STRUCTURE RESPONSE OF TWO STRUCTURES NEAR THE SIBLEY LIMESTONE QUARRY, LLC, TRENTON, MICHIGAN

Prepared by

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INTRODUCTION

A structure response study was conducted at two structures near the Sibley Limestone Quarry, LLC in Trenton, Michigan. The purpose of this study is to evaluate the impacts of blasting on structures, specifically cracking potential, compared with the cracking potential from the influences of normal, every day human activities and the effects of changes in climate.

The Welch resident, at 19473 Wedgewood, and the Marian Manor, at Quarry St., were instrumented on June 15 and 16, 2005 with velocity and crack gages. Velocity gages were mounted in each structure to compute both whole structure and mid-wall strains from blasting and human activities. Blast-induced motions were recorded during fifteen quarry blasts conducted between June 16 and July 28, 2005. Crack displacement gages, applied over existing wall cracks in each structure, were employed to measure dynamic crack displacements during blasting and human activity as well as static, or weather-induced, crack movement during changes in ambient temperature and humidity. This report summarizes the findings of structure and existing crack responses to blasting compared with responses to normal everyday human activities and weather changes. Wall and bending strains computed from structure motions were 80

for all dynamic events.

The location and identification of residential structures to the west and northeast of the quarry are shown in Figure 1. To the right (at a larger scale) is an inset showing the distribution of production blasts conducted throughout this study. These blasts were conducted along the east of the quarry both to the north and to the south. Figures 2 and 3 show the instrumentation within the Welch and Marian Manor structures, respectively.

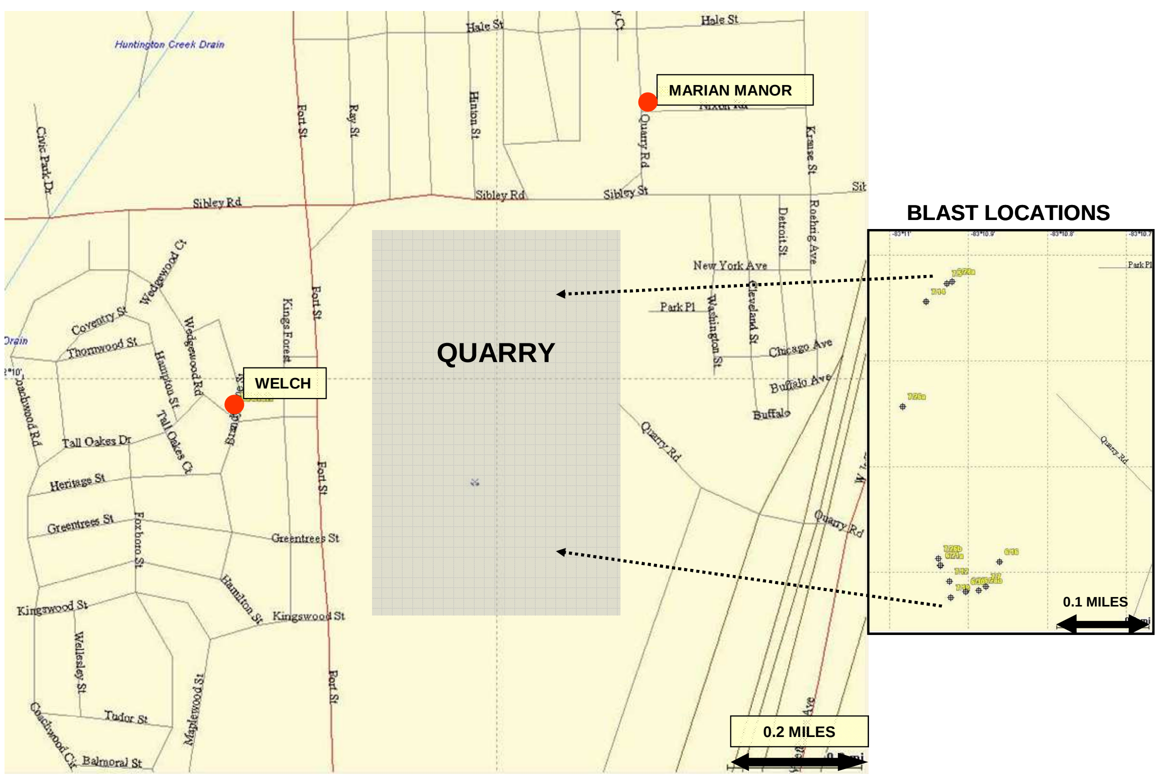
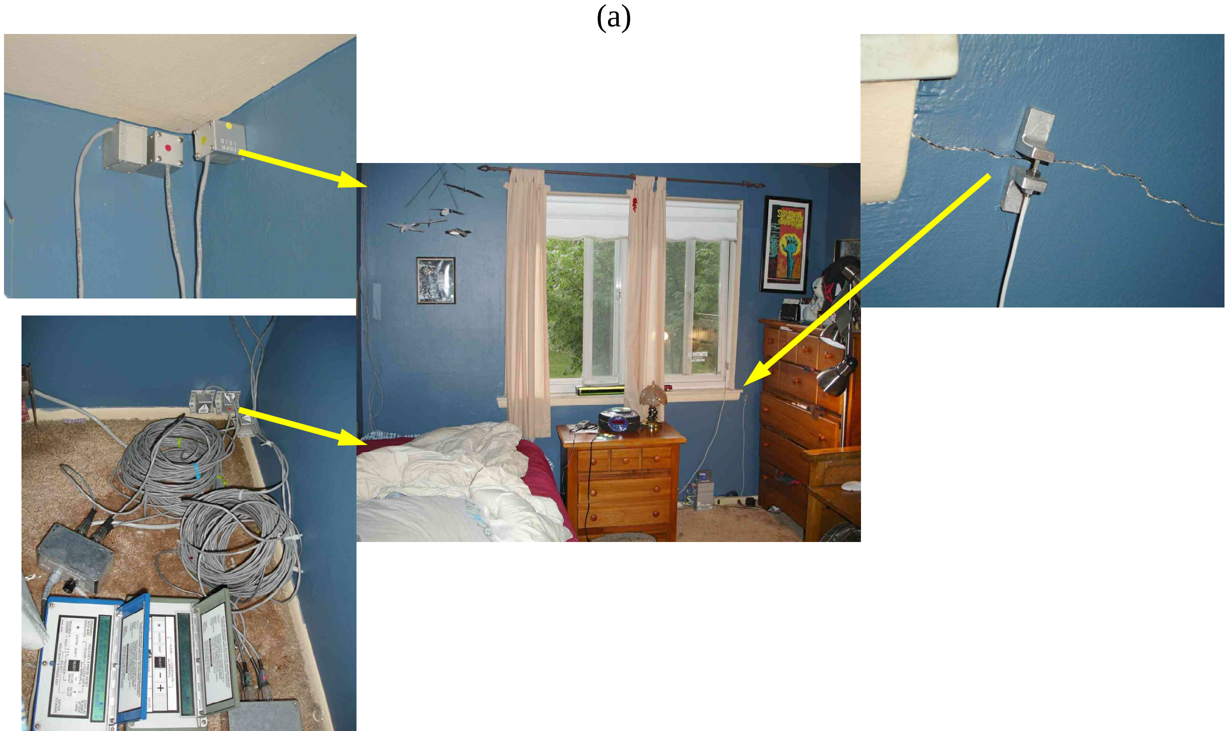


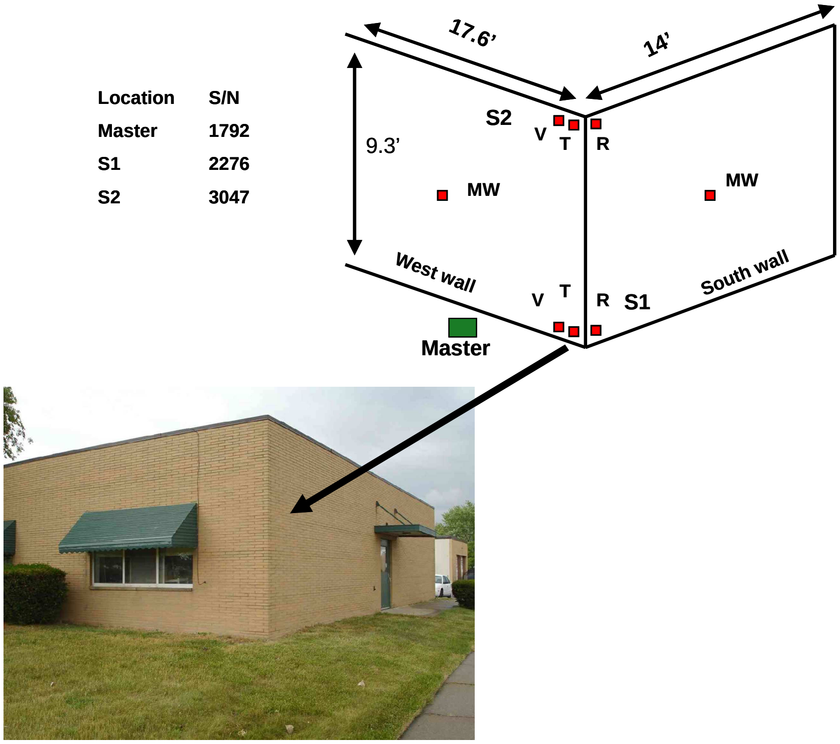
Figure 1 Location map showing production blast locations and structures used for instrumentation

(a)



(b)

Figure 2 Welch residence showing (a) schematic of vibration monitoring locations and (b) photographs of instrumentation in the second floor bedroom



(a)

(b)

Figure 3 Marian Manor wing "A" showing (a) schematic of vibration monitoring locations and (b) photographs of instrumentation

The Welch residence is a two-structure wood frame house with brick veneer and vinyl siding. The instruments were located in the second story south corner bedroom. A diagonal drywall crack from the lower corner of the southwest window was instrumented. The Marian Manor is a concrete masonry brick structure with an exterior comprising a fire-clay brick façade. The interior brick corner on the southwest was instrumented along with a crack in the brick near a window.

PROCEDURES

Vibration and Airblast Instrumentation

LARCOR® multi-component seismographs were used to digitally record four channels of seismic data from velocity transducers. The locations of the single component transducers placed in the upper (S2) corner, lower (S1) corner, and at the mid-wall (MW) of adjoining walls are indicated in Figures 2 and 3. In the Welch residence, a second story bedroom was used for the structure response study as the room contained a large diagonal crack starting from the lower right hand portion of the front windows facing in the southwest direction.

At the upper and lower structure corners two horizontal components, radial (R) and transverse (T) along with a vertical (V) component, were mounted. A fourth horizontal sensor was mounted at center of the adjoining walls (mid-wall, MW) at Marian Manor. Only one mid-wall sensor (southwest wall) was used at the Welch residence as the southeast wall bedroom was covered by a wall-sized mural photograph that could not be destroyed.

The exterior (master) unit consisted of a tri-axial geophone and an airblast microphone. The geophones, buried 4 inch in depth, were oriented so that the R component was parallel with the longest axes of the house. This orientation is based upon recording motions that are parallel to one of the house's translation axes rather than the traditional direction relative to the vibration source. The airblast microphone was installed 4 to 6 inch above the ground surface and used to record the pressure pulses transmitted through the air during blasting.

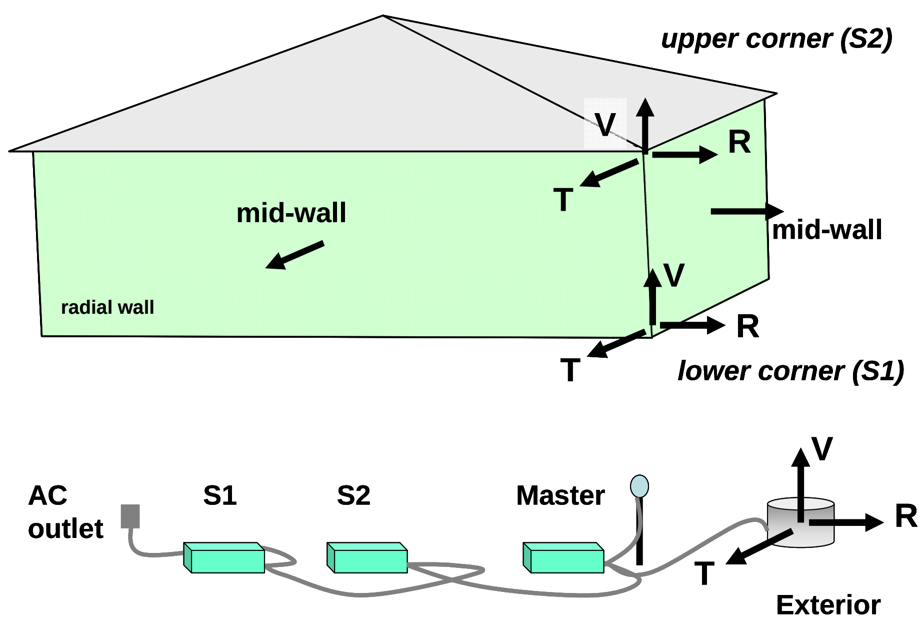


Figure 4 Position of seismographs connected in series for time-correlated measurements

Seismographs were connected in series, with the exterior master seismograph acting as the triggering unit and all other systems as slave units, generically shown in Figure 4. Transducers were affixed to the walls using hot glue to minimize damage during removal. Two corner transducers measured whole structure motions in the horizontal directions (R and T) and were used to calculate in-plane tensile strains. The mid-wall transducers measured horizontal motions during wall flexure and used to calculate bending strains.

Existing Crack Gage Instrumentation

To measure the effects of blasting and variations in climate conditions (temperature and humidity) on changes in the width of existing interior cracks, Kaman® eddy-current displacement gages were installed and data was collected using a SOMAT® field computer. A schematic of the data acquisition system and gages are given in Figure 5. Each Kaman gage consisted of mounting brackets, one of which served as a target plate, and an active element. Mounting brackets were placed on either side of the crack (crack gage) and on an un-cracked surface (null gage) such that the active element was secured against the target plate at a sufficient gap distance to allow the gage to function properly. Displacement data for the null gage was subtracted from the crack gage data to obtain net crack motions without the influence of the construction material alone.

Operation of eddy-current gages relies on the property of electrical induction. The sensor consists of a coil of wire driven by a high frequency current that generates a magnetic field around the coil. If a non-magnetic conductive target material is introduced into the coil field, eddy-currents are induced in the surface of the target material. These currents generate a secondary magnetic field in the target, inducing a secondary voltage in the sensor coil (active element), resulting in a decrease in the inductive reactance in the coil. This type of system is also known as variable impedance because of the significance of the impedance variations in defining its complex nature (Hitz and Welsby, 1997).

The three seismographs and Somat field computer were connected in series. Upon triggering, the master seismograph delivered a 1 volt pulse via the serial cable to activate and begin recording dynamic data during blasting events. This produced seismograph and dynamic crack/null gage records that were time-correlated. Time correlated data is critical for analysis of structural and crack response.

The master and slave seismographs each had a range of available settings for recording data. These settings include:

- trigger levels for the master unit set to 0.01 to 0.03 inch per second (ips) for ground velocity, and 125 decibels (dB) for airblast to avoid false triggering with high wind gust
- sample rate set at 512 samples per second
- a sampling duration of 8 seconds

These settings ensured the full data record was preserved with sufficient resolution.
The Kaman gage system was programmed to sample crack opening and closing every hour in response to diurnal environmental changes. In the dynamic or ‘burst’ mode, data was acquired every 0.001 seconds. Temperature and relative humidity were recorded using a SUPCO® data logger. A sample interval of 10 minutes was used.

The operating parameters of the Kaman gages are as follows:

- displacement monitoring range of 0.02 inches.
- output voltage range ± 5 volts.
- resolution of 3.94 micro-inch. (0.00000394 inch)
- frequency response of 10,000 Hertz (Hz).

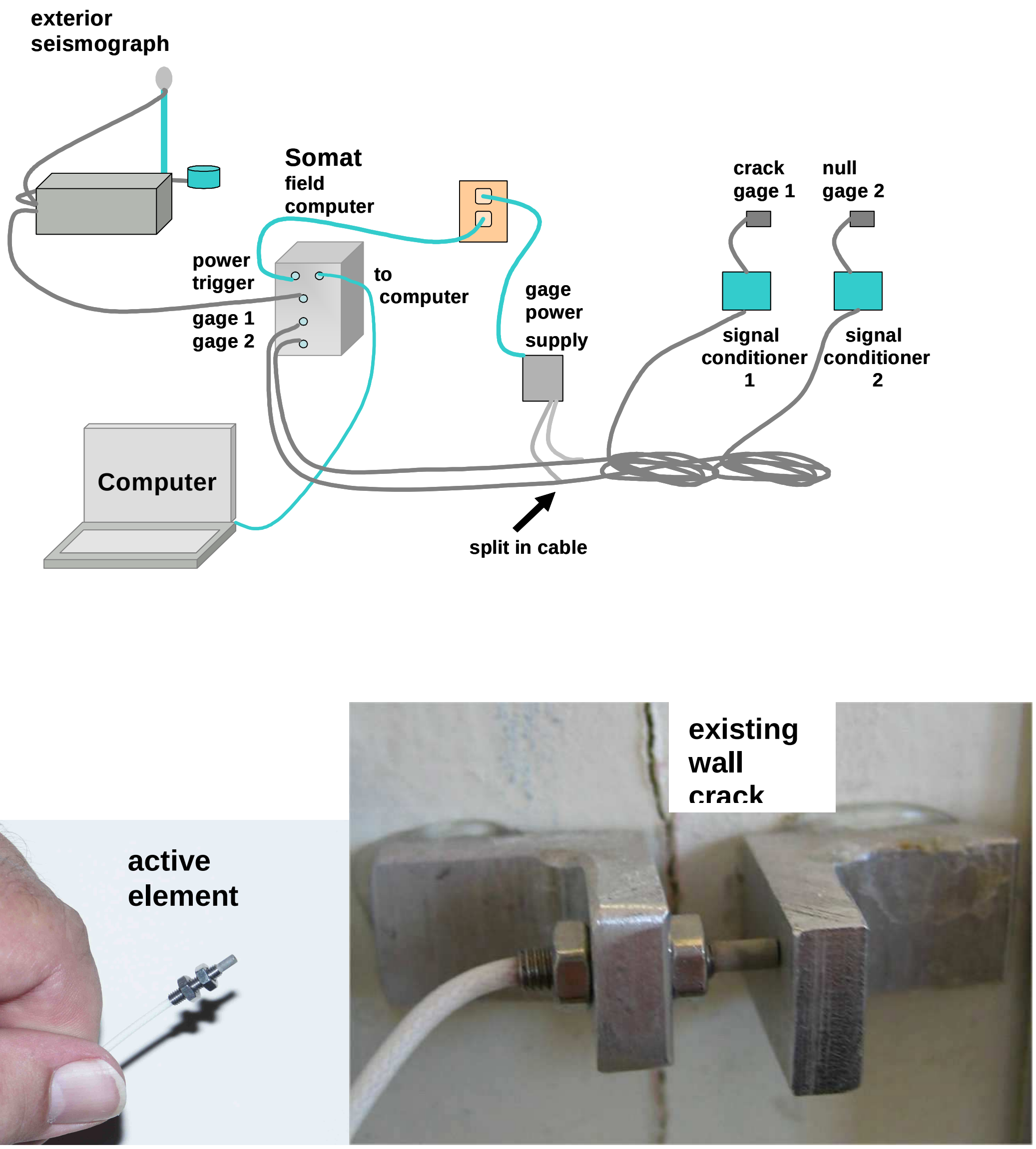


Figure 5 Displacement gage system used to measure opening and closing of an existing wall crack (above) and close-up of mounted crack gage (below)

Throughout this report, readers are reminded that the velocity unit of inches per second (ips) is one only of convenience. Neither the ground nor the structures move in inches but rather milli-inches. Structure damage in the form of cosmetic or threshold wall cracks do not result from high structure velocities, but rather high differential displacements in walls leading to high strains or strains that exceed the failure strain of the materials comprising the walls. Displacements are related with velocity in terms of frequencies of motions. Therefore the analysis herein emphasizes structure and wall strains as the indicator of potential damage to structures.

Data Analysis

Velocity data were analyzed using White 2000® and White 2003® software to plot velocity and displacement time histories and integrate velocity time histories. Two frequencies, or wave cycles (oscillations) per second, are of interest and were analyzed. The peak frequency is that frequency associated with the maximum velocity amplitude over the full time-history of motion. It is used to demonstrate compliance with frequency-based regulations. The predominant frequency, evaluated using Fast Fourier Transform (FFT) analysis, dominates over the entire time history. The FFT frequency carries the largest percentage of ground motion energy and is important when evaluating structure response.

Crack gage data were downloaded from the SOMAT™ field computer and analyzed using SOMAT WINTCS v.2.1.5 and v.2.1.4 and SOMAT™ DataXplorer v. 3 softwares. Crack displacement time histories were filtered using Data Filter, (Mercer, 2002) a spectrum filtering program to remove system noise and enhance the data signal-to-noise ratio.

RESULTS

<u>Ground Vibrations and Airblast</u>

Tables 1 and 2 summarize the test blasts conducted over a one and one-half month period. The charge weights used per delay for blasts did not significantly vary (from 84 to 94 lbs/delay) while the distances to each structure from the blasts were 2250 to 2880 ft and 1934 and 3635 ft at the Welch and Marian Manor, respectively. The larger range in distances for the Marian Manor resulted from the variation in blast site locations in the north-to south trend as shown in Figure 1.

Airblast levels at each structure shown in Tables 1 and 2 are exceptionally low and, for most blasts, were not detected by the sensitive airblast sensors (shown as <100 dB). Safe airblast limits for potential glass breakage, the threshold damage first observed in structures, is 150 to 160 decibels (dB). Airblast levels recorded were far below damaging level and therefore airblast did not contribute to cracking potential in structures.

The maximum, minimum and average ground motion velocities (in inches per second or ips) at the two structures were as follows:

	maximum	minimum	average
Welch	0.068	0.035	0.050
Marian Manor	0.165	0.010	0.054

The maximum values of 0.068 and 0.165 ips are considered to be very low. Structure threshold cracking does not start to develop until ground vibrations are far greater then 0.75 ips, depending on the frequencies of the vibrations. However, humans inside structure can generally

Table 1 Summary of quarry blast shot information, vibration and airblast results at the Welch residence

Shot Date	Shot Time	Distance From Structure (in)	Charge Weight/Delay (lb)	Scaled Distance (ft/lb$^{1/2}$)	Peak Particle Velocity (in/sec)	Peak Frequency (Hz)	FFT Frequency (Hz)	Airblast (dB)
06/16/05	1:03 PM	2938	94	303.03	nt			
06/21/05	12:59 PM	2485	94	256.31	nt			
06/21/05	1:12 PM	2625	94	270.75	0.038	14.2	21.25	106
06/23/05	1:00 PM	2817	94	290.55	0.048	32.0	22.13	106
06/28/05	1:00 PM	2625	84	286.41	0.035	85.3	36.38	100
06/28/05	1:27 PM	2879	94	296.95	nt			
06/30/05	12:16 PM	2814	94	290.24	0.035	17.0	22.13	106
07/05/05	1:05 PM	2593	84	282.92	0.068	85.3	42.63	100
07/07/05	12:59 PM	2914	94	300.56	nt			
07/12/05	12:59 PM	2707	94	279.21	0.040	23.2	22.00	106
07/14/05	1:00 PM	2451	84	267.43	0.050	85.3	30.63	100
07/19/05	1:02 PM	2750	94	283.64	0.040	64.0	20.75	100
07/26/05	1:06 PM	2249	84	245.39	0.063	32.0	5.50	100
07/26/05	1:12 PM	2708	94	279.31	0.073	28.4	5.25	100
07/28/05	1:13 PM	2707	94	279.21	0.060	19.6	19.00	100

nt - no trigger

Table 2 Summary of quarry blast shot information, vibration and airblast results at the Marian Manor

Shot Date	Shot Time	Distance From Structure (in)	Charge Weight/Delay (lb)	Scaled Distance (ft/lb$^{1/2}$)	Peak Particle Velocity (in/sec)	Peak Frequency (Hz)	FFT Frequency (Hz)	Airblast (dB)
06/16/05	1:03 PM	3360	94	346.56	0.020	23.2	18.75	<100
06/21/05	12:59 PM	3512	94	362.24	0.010	32.0	18.75	106
06/21/05	1:12 PM	3465	94	357.39	0.020	16.0	21.38	106
06/23/05	1:00 PM	3560	94	367.19	0.030	21.3	21.38	106
06/28/05	1:00 PM	1934	84	211.02	0.125	128.0	88.38	<100
06/28/05	1:27 PM	3544	94	365.54	0.018	25.6	10.63	112
06/30/05	12:16 PM	3572	94	368.42	0.020	21.3	21.50	106
07/05/05	1:05 PM	1958	84	213.64	0.165	85.3	85.63	<100
07/07/05	12:59 PM	35.14	94	3.62	0.020	28.4	15.25	106
07/12/05	12:59 PM	3537	94	364.81	0.020	28.4	21.63	106
07/14/05	1:00 PM	2107	84	229.89	0.160	85.3	80.25	106
07/19/05	1:02 PM	3622	94	373.58	nt			
07/26/05	1:06 PM	2699	84	294.49	0.040	85.3	89.88	<100
07/26/05	1:12 PM	3635	94	374.92	nt			
07/28/05	1:13 PM	3408	94	351.51	nt			

nt - no trigger

detect structure motions from ground vibrations as low as 0.02 ips.

The average frequencies at the peak velocities (referred to as “peak frequency”) were 44 and 41 Hertz (Hz) at the Welch and Marian Manor, respectively. These values are very high and rarely observed for limestone rock at the distances of the two study structures (up to 3534 ft from the blasting). This indicates that the limestone is very competent rock that can carry high frequencies for very long distances. With respect to cracking potential in structures, high ground motion frequencies (greater than 30 to 40 Hz) have a far lower potential to cause threshold cracking in structures than frequencies less than 10 Hz. At frequencies above 40 Hz, hairline cracking may occur with a very low probability at ground motions well above than 2 ips, or 29 times greater than the highest ground motion recorded at the Welch residence and 12 times greater than the highest ground motion recorded at the Marian Manor. Therefore, it is not possible that cracking in walls could possibly occur at these very low levels of ground motions.

Using the maximum ground motions recorded at the two structures along with the average and lowest frequencies (16 Hz and 14 Hz for Marian Manor and Welch), the worst case ground displacements may be computed, assuming that the time histories contain an underlying, nearly-sinusoidal harmonic wave. These displacements represent the "worst-case" soil deflections that could cause foundations to similarly deflect, assuming perfect coupling. The well-established relationship for displacement, D in inches, as a function of velocity, V in ips, and frequency, f in Hz, is as follows:

$$D = \frac{PPV}{2\pi f} \tag{1}$$

At the Welch residence, D may range from a minimum to maximum

$$D = \frac{PPV}{2\pi f} = \frac{0.068}{2\pi 44} = 0.00025in \qquad \text{to} \qquad D = \frac{PPV}{2\pi f} = \frac{0.068}{2\pi 14} = 0.00077in$$

which is 0.25 to 0.77 milli-inch (mills). Whereas at the Marian Manor, ground deflections range

$$D = \frac{PPV}{2\pi f} = \frac{0.165}{2\pi 41} = 0.00064in \qquad \text{to} \qquad D = \frac{PPV}{2\pi f} = \frac{0.165}{2\pi 16} = 0.00164in$$

or 0.64 to 1.64 milli-inch (mills). These ground deflections are exceptionally small and well within the elastic range of foundation soils and rock. As such, these low displacements cannot possibly lead to permanent deformations beneath foundations and hence damages within structures.

Attenuation of Ground Vibrations

The decay or attenuation of ground motions as a function of distance and varying charge weights used in blasting was analyzed and compared with other studies in limestone rock. In this manner, the geology of the Sibley quarry site may be simply evaluated with respect to expected or typical behavior and determine if directional anomalies exist at the site. The attenuation of ground vibrations in terms of the peak particle velocity (PPV) is plotted in Figure 6 as recorded at the exterior seismographs at each structure (one to the north and one to the west of the quarry). The decrease in PPV values with distance scaled to the square root of the maximum charge weight per delay (scaled distance, SD) appears to follow a trend line indicating an apparent absence of geologic anomalies. However, this trend is based on a very narrow range of charge weights used in the blast design. Additionally, only 23 data points recorded at both structures combined were used to establish this attenuation plot and this number is below the number recommended for reliable statistical correlation.. A minimum of 30 data points from each location are required to determine data correlation. Therefore, it was difficult to completely evaluate if the Sibley Quarry data is typical and representative of limestone rock with this limited data.

In order to evaluate representation, the Sibley data were compared with attenuation data from "typical" limestone quarries located elsewhere in the U.S. to determine if the Sibley quarry data fell within expected attenuation limits. This comparison is shown in Figure 7 where the Sibley data is shown in blue. The comparison quarry data are identified as "competent limestone" and "weathered limestone" (the two extremes is blasting difficulty based on rock hardness). The comparison blasts were deigned with a wide range of scaled distance values and represent a large number of data. In addition, very close-in seismographs were used to characterize the full range of peak ground motion attenuation at SD values less than 40. These close-in seismographs were not placed at structures but within the quarry permit property and between the quarry and structures. This full range of SD values define trend lines with a high degree of confidence as indicated with the R^2 correlation coefficient.

The Sibley limestone is considered to be competent. Therefore, this data was evaluated in terms of the "competent limestone" data (black symbols) spread or scatter about the 50-percentile best fit (solid black line) through the data. the upper 100-percentile (100%-tile) and lower 0-percentile lines are shown as black dashed lines and represent the highest and lowest possible range of typical ground vibration data. No data fell outside the dashed line. The dashed lines were extended

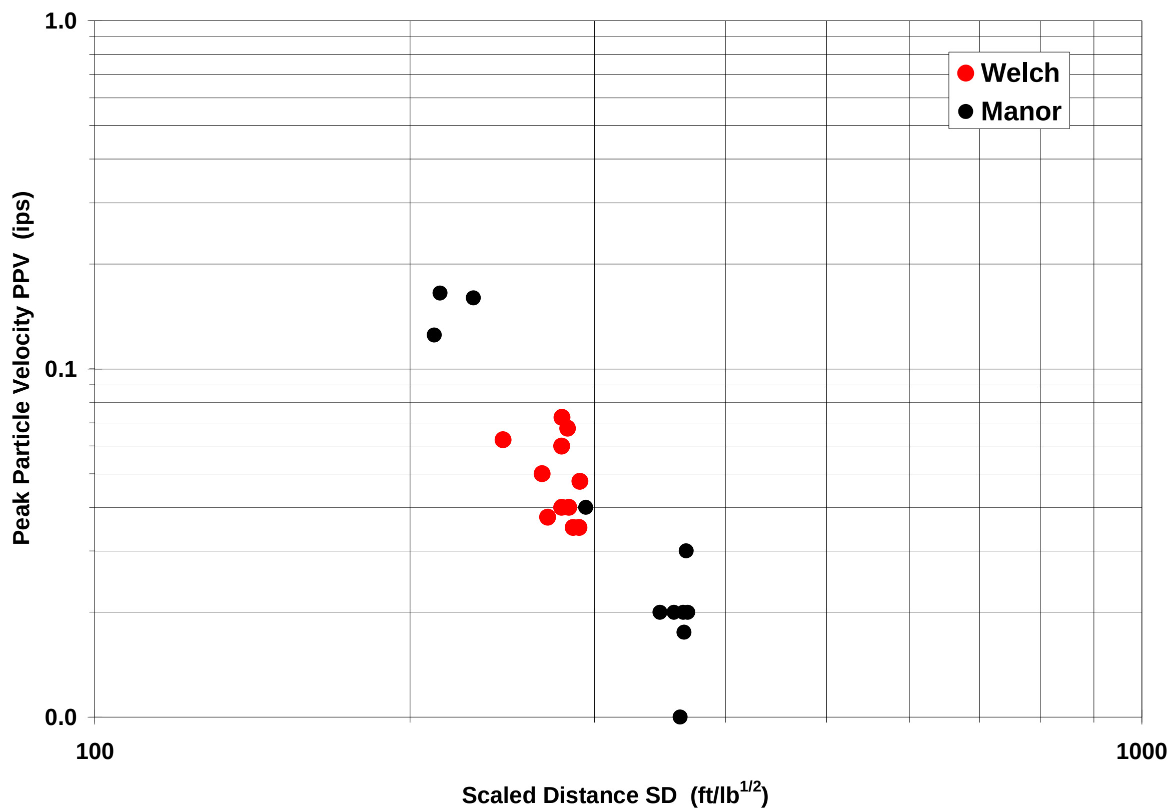


Figure 6 Ground motion attenuation for quarry production blasts during the study

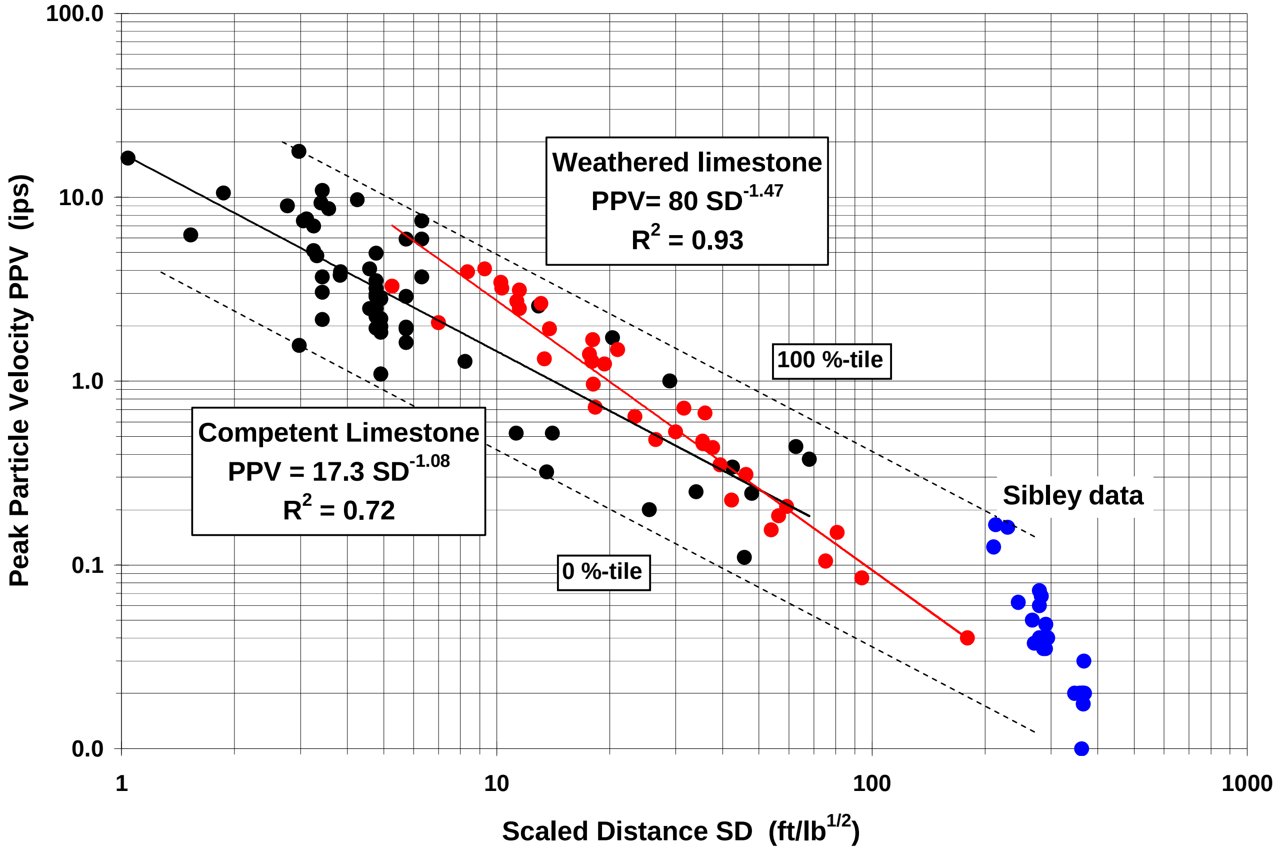


Figure 7 Ground motion attenuation for three limestone quarries (Sibley data is shown in blue)

to include and bracket the Sibley quarry data at higher SD values. Clearly this shows that the Sibley data fell within the expected scatter for typical competent limestone ground motion attenuation. Therefore, it can be concluded that the Sibley ground motion data recorded at the test structures are normal and expected. The data represent typical ground motions for competent limestone and do not exhibit geological anomalies.

Ground Vibration and Frequency

Figure 8 is a plot of the peak particle velocities (PPV) and frequency of ground motion at the PPV at the two structures. The safe blasting criteria recommended by the U.S. Bureau of Mines (Siskind, et al., 1980) is included as an upper bound line for threshold cracking in drywall (the weakest structure component in dwellings). The safe criterion is based on over 40 years of research and crack observations that have never been scientifically challenged to date. Lower bounds for crack observations in brick mortar and concrete slabs are also included to show the lowest possible vibration limits at which cracking starts in those materials .

The peak ground velocities recorded at the study structures are far below the upper bounds and will within the safe region below the threshold cracking bounds. As such, it is not possible for these levels of ground motions and frequencies to cause hairline or threshold cracking in structures. In addition cracking in mortar or foundation slab cracking are not possible at these very low levels of ground vibrations.

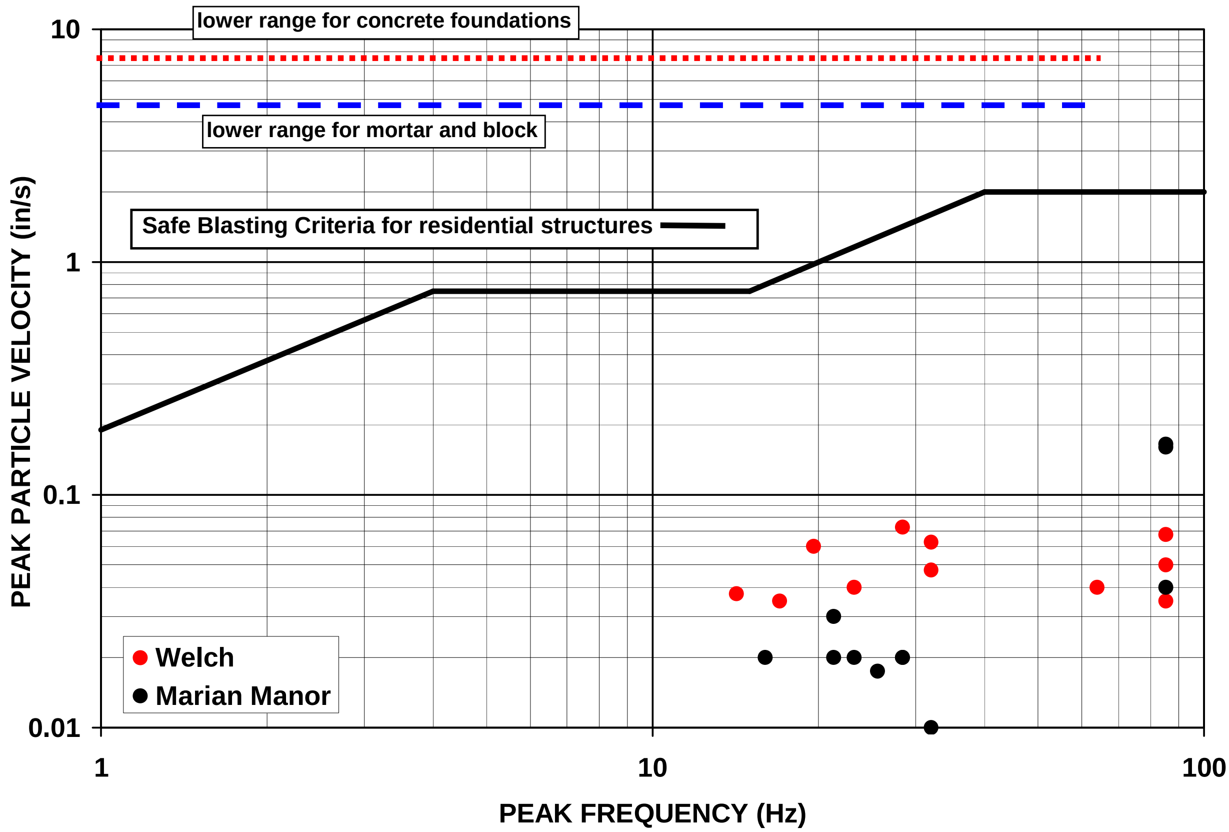


Figure 8 Peak particle velocity versus frequency at the peak velocity

Structure Response to Blasting

Seismograph reports for structures response for all blasting events are given in Appendices A and B for the Welch and Marian Manor structures. Summary tables appear at the beginning of each appendix. The data are presented by blast date and time and include peak velocity values and frequencies for the three components of ground motion, lower (S1) corner, upper (S2) corner, and horizontal motion at the mid-wall.

Tables 3 and 4 summarize the ground motion and airblast data in comparison with structure response for the southwest and southeast walls at the Welch residence and west and south walls at the Marina Manor, respectively. The peak velocities recorded by the sensors at corners and mid-walls are given and compared with ground motion and airblast data for similar horizontal components (or compass directions).

Comparison of Structure Response with Ground Motions and Airblast

It is often useful to visually compare the horizontal motions of a structure (upper and lower corners and mid-wall) with horizontal components of ground motion and airblast excitations driving structure motions. Representative velocity time histories for the blasts generating the greatest peak

Table 3 (a) Summary of structure response for the southwest wall at the Welch residence

Shot Date	Shot Time	GROUND MOTION AND AIRBLAST				S1- WALL BASE			S2- WALL TOP		
		Transverse Peak Velocity	Peak Frequency	FFT Frequency	Airblast	Peak Corner Velocity	Peak Frequency	FFT Frequency	Peak Corner Velocity	Peak Frequency	FFT Frequency
		(in/sec)	(Hz)	(Hz)	(psi)	(in/sec)	(Hz)	(Hz)	(in/sec)	(Hz)	(Hz)
06/16/05	1:03 PM	no trigger									
06/21/05	12:59 PM	no trigger									
06/21/05	1:12 PM	0.038	14.2	21.25	0.0006	0.035	18.2	5.13	0.030	11.1	5.13
06/23/05	1:00 PM	0.045	36.5	21.50	0.0006	0.025	19.6	21.25	0.025	17.0	9.50
06/28/05	1:00 PM	0.020	42.6	21.88	0.0003	0.015	28.4	13.38	0.010	0.0	10.13
06/28/05	1:27 PM	no trigger									
06/30/05	12:16 PM	0.035	17.0	22.13	0.0006	0.015	15.0	7.88	0.015	13.4	7.88
07/05/05	1:05 PM	0.035	19.6	42.50	0.0003	0.015	42.6	12.63	0.015	28.4	8.50
07/07/05	12:59 PM	no trigger									
07/12/05	12:59 PM	0.040	23.2	22.00	0.0006	0.020	21.3	5.13	0.030	13.4	7.13
07/14/05	1:00 PM	0.038	51.2	42.63	0.0003	0.020	51.2	8.25	0.015	19.6	8.25
07/19/05	1:02 PM	0.040	64.0	20.75	0.0003	0.025	11.1	9.13	0.025	10.2	9.13
07/26/05	1:06 PM	0.055	32.0	13.13	0.0003	0.035	12.1	6.00	0.025	8.5	6.00
07/26/05	1:12 PM	0.073	28.4	5.25	0.0003	0.040	18.2	5.25	0.045	12.1	5.25
07/28/05	1:13 PM	0.060	19.6	19.00	0.0003	0.025	17.0	8.50	0.030	15.0	8.50

Table 3 (b) Summary of structure response for the southeast wall at the Welch residence

Shot Date	Shot Time	GROUND MOTION AND AIRBLAST: Radial Peak Velocity (in/sec)	GROUND MOTION AND AIRBLAST: Peak Frequency (Hz)	GROUND MOTION AND AIRBLAST: FFT Frequency (Hz)	GROUND MOTION AND AIRBLAST: Airblast (psi)	S1- WALL BASE: Peak Corner Velocity (in/sec)	S1- WALL BASE: Peak Frequency (Hz)	S1- WALL BASE: FFT Frequency (Hz)	S2- WALL TOP: Peak Corner Velocity (in/sec)	S2- WALL TOP: Peak Frequency (Hz)	S2- WALL TOP: FFT Frequency (Hz)	WEST MID-WALL: Peak Wall Velocity (in/sec)	WEST MID-WALL: Peak Frequency (Hz)	WEST MID-WALL: FFT Frequency (Hz)
06/16/05	1:03 PM	no trigger												
06/21/05	12:59 PM	no trigger												
06/21/05	1:12 PM	0.038	19.6	21.00	0.0006	0.025	18.2	7.13	0.025	7.1	7.13	0.060	19.6	20.75
06/23/05	1:00 PM	0.048	32.0	22.13	0.0006	0.020	28.4	12.00	0.020	14.2	7.38	0.050	21.3	21.25
06/28/05	1:00 PM	0.033	28.4	14.13	0.0003	0.015	21.3	13.00	0.015	19.6	13.50	0.030	28.4	27.13
06/28/05	1:27 PM	no trigger												
06/30/05	12:16 PM	0.035	23.2	22.25	0.0006	0.030	21.3	12.38	0.025	8.5	7.13	0.040	19.6	21.88
07/05/05	1:05 PM	0.065	21.3	14.25	0.0003	0.015	16.0	12.75	0.020	9.8	7.13	0.035	21.3	12.75
07/07/05	12:59 PM	no trigger												
07/12/05	12:59 PM	0.035	32.0	22.38	0.0006	0.025	17.0	7.25	0.025	14.2	7.25	0.055	14.2	15.38
07/14/05	1:00 PM	0.048	21.3	22.50	0.0003	0.025	23.2	14.38	0.030	9.8	5.50	0.040	28.4	5.50
07/19/05	1:02 PM	0.038	23.2	22.38	0.0003	0.035	19.6	12.38	0.020	28.4	8.50	0.045	25.6	9.13
07/26/05	1:06 PM	0.063	32.0	5.50	0.0003	0.050	10.6	12.25	0.035	8.2	5.63	0.045	17.0	6.13
07/26/05	1:12 PM	0.065	32.0	24.50	0.0003	0.050	18.2	12.13	0.035	10.2	5.75	0.060	16.0	5.25
07/28/05	1:13 PM	0.048	25.6	22.00	0.0003	0.035	15.0	12.25	0.025	18.2	8.75	0.045	14.2	8.50

Table 4 (a) Summary of structure response for the west wall at the Marian Manor

Shot Date	Shot Time	GROUND MOTION AND AIRBLAST				S1- WALL BASE			S2- WALL TOP			WEST MID-WALL		
		Radial Peak Velocity	Peak Frequency	FFT Frequency	Airblast	Peak Corner Velocity	Peak Frequency	FFT Frequency	Peak Corner Velocity	Peak Frequency	FFT Frequency	Peak Wall Velocity	Peak Frequency	FFT Frequency
		(in/sec)	(Hz)	(Hz)	(psi)	(in/sec)	(Hz)	(Hz)	(in/sec)	(Hz)	(Hz)	(in/sec)	(Hz)	(Hz)
06/16/05	1:03 PM	0.015	23.2	15.13	<0.0003	0.010	0.0	15.13	0.025	25.6	15.13	0.080	36.5	29.63
06/21/05	12:59 PM	0.008	32.0	18.13	0.0006	0.005	0.0	18.25	0.015	36.5	18.38	0.035	32.0	30.50
06/21/05	1:12 PM	0.013	23.2	20.88	0.0006	0.010	0.0	26.88	0.025	23.2	21.00	0.040	28.4	29.50
06/23/05	1:00 PM	0.013	32.0	22.00	0.0006	0.010	0.0	22.38	0.025	25.6	22.50	0.050	36.5	29.88
06/28/05	1:00 PM	0.035	51.2	11.25	<0.0003	0.025	32.0	11.25	0.045	23.2	19.00	0.100	42.6	30.50
06/28/05	1:27 PM	0.010	28.4	11.50	0.0012	0.010	0.0	11.75	0.030	25.6	21.75	0.050	25.6	27.13
06/30/05	12:16 PM	0.010	51.2	21.50	0.0006	0.010	0.0	21.75	0.025	25.6	22.50	0.050	32.0	21.50
07/05/05	1:05 PM	0.065	64.0	98.50	<0.0003	0.025	64.0	35.13	0.035	64.0	35.00	0.100	51.2	14.38
07/07/05	12:59 PM	0.013	21.3	15.25	0.0006	0.015	21.3	15.25	0.015	28.4	27.75	0.050	25.6	29.38
07/12/05	12:59 PM	0.018	36.5	46.00	0.0006	0.010	0.0	16.38	0.015	42.6	26.13	0.045	42.6	21.88
07/14/05	1:00 PM	0.043	85.3	4.25	0.0006	0.020	64.0	18.38	nd			0.075	51.2	30.63
07/19/05	1:02 PM	nd				0.010	0.0	42.75	0.020	32.0	21.00	0.045	28.4	20.88
07/26/05	1:06 PM	0.025	36.5	11.25	<0.0003	0.020	28.4	11.50	0.035	28.4	13.88	0.065	36.5	30.50
07/26/05	1:12 PM	nd				0.015	28.4	4.38	0.025	23.2	14.63	0.080	28.4	30.13
07/28/05	1:13 PM	nd				0.005	0.0	15.50	0.015	28.4	10.00	0.040	32.0	29.50

nd - no data

Table 4 (b) Summary of structure response for the south wall at the Marian Manor

Shot Date	Shot Time	GROUND MOTION AND AIRBLAST				S1- WALL BASE			S2- WALL TOP			SOUTH MID-WALL		
		Transverse Peak Velocity	Peak Frequency	FFT Frequency	Airblast	Peak Corner Velocity	Peak Frequency	FFT Frequency	Peak Corner Velocity	Peak Frequency	FFT Frequency	Peak Wall Velocity	Peak Frequency	FFT Frequency
		(in/sec)	(Hz)	(Hz)	(psi)	(in/sec)	(Hz)	(Hz)	(in/sec)	(Hz)	(Hz)	(in/sec)	(Hz)	(Hz)
06/16/05	1:03 PM	0.020	23.2	18.75	<0.0003	0.020	28.4	15.38	0.030	23.2	15.38	0.030	25.6	15.25
06/21/05	12:59 PM	0.010	32.0	18.75	0.0006	0.005	0.0	19.63	0.005	0.0	20.38	0.025	23.2	20.75
06/21/05	1:12 PM	0.020	16.0	21.38	0.0006	0.010	0.0	13.13	0.015	25.6	20.88	0.045	21.3	20.75
06/23/05	1:00 PM	0.030	21.3	21.38	0.0006	0.020	28.4	21.00	0.015	23.2	21.38	0.045	25.6	22
06/28/05	1:00 PM	0.043	23.2	23.13	<0.0003	0.025	25.6	23.38	0.035	18.2	18.75	0.050	25.6	20.13
06/28/05	1:27 PM	0.018	25.6	10.63	0.0012	0.010	0.0	10.25	0.025	19.6	10.25	0.035	18.2	20.38
06/30/05	12:16 PM	0.020	21.3	21.50	0.0006	0.010	0.0	22.88	0.015	36.5	21.63	0.030	23.2	21.88
07/05/05	1:05 PM	0.043	42.6	14.13	<0.0003	0.030	21.3	14.25	0.035	17.0	14.13	0.070	21.3	21.13
07/07/05	12:59 PM	0.015	21.3	13.75	0.0006	0.015	25.6	13.63	0.020	18.2	15.63	0.035	19.6	20.88
07/12/05	12:59 PM	0.020	28.4	21.63	0.0006	0.015	28.4	22.88	0.015	36.5	22.00	0.045	32	22.25
07/14/05	1:00 PM	0.040	85.3	4.13	0.0006	0.020	32.0	4.13	0.030	28.4	4.13	0.045	19.6	20.38
07/19/05	1:02 PM	nd				0.015	32.0	20.50	0.020	21.3	20.88	0.035	23.2	21.13
07/26/05	1:06 PM	0.028	32.0	27.50	<0.0003	0.020	23.2	27.50	0.015	18.2	18.88	0.035	17	11.25
07/26/05	1:12 PM	nd				0.015	28.4	14.88	0.020	23.2	19.88	0.04	18.2	10.13
07/28/05	1:13 PM	nd				0.010	0.0	16.63	0.015	25.6	16.75	0.025	19.6	10.13

nd - no data

ground motion were selected for the two structures and illustrated herein. For the Welch residence, both blasts on July 26 produced the largest overall responses in the upper structure and mid-walls. For the Marian Manor, the blast on June 16 was selected as this was the only blast for which crack response was measured (as explained later)

Figures 9 through 11 show typical time histories comparing ground motions and airblast with structure motions for the two structures. For the Welch residence, the two blasts on 7/26/05 are shown as these blasts generated the highest ground vibration during the study and therefore represent the “worst case” blasts. Furthermore, the blast at 1:12 pm generated low frequencies in the ground that tended to be picked up in the structure. The blast on 6/16/05 was used for the Marian Manor as this was the only blast with crack response data.

The Marian Manor data in Figure 11 show that the lower corner (S1) vibration time history compares closely with the ground velocities for the radial (south) and transverse (west) components, indicating good coupling of the structures with the foundation. The difference between S1 and S2 amplitudes shows the influence of ground motion on the upper corner response (S2) and there is small amplification of the ground motions in the upper structure. As expected, the mid-wall response on the south, stiffer wall is far lower in amplitude than the mid-wall response on the west wall. The crack motions are driven by the south wall displacements as shown by comparing the S2 and crack time histories. This is because the wall perpendicular to the wall containing the crack always drives the crack. The influence of airblast is generally small at the Marian Manor.

In the Welch residence, the southwest wall crack response is clearly influenced by the southeast wall or the wall perpendicular to the crack wall during both blasts on 7/26/05 as shown in Figures 9 (a) and 10 (a). The lower corner motion (S1) shows a better correlation with crack peaks than the upper corner (S2) motion. This is perhaps because the crack is located at the base of the front window and directly in line with the lower rather than the upper corner. As such, when the south corner base moves into the room, the crack closes (negative peak) and as the base moves outward, the crack opens (positive peak) as expected.

It is shown that the lower and upper corners move in phase when considering each wall independently. However the lower corner (S1) shows a slighter larger displacement than the upper structure (S2) in the southeast wall whereas in the southwest wall the opposite is true. This is indicative of a slight torsion effect that takes place within the gross structure. This can be shown in Figure 12 where the upper and lower corners of opposing walls are plotted together for the two blasts on 7/26/05. Both blasts generated upper corner responses S2 in the two walls that are out of phase (e.g., one wall moving in toward the house while the opposing wall moves out). For the blast at 1:12 pm, both corner responses (S1 and S2) were out of phase. Such behavior often occurs when ground motion predominant frequencies (FFT frequencies) are at or below the natural frequency of structures. The FFT frequencies for the component containing the PPV for the blasts were 5.5 Hz and 5.25 Hz as shown in Table 1. At these frequencies, the upper structure motions may be somewhat larger than the ground motions and respond at corresponding ground motion frequencies. Walls of different dimensions located at right angles may exhibit out of phase motions. However, for both structures, the deflection amplitudes are well within the elastic range of the construction materials. All materials respond elastically, regardless of frequencies of motion and no material cracking is possible at the low levels of blast vibration recorded during this study..

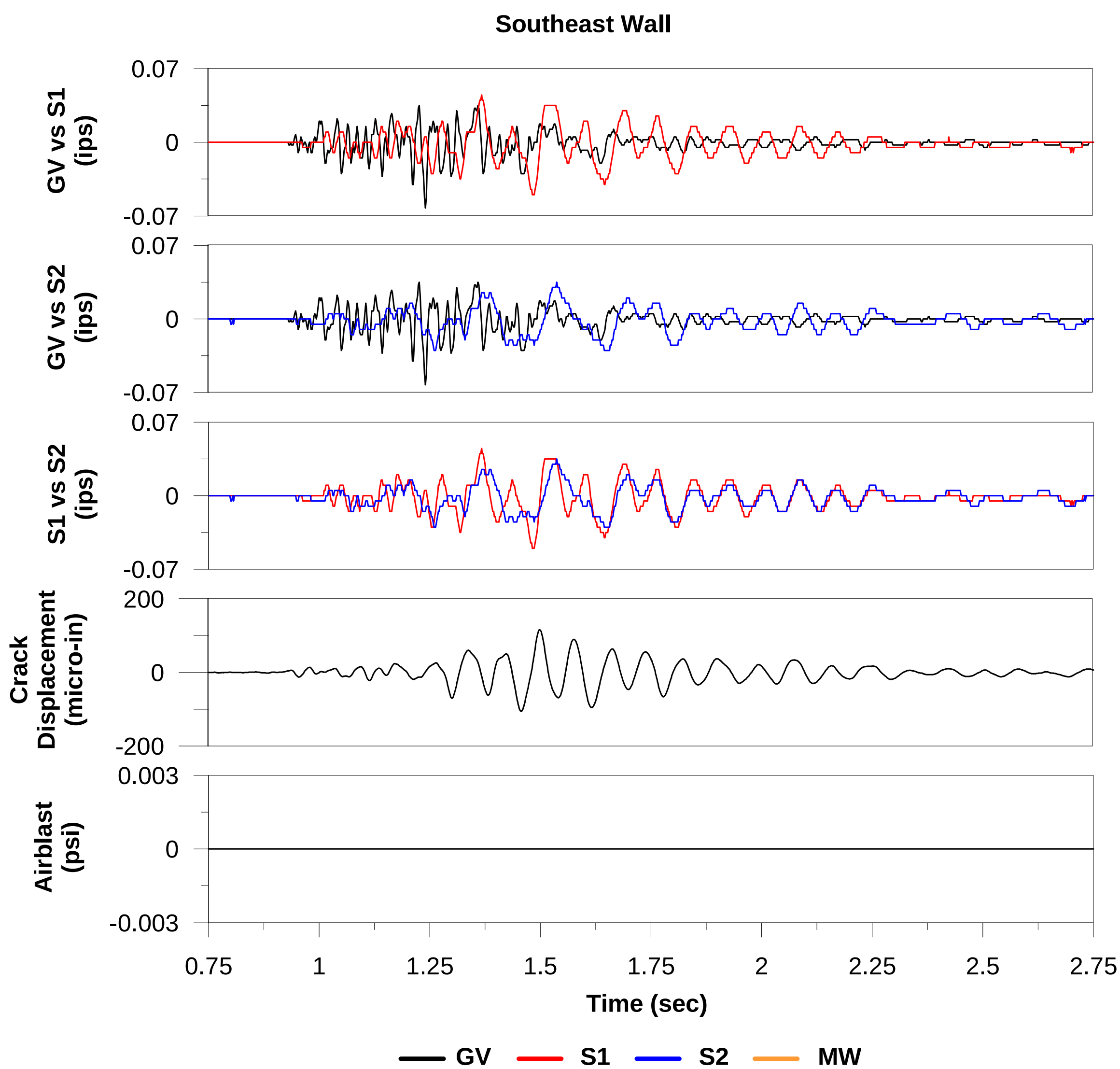


Figure 9 (a) Time history comparisons of radial ground velocity (GV), southeast wall lower (S1) and upper (S2) corners, southwest wall crack displacement and airblast for the Welch residence, blast on 7/26/05 at 1:06 pm
(note – motion in the southeast wall affects crack motions in the southwest wall)

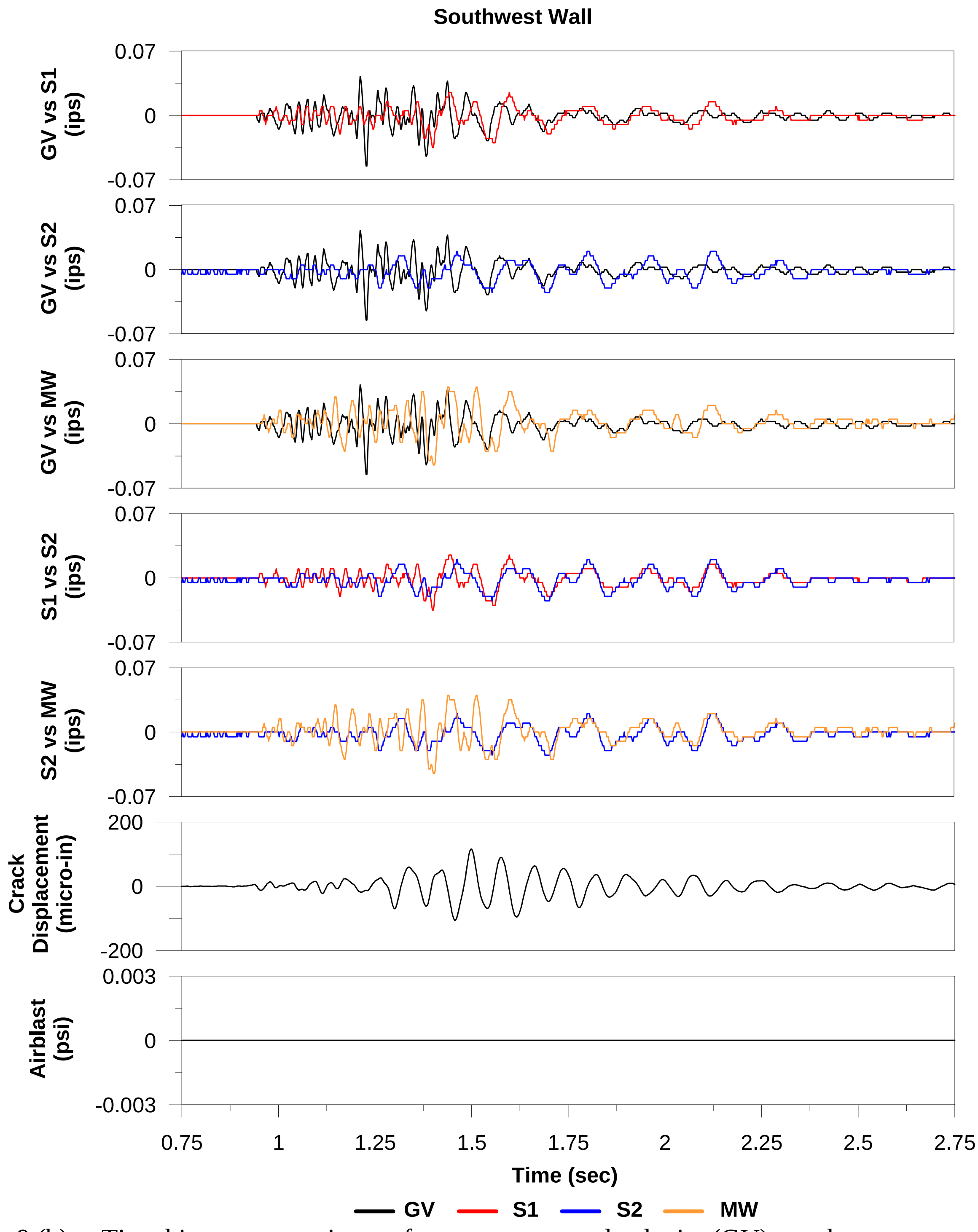


Figure 9 (b) Time history comparisons of transverse ground velocity (GV), southwest wall lower (S1) corner, upper (S2) corner, and mid-wall (MW), southwest wall crack displacement and airblast for the Welch residence, blast on 7/26/05 at 1:06 pm

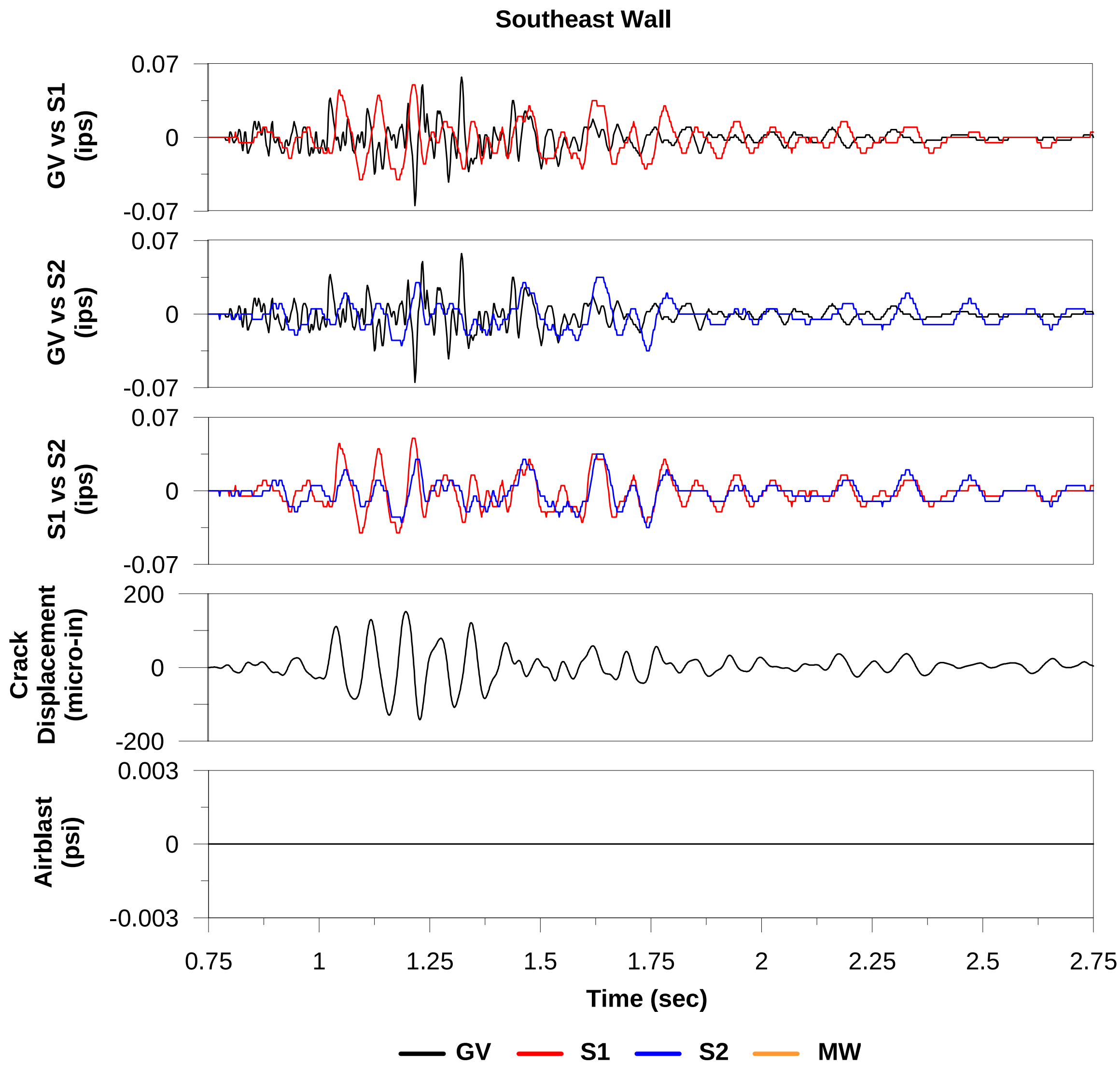


Figure 10 (a) Time history comparisons of radial ground velocity (GV), southeast wall lower (S1) and upper (S2) corners, southwest wall crack displacement and airblast for the Welch residence, blast on 7/26/05 at 1:12 pm (note – motion in the southeast wall affects crack motions in the southwest wall)

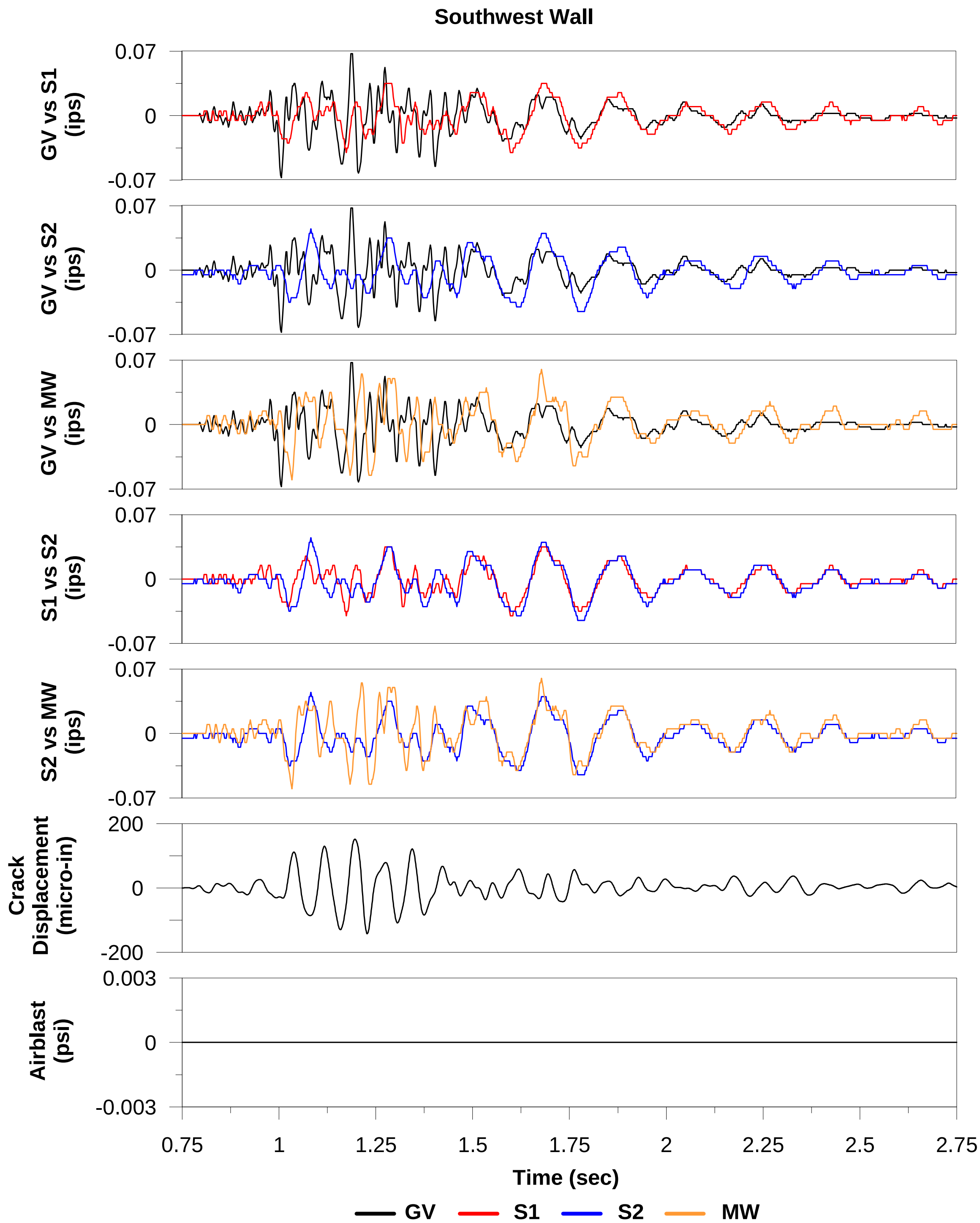


Figure 10 (b) Time history comparisons of transverse ground velocity (GV), southwest wall lower (S1) corner, upper (S2) corner, and mid-wall (MW), southwest wall crack displacement and airblast for the Welch residence, blast on 7/26/05 at 1:12 pm

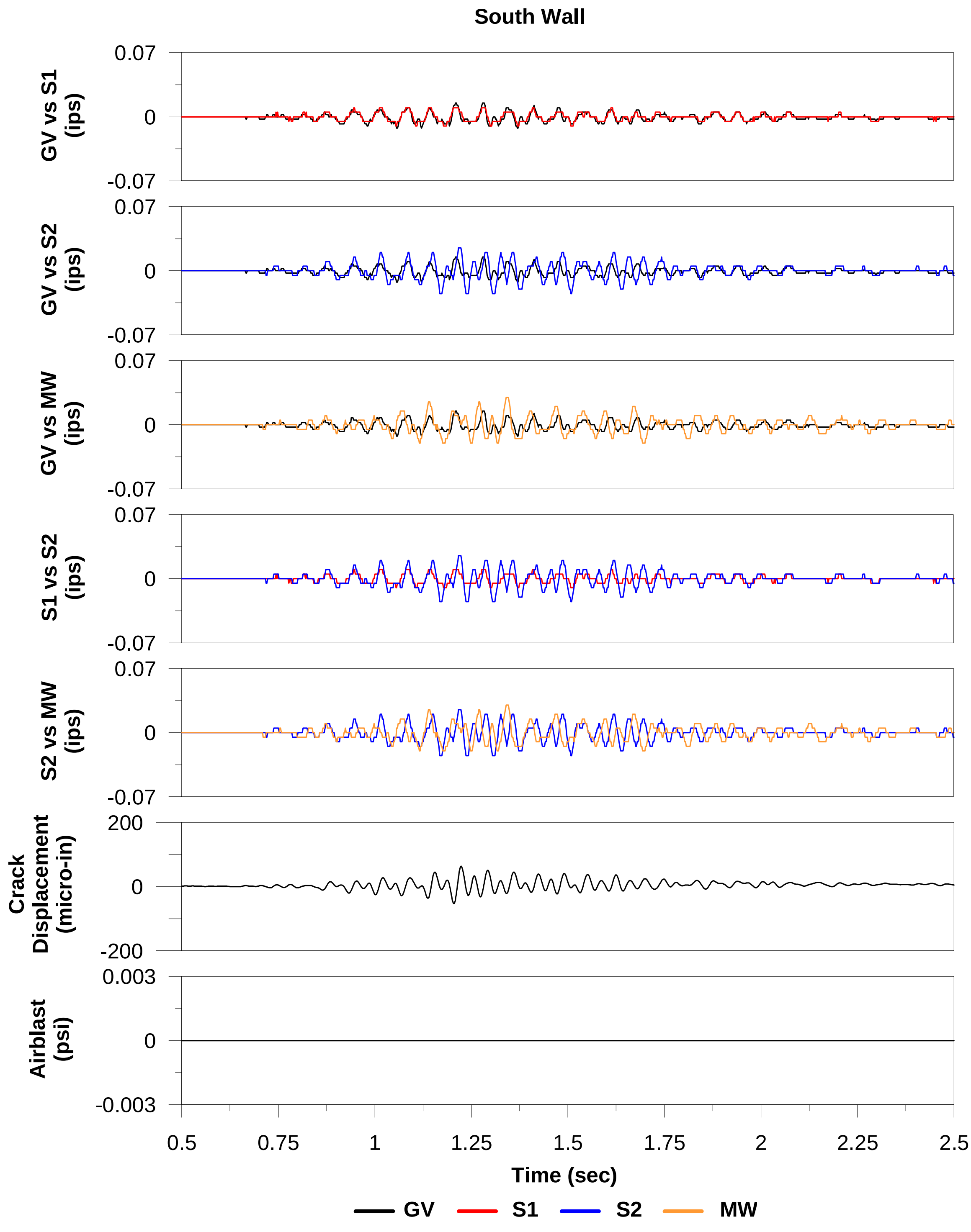


Figure 11 (a) Time histories comparison of radial ground velocity (GV), lower (S1) corner, upper (S2) corners, south mid-wall (MW), and airblast for the Marian Manor, blast on 6/16/05 at 1:03 pm

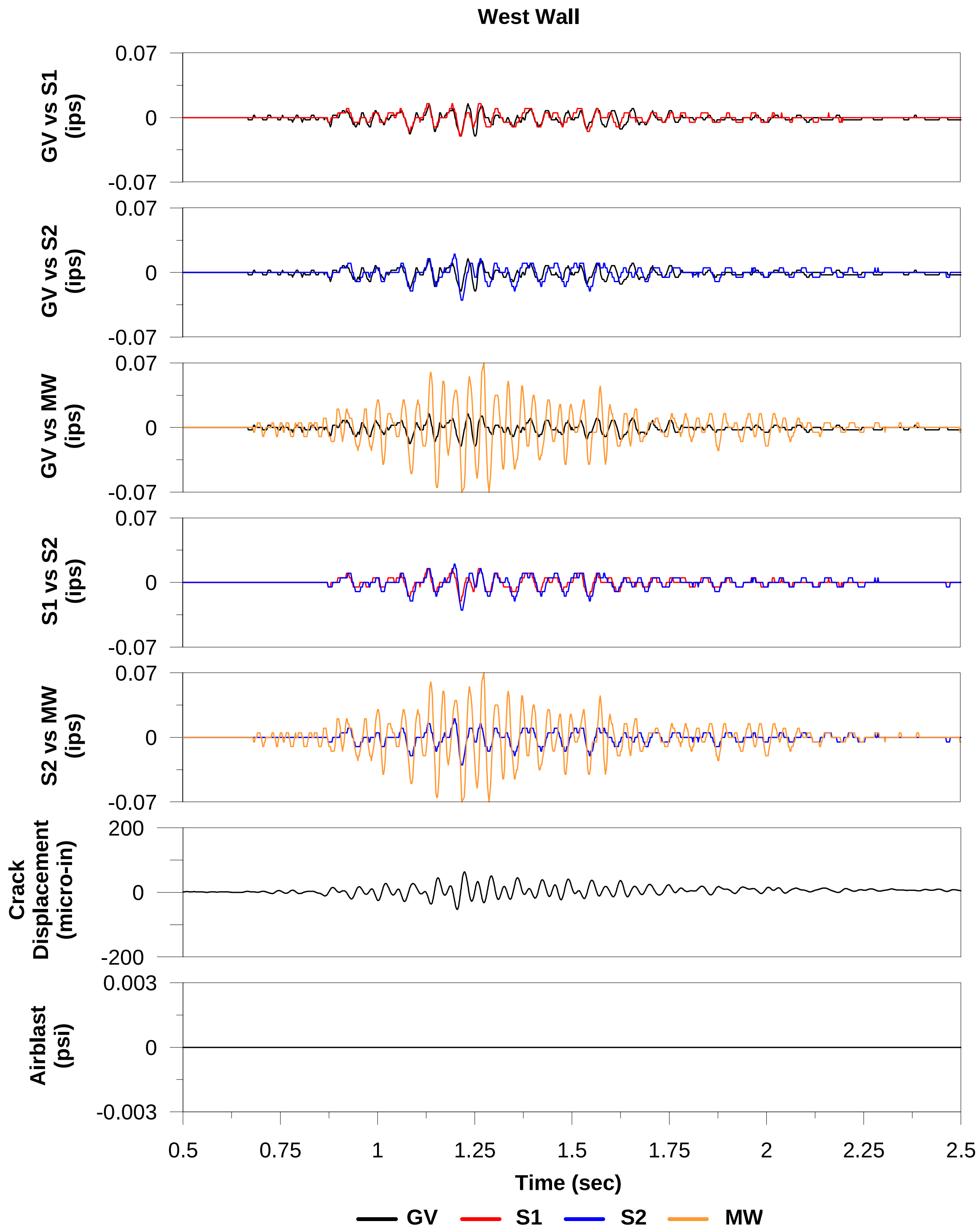
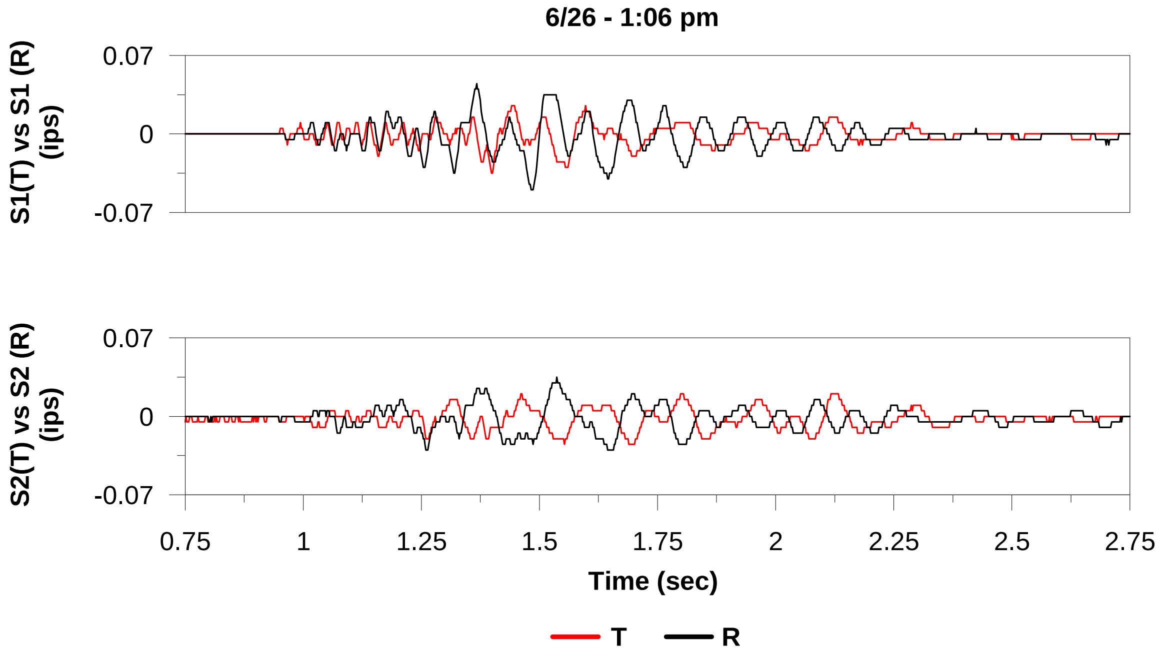


Figure 11 (b) Time histories comparison of transverse ground velocity (GV), lower (S1) corner, upper (S2) corners, west mid-wall (MW), and airblast for Marian Manor, blast on 6/16/05 at 1:03 pm



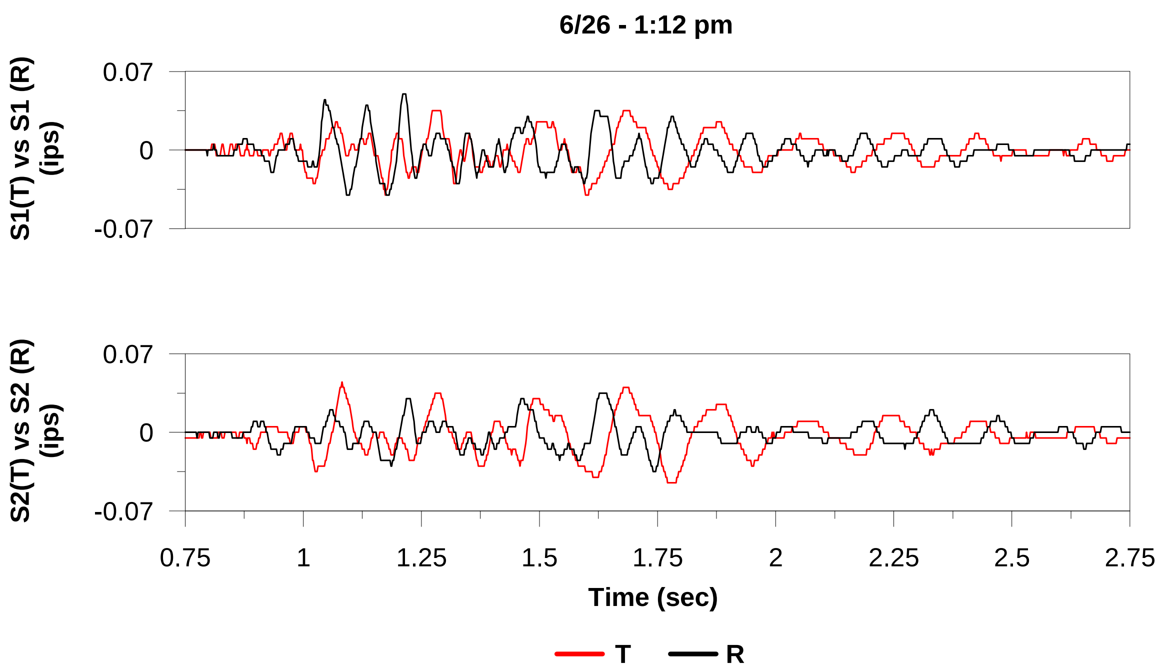


Figure 12 Welch comparisons of upper and lower corner responses comparing two different walls

Natural Frequency and Damping Ratio

In a structure response study, it is important to compute natural frequency and damping to determine if these dynamic parameters fall within the normal range of values for all structure types. As such, this ensures that safe blasting criteria developed for typical one-and two-story structures apply in a general sense to all structure types.

Natural frequency is the frequency at which structures oscillate freely after excitation energy is removed, or ground motions arrest. If the blasting ground vibrations arrive at a structure carrying a predominant frequency component identical to or less than the natural frequency of the structure, a portion of the blast wave energy above a certain threshold will readily transmit into the structure and start the structure in motion. The natural frequency match will cause the structure to continue to vibrate for a longer time compared with the ground motion. Often very long duration shaking will cause residents to notice the blast and become fearful that damage may be occurring within the structure. However, damage can only occur if the amplitude of shaking is high, resulting in wall strains that may promote cracking (e.g., at strains greater then the failure strains of the materials).

Fundamental frequencies of most residential structures range from 4 Hz to 12 Hz. Keeping the ground motion frequencies above this range may help minimize the sensation that the structure is being harmed by long duration vibrations when, in fact, the amplitudes are far below those that could cause cracking.

The natural frequency of a structure can be evaluated during upper structure “free-response” when upper structure shaking continues after ground motions have arrested. The free-response method identifies that portion of the upper structure (S2) time history corresponding with zero ground motions after passage of the blast wave, where the upper structure response begins to decay slowly to zero. The frequency of this trailing response is often assumed to represent the natural frequency of the structure if the signal-to-noise ratio is fairly high and frequencies are uniform.

During this decay portion, the structure damping coefficient can be computed. Damping is a natural phenomenon that occurs in all materials when subjected to an impulse force. Structure motions from excitations (ground velocities) are naturally attenuated during energy dissipation and eventually come to rest. The percentage of critical damping, β, is a measure of structure rigidity and how fast the energy of excitation decays in the structure. Damping is calculated using two successive peaks during the free response decay portion, P1 and P2.

The percentage of critical damping is calculated as follows:

$$\beta = \left(\frac{1}{2\pi}\right)\ln\left(\frac{P_1}{P_2}\right) \tag{2}$$

where

β = percentage of critical damping (%)
P_1 = amplitude of the first peak (ips)
P_2 = amplitude of the next successive peak (ips)

To compute a structure’s natural frequency and damping, sufficient energy from ground motions or airblast is required. Previous studies by Aimone, et al. (2003) have shown that ground motion energy above 0.3 ips or airblast levels above 117 dB, both at predominate frequencies near the structure’s natural frequency, are required. Blasting over the time period of this study did not provide sufficient energy in either the ground or in the air to compute damping and natural frequency in the stiff, one-story Marian Manor because ground vibrations were very low (0.16 ips or less) and airblast was at the lowest detection limits of the transducers. Only the minimum (threshold) energy

Table 5 Summary of natural frequencies computed for the Welch residence

Shot Date	Time	SOUTHEAST WALL (R) P1 (ips)	SOUTHEAST WALL (R) P2 (ips)	SOUTHEAST WALL (R) Natural Frequency (Hz)	SOUTHEAST WALL (R) Damping (%)	SOUTHWEST WALL (T) P1 (ips)	SOUTHWEST WALL (T) P2 (ips)	SOUTHWEST WALL (T) Natural Frequency (Hz)	SOUTHWEST WALL (T) Damping (%)
06/16/05	1:03 PM	nt				nt			
06/21/05	12:59 PM	nt				nt			
06/21/05	1:12 PM	n/a		8.0		n/a			
06/23/05	1:00 PM	n/a				n/a		10.0	
06/28/05	1:00 PM	n/a				n/a			
06/28/05	1:27 PM	nt				nt			
06/30/05	12:16 PM	n/a		7.0		n/a		9.0	
07/05/05	1:05 PM	n/a				n/a			
07/07/05	12:59 PM	nt				nt			
07/12/05	12:59 PM	n/a		8.0		n/a		8.0	
07/14/05	1:00 PM	n/a				n/a			
07/19/05	1:02 PM	n/a		9.0		n/a			
07/26/05	1:06 PM	n/a				n/a			
07/26/05	1:12 PM	n/a				n/a			
07/28/05	1:13 PM	n/a				n/a			
AVERAGE				**8.0**	**np**			**9.0**	**np**

nt - no trigger
n/a - not applicable
nd - no data
np - not possible

required to start the southeast and southwest walls of the two-story Welch structure moving at the natural frequency was observed for only a few blasts as shown in Table 5. It was not possible to compute damping as the structure motion signal-to-noise ratios were too low to accurately define a strong non-linear decay in peaks motion amplitudes.

Shown in Table 5, the natural frequency for the Welch residence averaged 9 Hz and 8 Hz in the southeast and southwest wall, respectively. These values are typical of wood-frame structures and well within the range of expected values

Upper structure amplification of ground velocities

Amplification is a comparative measure of the maximum structure response to ground vibration at the same point in time in terms of velocity. It is similar to the term "dynamic amplification factor" used by seismologists to describe the effects of earthquakes on structures. Amplification occurs when motion at S2 becomes measurably larger than the motion at GV. Amplification factor (AF) was defined for blasting vibrations by the U.S. Bureau of Mines (Siskind, et al., 1980) as the ratio of the peak upper structure velocity ($S2_{peak}$) divided by the preceding ground velocity (GV) of the same phase, positive or negative, that most likely drove the structure, or

$$AF = \left(\frac{S2_{peak}}{GV} \right) \quad (3)$$

AF was originally used by the U.S. Bureau of Mines as an arbitrary indicator of cracking potential in structures. It was hypothesized that if a structure's AF far exceeded the normal range for residential structures, cracking in walls may occur. Such a hypothesis has never been confirmed and there has been no correlation of observed cracking with specific high values of AF factors.

It was determined by Siskind (1980) and Aimone-Martin, et al., (2003) that typical one- and two-story residential structures will respond to blasting with AF values ranging from less than 1 for very stiff structures, to 4, averaging 2 to 4. Within this "normal" range, cracking potential in structures from blasting vibrations is not of concern. However, there has been no direct correlation with crack observations have been reported for AF in excess of 5 that have typically been measured in 2-story and taller structures.

To calculate AF, the time-correlated waveforms for the ground (GV) and the upper structure corner (S2) for the same horizontal component were compared. For each structure, only one blast produced data with sufficient signal-to-noise ratio to allow AF to be computed. These blasts were on 7/26/05 at 1:12 pm and 6/28/05 at 1:00 pm for the Welch residence and Marian Manor, respectively. AF values were computed to be 1.6 and 1.9 for the Welch and Marian Manor structures. The amplification factors fall at the lower end of the average range established by the U.S. Bureau of Mines and others for wood framed and concrete brick dwellings. Therefore, such amplification factors will not contribute to wall cracking at ground vibration levels below 0.75 ips peak ground motion as verified by the U.S. Bureau of Mines criteria.

Strains Calculated for Structure Walls

The magnitude of induced strains in structure components ultimately determines the likelihood of cosmetic cracking in residences. Global, or whole structure, shear strains leading to in-plane tensile wall strains and mid-wall bending strains arise from corner distortions and wall flexure as illustrated in Figure 13. It is the practice to use gypsum core drywall board material as the representative construction material that would first show signs of distress when structures are subjected to high dynamic blasting strains. This "threshold" material is thus associated with "threshold" cracking potential. Therefore, wall strains are computed for the largest blasts (e.g., the blast that generated the highest horizontal ground motions or a blast with a significant airblast

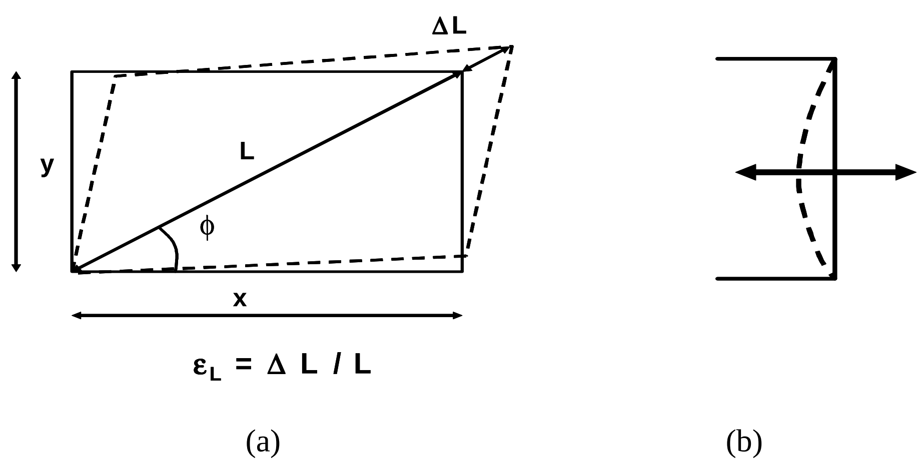


Figure 13 In-plane tensile strain (a) and out of plane mid-wall bending (b)

affecting wall flexure) and the strains are compared with failure strain levels required to crack existing “threshold” wall materials.

Global shear strain is determined by the following:

$$\gamma_{\max} = \left(\frac{\delta_{\max}}{L} \right) \tag{4}$$

where

γ_{max} = global shear strain (micro-strains or 10^{-6})
δ_{max} = maximum differential displacement, S2 – S1 (inch)
L = height of the wall subjected to strain (inch)

In-plane tensile strains, important in the assessment of wall cracking potential, are a function of the wall dimensions. The maximum tensile strain, ε_{Lmax}, is calculated from global shear strain by the equation:

$$\varepsilon_{L\max} = \gamma_{\max}(\sin\theta)(\cos\theta) \tag{5}$$

where θ is the interior angle of the longest diagonal of the wall subjected to strain with reference to a horizontal. Theta, θ, is calculated by taking the inverse tangent of the ratio of wall height to wall length.

Bending strains in walls were also computed. Walls of structures, which approximate flexible plates, tend to flex in a direction perpendicular to the plane of the wall with maximum displacements in the first mode of response at the middle of the wall. Such wall flexure is directly related to the bending strain induced in the walls and were modeled as a beam fixed at both ends, at the foundation (S1) and at the roof (S2). It has been determined that the foundation is well-coupled to the ground, or “fixed”. However, the roof can be modeled with varying degrees of “fixity”, ranging from relatively unconstrained to highly fixed. Bending strain is most conservatively estimated with the fixed-fixed analogy because this model predicts the highest strains in walls per unit of maximum relative displacement. These out-of-plane bending strains can be calculated as:

$$\varepsilon_L = \left(\frac{6d\Delta\delta_{\max}}{L^2} \right) \tag{6}$$

where

ε_L = bending strain in walls (micro-strains or 10^{-6})
d = the distance from the neutral axis to the wall surface, or one half the thickness of the wall subjected to strain (inch)

The results of strain calculations for both structures are summarized in Tables 6 and 7, for Welch residence and the Marian Manor, respectively. For the Welch dwelling, the largest computed in-plane wall strains are associated with the two blasts on 7/26/05 generating the highest ground motions and lowest frequencies. These strain levels are extremely low. The maximum computed in-plane tensile strains were 5.16 and 6.22 micro-strains (0.00000516 and 0.00000622 strains) in the southeast and southwest walls, respectively, for the blasts on 7/26/05. The largest overall bending strain, 4.5 micro-strain, was computed for the southwest wall during the blast at 1:12 pm.

In comparison with the Welch residence, the in-plane tensile and bending strains in the Marian Manor were far smaller as a result of the stiffer concrete block construction. Similar to the

Welch structure, the two blasts on 7/26/05 produced the largest in-plane strains of 1.92 and 2.48 micro-strains (0.0000019 and 0.00000248 strains). These strain values are exceptionally low and far below levels that could possibly induce cracking in brick or mortar materials. The highest bending strain was 1.34 micro-strains.

According to Dowding (1985), the range of failure strains in the gypsum core of drywall is 300 to 500 micro-strains. Exterior fire brick façade does not fail before 500 to 1000 micro-strains. Using the maximum observed tensile strain of 6.22 (Welch), the minimum factor of safety against drywall cracking is 48, and well above the safe limits of cracking. For concrete brick, using the maximum tensile strain of 2.48, the factor of safety against brick cracking is 403. At such low levels of blasting, the induced strains could never exceed the elastic limit of the wall materials. Hence, no permanent deformation could have occurred and cracking both in interior drywalls and brick walls were not caused by blasting activities at the levels recorded during this project.

Table 6 Calculated strains for all blasts at the Welch structure in comparison with ground motion velocity components

Shot Date	Shot Time	Maximum differential wall displacement, S2-S1 (in)		Maximum shear strain (micro-strain)		Maximum in-plane tensile strain (micro-strain)		Maximum bending strain (micro-strain)	Maximum ground velocity (in/sec)		Peak Crack Motion
		SW Wall (T)	SE Wall (R)	SW Wall (T)	SE Wall (R)	SW Wall (T)	SE Wall (R)	SW Wall (T)	Transverse	Radial	(micro-in)
6/16/2005	1:03 PM	nr									
6/21/2005	12:59 PM	nt									
6/21/2005	1:12 PM	0.00044	0.00053	5.89	4.89	2.94	2.37	2.78	0.038	0.038	75.5
6/23/2005	1:00 PM	bad data at S1							0.045	0.048	72.1
6/28/2005	1:00 PM	0.00026	0.00035	3.89	2.89	1.94	1.40	0.93	0.020	0.033	38.7
6/28/2005	1:27 PM	nt									
6/30/2005	12:16 PM	0.00039	0.00068	7.56	4.33	3.78	2.10	1.41	0.035	0.035	47.5
7/5/2005	1:05 PM	0.00041	0.00046	5.11	4.56	2.55	2.20	1.37	0.035	0.065	71.5
7/7/2005	12:59 PM	nt									
7/12/2005	12:59 PM	0.00068	0.00051	5.67	7.56	2.83	3.66	2.57	0.040	0.035	45.7
7/14/2005	1:00 PM	0.00033	0.00058	6.44	3.67	3.22	1.77	1.43	0.038	0.048	nd
7/19/2005	1:02 PM	0.00049	0.00062	6.89	5.44	3.44	2.63	1.74	0.040	0.038	66.8
7/26/2005	**1:06 PM**	**0.00070**	**0.00112**	**12.44**	**7.78**	**6.22**	**3.76**	2.52	**0.055**	**0.063**	**115.7**
7/26/2005	**1:12 PM**	**0.00096**	**0.00084**	**9.33**	**10.67**	**4.67**	**5.16**	**4.50**	**0.073**	**0.065**	**151.6**
7/28/2005	1:13 PM	0.00050	0.00076	5.56	8.44	2.69	**4.22**	2.44	**0.060**	0.048	87.7

Table 7 Calculated strains for all blasts at the Marian Manor in comparison with ground motion velocity components

Shot Date	Shot Time	Maximum differential wall displacement, S2-S1 (in)		Maximum shear strain (micro-strain)		Maximum in-plane tensile strain (micro-strain)		Maximum bending strain (micro-strain)		Maximum ground velocity (in/sec)		Peak Crack Motion
		West Wall (T)	South Wall (R)	West Wall (T)	South Wall (R)	West Wall (T)	South Wall (R)	West Wall (T)	South Wall (R)	Transverse	Radial	(micro-in)
06/16/05	1:03 PM	0.00024	0.00029	2.60	2.15	1.07	0.85	1.12	0.61	0.020	0.015	64.0
06/21/05	12:59 PM	0.00011	0.00022	1.97	0.99	0.81	0.39	0.45	0.47	0.010	0.008	
06/21/05	1:12 PM	0.00020	0.00031	2.78	1.79	1.15	0.71	0.67	0.96	0.020	0.013	
06/23/05	1:00 PM	0.00048	0.00029	2.60	4.30	1.07	1.70	0.93	0.89	0.030	0.013	
06/28/05	1:00 PM	0.00038	0.00048	4.30	3.41	1.78	1.34	1.34	1.11	0.043	0.035	
06/28/05	1:27 PM	0.00024	0.00030	2.69	2.15	1.11	0.85	0.73	0.81	0.018	0.010	
06/30/05	12:16 PM	0.00026	0.00029	2.60	2.33	1.07	0.92	0.60	0.72	0.020	0.010	
07/05/05	1:05 PM	0.00042	0.00041	3.67	3.76	1.52	1.49	1.08	1.10	0.043	0.065	
07/07/05	12:59 PM	0.00025	0.00029	2.60	2.24	1.07	0.88	0.79	0.83	0.015	0.013	
07/12/05	12:59 PM	0.00027	0.00023	2.06	2.42	0.85	0.96	0.60	0.76	0.020	0.018	
07/14/05	1:00 PM	0.00027	0.00034	3.05	2.42	1.26	0.96	1.05	1.05	0.040	0.043	
07/19/05	1:02 PM	0.00028	0.00038	3.41	2.51	1.41	0.99	0.70	0.92	nd	nd	
07/26/05	1:06 PM	0.00035	0.00067	6.00	3.14	2.48	1.24	1.12	0.89	0.028	0.025	
07/26/05	1:12 PM	0.00027	0.00052	4.66	2.42	1.92	0.96	1.11	0.96	nd	nd	
07/28/05	1:13 PM	0.00016	0.00041	3.67	1.43	1.52	0.57	0.64	0.65	nd	nd	

CRACK DISPLACEMENTS

Crack Response to Blasting Vibrations

The dynamic responses of existing interior cracks were measured at the two structures during blasting events. Crack displacement time histories are given in Appendices C and D for the Welch residence and the Marian Manor, respectively. Unfortunately the crack measurement system at the Marian Manor was unreadable and download software problems occurred after the first blast. Therefore, the dynamic (blast-induced) crack response for only the first blast was possible.

Tables 6 and 7 show the peak crack displacement measured during blasting for each structure. The peak dynamic crack displacements at the Welch dwelling ranged from 47.5 to 151.6 micro-inch (0.0000475 to 0.0001516 inch). The single peak crack displacement at the Marian Manor was 64 micro-inch. (0.000064 inch).

Long-Term or Environmental and Weather-Induced Crack Response

The width of existing wall cracks is highly sensitive to changes in ambient temperature and humidity. Many of the existing exterior and interior cracks in a structure are attributed to blasting. However, it is often the case that the dynamic response of cracks to blasting is small compared with the static, or slow, opening and closing of existing cracks with diurnal (or 24-hour) fluctuations in temperature and humidity. To show this comparison, long-term weather-induced changes in crack widths were measured and recorded on an hourly basis throughout the study period. Changes in crack widths were plotted against time for each structure crack and shown in Figures 14 and 15 for Welch and Marian Manor structures, respectively. A positive increase in crack displacement corresponds with opening of the crack.

In general, crack movement follows the trend in the ambient humidity (exterior or interior). When humidity increases, the crack opens and this occurs most predominately very early in the mornings, well before dawn. During the day, as temperature increases and humidity decreases, the cracks tend to close. It is this daily cycle that produces high stresses on the crack and in particularly, at the tips or ends of the cracks, causing the crack to grow slowly over time under the right conditions.

The largest variation in crack width over a one-half day cycle was measured and shown with arrows in Figures 14 and 15. The largest change over a daily half-cycle was 20,105 and 5425 micro-inch (0.020105 and 0.005425 inch) for the Welch and Marian Manor structures, respectively. These are significant changes that lead to high stresses at the crack tips. High changes in crack width are correlated with extensions and elongations of cracks. The static, or weather-induced, crack displacements far exceeded the largest dynamic, blast-induced, changes in crack widths. Therefore, there is a high probability that wall cracks are climate-induced rather than blast-induced.

To further illustrate this comparison, daily changes in crack width over an 8-day period are compared with the dynamic crack motions for the most significant blast on 6/283/05 at 1:12 pm in Figure 16 for Welch residence. A similar comparison is shown in Figure 17 for the Marian Manor for the blast on 6/16/05. The largest blast response for the Welch drywall crack was 151.6 micro-inch zero-to-peak and 293 micro-inch peak-to-peak. As such, the weather-induced crack displacement was 68 times greater than the displacement for blasting (comparing peak-to-peak values).

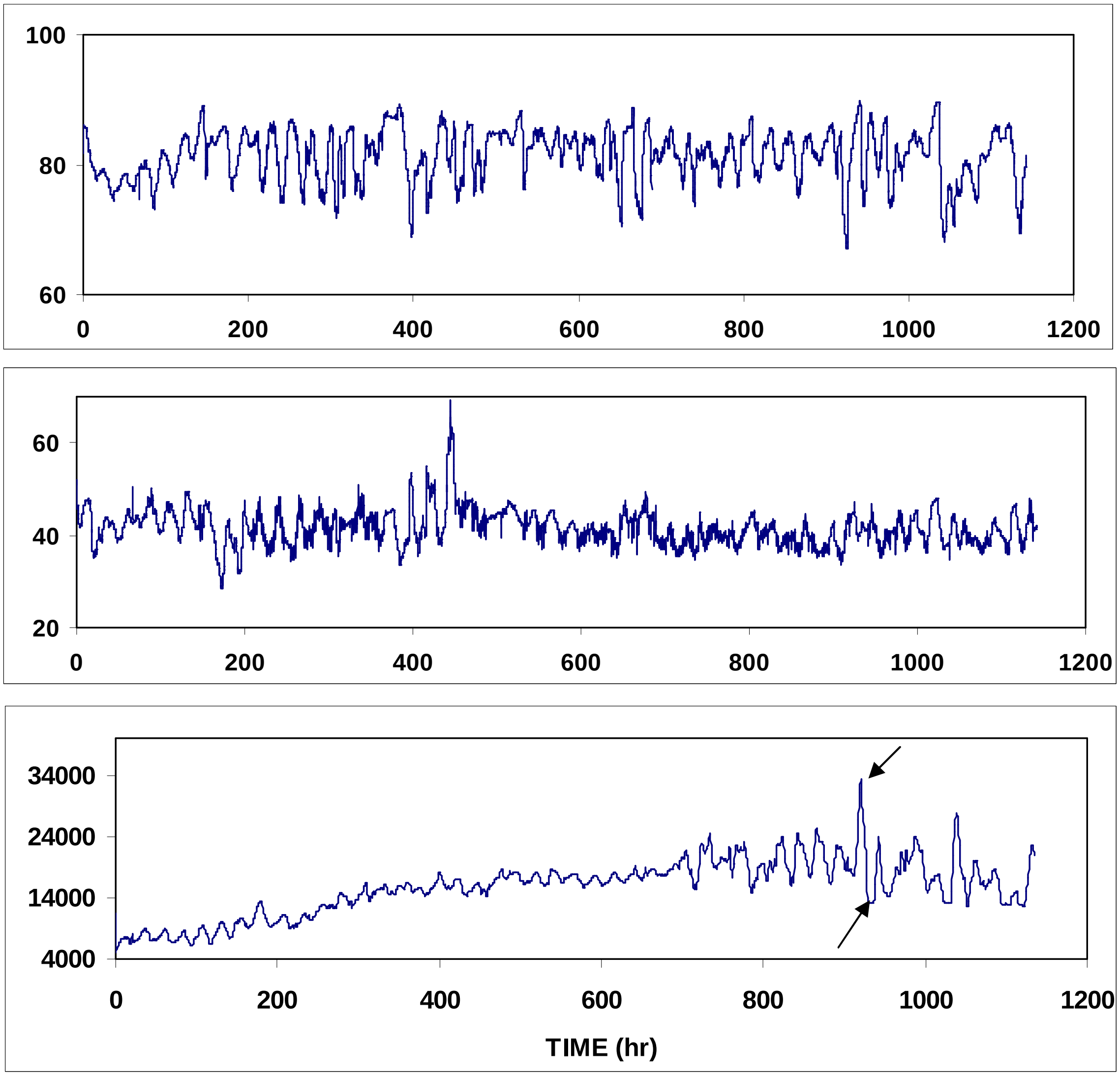


Figure 14 Variation in ambient temperature, humidity and corresponding crack displacement over project duration for the Welch Residence

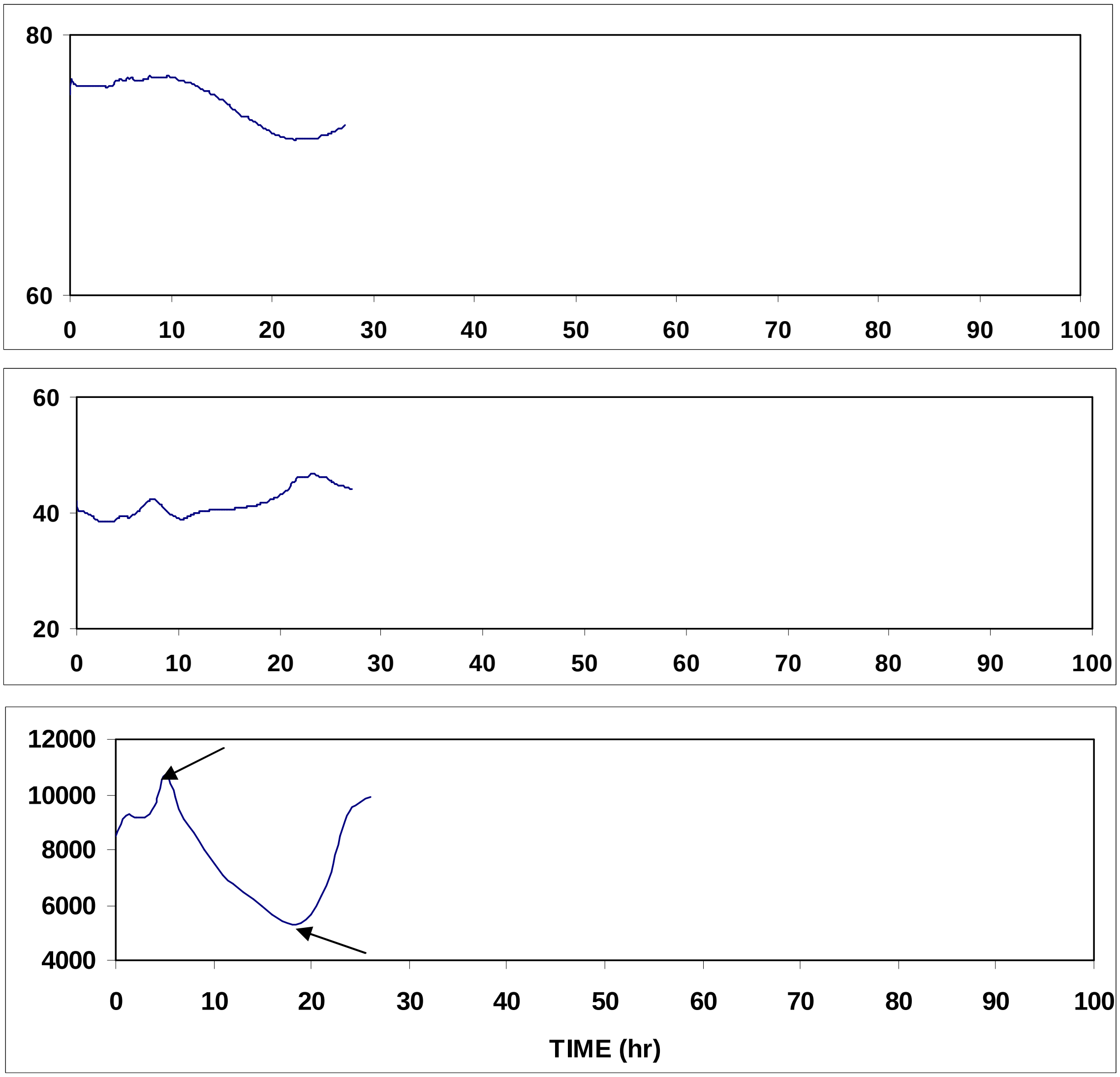


Figure 15 Variation in ambient temperature, humidity and corresponding crack displacement over 24 hours for the Marian Manor

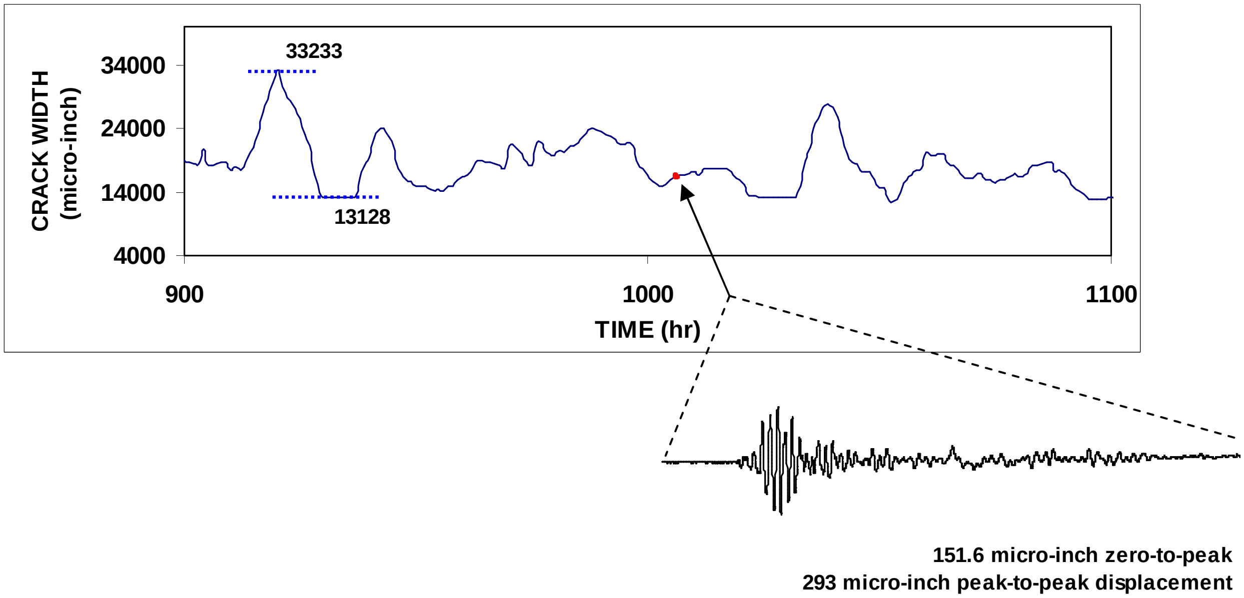


Figure 16 Comparison of dynamic crack displacement time history at the Welch residence for the blast on 7/28/05 at 1:12 pm (shown in the lower right) with static crack movement in response to climate over an 8-day period

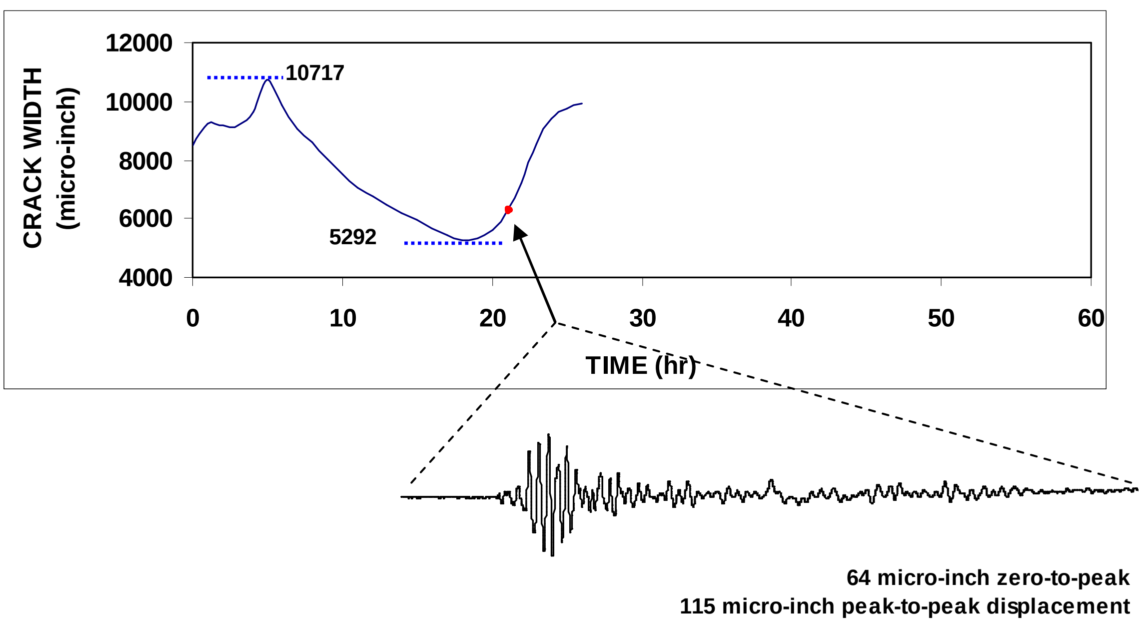


Figure 17 Comparison of dynamic crack displacement time history at the Marian Manor for the blast on 6/16/05 at 1:03 pm (shown in the lower right) with static crack movement in response to climate over a 1-day period

The blast-induced response for the Marian Manor concrete block crack was 64 micro-inch zero-to-peak and 115 micro-inch peak-to-peak. The weather-induced peak-to-peak crack motion was 47 times greater than the blast-induced motions.

It is therefore concluded that the large weather-induced changes in crack width are the greatest contributing factor to crack extension and widening over time. Blasting influences on changes in crack widths and the potential formation of cracks in structures are negligible compared with cracking potential from the influence of climate changes. Hence, blasting is unlikely to be the source of the existing cracks used in this study.

HUMAN-INDUCED STRUCTURE MOTIONS

Each structure was subjected to vibrations from normal, everyday activities such as opening and closing doors and windows, walking into rooms, and dropping objects on floors. Both peak structure wall motions (in terms of velocity) and crack displacements were measured and compared with crack responses during blasting events.

Human-induced structure velocity time-histories are shown in appendices E and F while appendices G and H show crack displacement time histories. Tables 8 and 9 are summaries of computed wall strains and recorded peak crack displacements from various human-induced activities in the Welch and Marian Manor structures, respectively.

In all but two cases, human-induced activities in the Welch residence generated peak crack displacement greater than the largest blast-induced displacement (151 micro-inch). These are slamming the first floor front door (148 micro-inch) and gently closing the second story bedroom door (135 micro-inch). The largest peak crack displacement occurred when the heal of a shoe was dropped onto the floor (950 micro-inch).

Table 8 Summary of structure response during normal household activities in the Welch residence

Event	S2			S1			MW	Peak Crack
	T	R	V	T	R	V	southwest	
	(ips)							(micro-in)
hit SE wall	0.04	0.24	0.22	0.325	1.1	0.26	0.125	182
hit SW wall	0.295	0.045	0.08	0.98	0.42	0.08	0.98	530
shut left window	0.06	0.41	0.2	0.135	0.3	0.22	0.44	283
slam left window	0.125	0.64	0.504	0.385	1.04	0.38	0.8	725
shut left window	0.14	0.68	0.64	0.52	1.62	0.502	1.16	1201
backpack fall on floor	0.045	0.03	0.22	0.06	0.03	0.22	0.17	552
heal drop on floor	0.06	0.035	0.38	0.085	0.035	0.38	0.28	950
walk into room 1	0.03	0.02	0.16	0.03	0.015	0.16	0.095	414
walk into room 2	0.015	0.01	0.04	0.01	0.005	0.04	0.03	313
gently shut door	0.02	0.015	0.04	0.015	0.01	0.04	0.08	283
slam door	0.14	0.12	0.14	0.095	0.085	0.14	0.6	700
slam daughter's door 1	0.04	0.035	0.02	0.025	0.03	0.02	0.105	194
slam bathroom door	0.035	0.09	0.04	0.03	0.04	0.04	0.14	425
slam front door 1st floor	0.035	0.02	0.06	0.025	0.02	0.06	0.05	148
gently close door	0.015	0.015	0.02	0.02	0.015	0.02	0.07	135
slam daughter's door 2	0.045	0.045	0.02	0.04	0.03	0.02	0.15	787

Table 9 Summary of structure response during normal household activities in the Marian Manor

Event	S2			MW	S1			MW	Peak Crack
	T	R	V	west	T	R	V	south	
	(ips)								(micro-in)
slam west window	0.005	0.01	0.01	0.045	0.005	0	0.01	0.01	43.3
hit west wall	0.005	0.01	0.01	0.095	0.0005	0	0	0.02	54.6
hit south wall	0.005	0.015	0.01	0.03	0.005	0.005	0	0.045	39.8
slam door	0.02	0.02	0.01	0.08	0.005	0.005	0.01	0.055	22.6
shut door gently	0.005	0.005	0.01	0.055	0	0	0	0.015	18.6

Figure 18 shows a comparison of wall strain in the southwest wall from differential displacements of the southwest (driving) wall and the crack time history (bottom diagram) for the blast producing the largest crack motions (Figure 18 a) and for slamming the left window on the southwest wall (near the crack, in Figure 18 b). The wall strain and peak crack displacement induced by closing the window are over 2.5 times and 4.8 times greater than blast-induced values.

It was more difficult for human-induced activities to displace the crack through the concrete block of the one-story Marian Manor. The brick construction is relatively stiff and requires large dynamic forces to move the walls, thereby displacing the crack. Slamming the west wall window and hitting the west wall (the wall containing the crack) only produced 54.6 and 43.3 micro-inch peak displacements, respectively. The blast on 6/16/05 produced a peak crack response of 64 micro-inches.

CONCLUSIONS

The following conclusions are drawn from this study:

- The Sibley quarry ground vibration data recorded at the two sturdy structures fell within the expected data scatter for a typical competent limestone quarry. Therefore it was concluded that the Sibley ground motion data recorded at the test structures are representative of typical quarry data and do not pose unusual geological trends or anomalies.

- During this study the maximum peak particle velocity (PPV) at the Welch residence and at the Marian Manor were 0.068 ips and 0.165 ips. These values are considered to be very low and well within safe limits protecting structures from damage.

- Based on the worst case ground motion frequencies, the greatest computed ground displacements generated during the maximum PPV recorded during this study are 0.00077 inch and 0.00164 inch at the Welch and Marian Manor structures, respectively. These ground deflection are exceptionally small and well within the elastic range of foundation soils and rock. As such, they cannot possibly lead to permanent deformations beneath foundations and hence damages within structures.

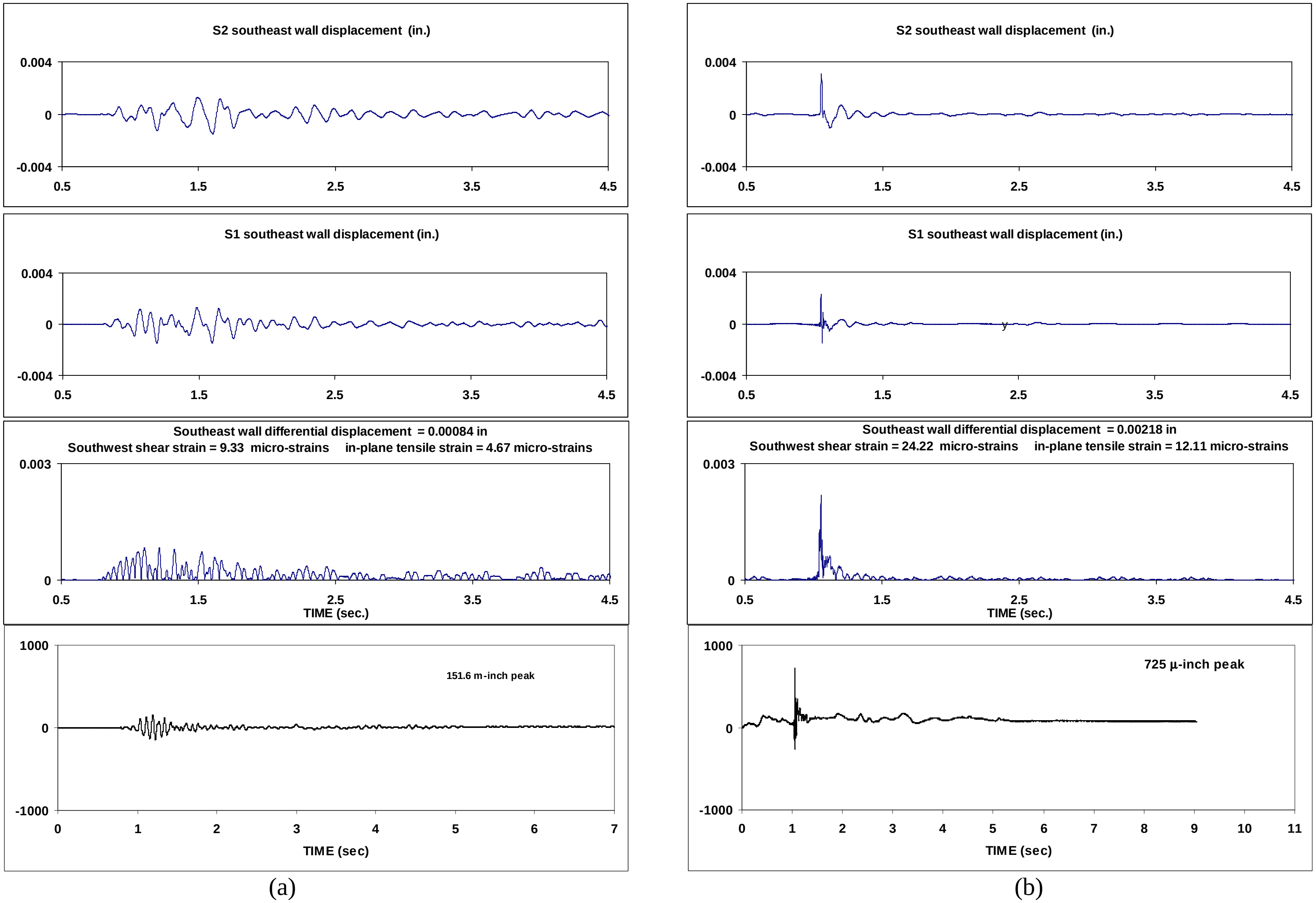


Figure 18 Comparison of wall and crack displacements at the Welch residence for (a) the blast on 7/28/05 at 1:12 pm with (b) slamming the southwest wall left window (note crack and corner response time scales are not the same

- Peak particle velocity (PPV) and frequency at the PPV were plotted within safe blasting criteria to demonstrate that all blasts conducted at the quarry during this study fell well below the upper limits for threshold cracking in structures. As such, it is not possible for these levels of ground motions and frequencies to cause hairline or threshold cracking in structures.

- Dynamic structure properties of natural frequency and damping could not be computed at the Marian Manor as structure motions were too low to accurately determine these parameters. Similarly, damping of the Welch structure motions was not possible as vibration motions were too low. The natural frequency of the Welch structure was determined as reported as 8 and 9 Hz in the southeast and southwest walls, respectively. These values are well within the expected range for a two-story structure.

- The calculation of amplification factors (AF) could only be performed for the blast with the highest ground motion at each structure. AF were computed to be 1.6 and 1.9 for the Welch and Marian Manor structures and were well within normal, expected values for one- and two-story structures.

- Wall strains were computed and compared with strains required to cause threshold cracking in drywall (Welch) and brick (Marian Manor). For the Welch residence, the maximum computed in-plane tensile strains were 3.76 and 6.22 micro-strains in the southeast and southwest walls, respectively. The largest overall bending strain, 4.5 micro-strain, was computed for the southwest wall. In-plane tensile and bending strains in the Marian Manor were far smaller as a result of the stiffer concrete block construction. The largest in-plane strain was 2.48 micro-strains (0.00000248 strains) and the largest bending strain was 1.34 micro-strains. These strain values are exceptionally low and far below levels that could induce cracking. The range of failure strains in the gypsum core of drywall is 300 to 500 micro-strains. Exterior fire brick façade does not failure before 500 to 1000 micro-strains. Thus, the minimum factor of safety against drywall cracking is 48 (Welch) and for cracking in brick it is 403 (Marian Manor). At such low levels of blasting vibrations, the induced strains could never exceed the elastic limit of the wall materials. Hence, no permanent deformation can occurred and cracking in interior drywalls and brick walls were not caused by blasting activities at the levels recorded during this project.

- An existing interior crack was instrumented at each structure to record dynamic, blast-induced peak crack displacements. Blast-induced crack displacements at the Welch dwelling ranged from 47.5 to 151.6 micro-inch (0.0000475 to 0.0001516 inch). The single peak crack displacement at the Marian Manor was 64 micro-inch (0.000064 inch)

- Long-term, weather-induced changes in crack widths were measured and recorded on an hourly basis. The largest change over a daily half-cycle was 20105 and 5425 micro-inch (0.020105 and 0.005425 inch) for the Welch and Marian Manor structures, respectively. The static, or weather-induced, crack displacements far exceeded the largest dynamic, blast-induced, changes in crack widths. As such, the weather-induced crack displacement was 132 and 85 times greater than the displacement for blasting for the Welch structure and Marian Manor, respectively. Blasting had a negligible effect on crack opening and closing when compared with the normal effects of weather.

- Normal, everyday human activities were performed to measure structure and crack motions and compare these with blast-induced motions. The second story of the Welch residence responded to human activities with peak crack motions greater than the largest blast-induced motions for all but two activities. The largest crack response generated by closing the southwest window was 950 micro-inch compared with the largest blast-induced peak crack motion of 151 micro-inch. The Marian Manor did not show significant crack response from human activities greater then the blast-induced peak displacement based on the relative stiffness of the concrete block structure.

- In summary, it is not possible that the low levels of blasting caused any threshold cracking in either the Welch or Marian Manor structure. Weather-induced wall material crack displacements were found to be one to two orders of magnitude greater than blast-induced movement. In the two-story structure, many human-induced activities generated crack displacements up to 6.3 times greater than those generated from blasting. Blasting cannot possibly cause threshold material cracking when ground vibrations amplitude and frequencies fall within (below) the safe blasting criteria. All observed cracks in both structures are deemed normal and due to aging, changes in weather, and normal wear and tear.

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