by the largest vibratory event (0.435 ips) occurring during the same time period.
- On-site inspection of vibration-inducing construction activities would greatly diminish the difficulty in identifying specific sources of excitation. Unique data acquisition triggers make the monitoring process accurate, albeit difficult.
- Electrical noise, human sources, and other unidentifiable activities may trigger systems while monitoring construction vibrations, and methods of identifying these events are important.
- For excitation frequencies between 20 and 30 Hz, crack displacement correlated best with peak particle velocity.

The comparison of measurements and results from the structures at the Connecticut and Las Vegas sites led to the following conclusions:

- Due to shorter wavelengths, nearby construction vibrations at the Las Vegas site from movable sources produced response more localized in time than were those from longer wavelength, distant blast-induced vibrations at the Connecticut site.
- Localized in time, component responses at the Las Vegas structure may have been accentuated by the small radii of curvature of the excitation vibrations.
- Cracks at the Connecticut and Las Vegas sites reacted to changes in humidity with different sensitivities, which may be the result of differences in construction.

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## *Appendix A*

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### *Conversion Factors for Connecticut and Las Vegas Instruments*

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#### Connecticut

Kaman Gage/Crack 1, Apartment Addition

- $\text{mils} = (-0.014317) + (2.2085 * V) - (0.31059 * (V^2)) + (0.29495 * (V^3)) - (0.080215 * (V^4)) + (0.0095915 * (V^5)); V = \text{SoMat Voltage}$

Kaman Gage/Crack 2, Living Room Ceiling

- $\text{mils} = (-0.034437) + (2.2312 * V) - (0.47524 * (V^2)) + (0.40754 * (V^3)) - (0.10769 * (V^4)) + (0.011889 * (V^5)); V = \text{SoMat Voltage}$

Kaman Gage/Crack 3, Bedroom Wall

- $\text{mils} = (-0.018637) + (2.4437 * V) - (0.7242 * (V^2)) + (0.52212 * (V^3)) - (0.013227 * (V^4)) + (0.013873 * (V^5)); V = \text{SoMat Voltage}$

Kaman Gage/Crack 4, Null (in Apartment)

- $\text{mils} = (-0.0019227) + (2.2032 * V) - (0.25826 * (V^2)) + (0.25371 * (V^3)) - (0.069226 * (V^4)) + (0.0086497 * (V^5)); V = \text{SoMat Voltage}$

micrometers = mils\*25.4

microinches = micrometers\*39.37

#### Las Vegas

LVDT Displacement Gages, All Crack and Null Response

- $\text{micrometers} = V/7.87; V = \text{eDAQ Voltage}$
- $\text{microinches} = \text{micrometers} * 39.37$

Geosonics Inc. Tri-axial 4.5 Hz Geophone

- $\text{Longitudinal, Transverse and Vertical Velocity (in/sec)} = V/1000/0.8$

## ***Appendix B***

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### ***Specifications for Construction Equipment in Las Vegas***

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#### **Hitachi 1200 EX Super Trackhoe**

- Operating Weight = 108,000 kg
- Body Weight = 77,170 kg
- Normal Bucket Capacity = 3.8 m<sup>3</sup>
- Track Width = 710 mm
- Track Length = 6,410

#### **Tesmec TRS 1175XL Chainsaw**

- Weight w/ 10-foot Boom and 26-inch Rotary Cutters = 61,235 kg
- Chainsaw Transport Length w/ 10-foot Boom = 12.4 m
- Net Flywheel Horsepower at Full Load RPM = 370

#### **Dynapac CC 522 Single-Drum Roller**

- Operating Mass = 26,130 lbs
- Frequency/Amplitude High = 3,000 vpm / 0.028 in.
- Frequency/Amplitude Low = 3,000 vpm / 0.012 in.
- Centrifugal Force High/Low = 28,800 / 14,625 lbs
- Speed = 0 to 7.5 mph
- Drum Width/Diameter = 77 in. (6.4 ft.) / 55 in. (4.6 ft.)

#### **Ingersoll-Rand SD 115F Single-Drum Roller**

- Operating Mass = 26,420 lbs
- Frequency = 18.3-31.5 Hz
- Nominal Amplitude = 0.054 / 0.027 in.
- Centrifugal Force High/Low = 55,000 / 27,400 lbs
- Speed = 0 to 7.1 mph
- Drum Width/Diameter = 84 in. (7 ft.) / 59 in. (5 ft.)