

# **Micro-meter Crack Response to Rock Blast Vibrations, Wind Gusts & Weather Effects**

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## **ABSTRACT**

Micro-meter dynamic crack responses in a two story structure to rock blasting- and wind gust- excitation are compared to those induced by long term climatological effects. These measurements substantiate the conservancy of the 12.5 mm/s (0.5 in./s) blasting vibration control to protect residential structures against cracking. The test structure was instrumented with both velocity transducers to measure superstructure motions as well as special micro-meter displacement sensors to measure crack expansion and contraction. Crack responses to 48 to 64 km/hr (30-40 mph) wind gusts were equal to those generated by blasting vibrations of 11.5 mm/s (0.45 in./s) or 90% of the control limit. There are few reports of wind responses in the rock blasting literature because the long period of the wind “drift” motions produces low structural velocity responses, which are below the noise (and thus trigger levels) of most standard seismographs. The paper closes with an assessment of the frequency of occurrence of significant wind gusts during an average weather year.

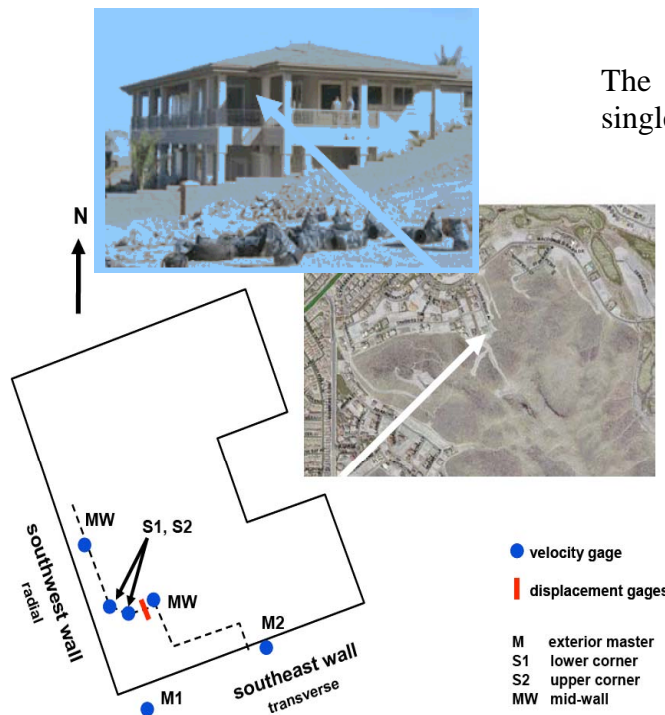
## **INTRODUCTION**

This paper describes the importance of wind gust deformation of residential structures and its measurement relative to that caused by typical construction blasting activity. Earlier work by Dowding and McKenna (2005) has established the importance of changes in temperature and humidity on micro-meter crack deformation in comparison to the effects of mine blasting. Measurements in this case study now extend weather effects to include wind. While it has been long known that air over pressures from blasting activity could cause house response, little information has been available to demonstrate the analogous response that should be

produced by wind gust induced over pressures. Since engineered structures and homes are designed for wind induced effects through standard velocity-pressure relationships, it has long been argued (Siskind, 1999 for instance) that wind must be deforming structures. Furthermore standard calculations show that typical storm wind speeds produce wind over pressures greater than those produced by the upper bound of allowable blast induced over pressures. But why has there been so little confirmation of such an obvious presumed effect?? Measurements in this paper demonstrate that little information has been collected because typical velocity transducers employed in measuring blast response may not be able to sense wind pressure induced “drift” because of its low frequency. Displacement sensing crack deformation sensors can. Finally the paper ends with an analysis of the frequency of occurrence of significant wind gust event and an assessment of their importance.

## PROJECT SETTING

The house shown in Figure 1 was instrumented in conjunction with a study by the City of Henderson, NV to determine the applicability of national blasting control limits (Henderson, 2005). As shown by the aerial photograph, it was located on a ridge near other development to the southwest, which exposes it to both wind and blast induced excitation and is an ideal example for a comparison between the two effects.

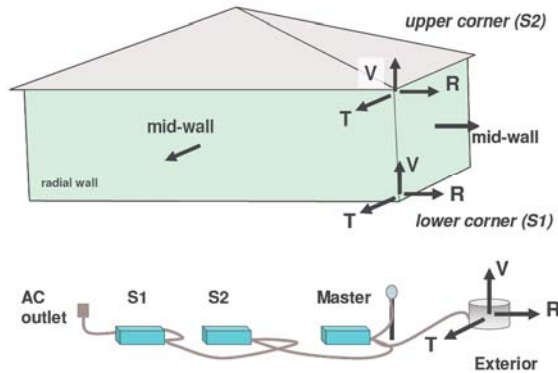


**Figure 0: Location of House and Instrumentation**

The house was fitted with both single axis transducers as well as crack and null sensors on the second story as shown in Figures 1, 2 and 3. Single component velocity transducers were placed in the upper (S2) and lower (S1) corner and on two mid-walls (MW) as shown in Figure 2. The exterior (master) unit consisted of the standard tri-axial geophone and air overpressure microphone manufactured by LARCOR (Aimone, 2005). Master geophones were aligned so that the radial, R, component was oriented parallel to the

longest axis of the house as shown. This orientation, rather than the traditional R toward the blasting, was chosen so house responses and ground motions in the same

direction could be compared directly without modification. Geophones were buried 10 cm in the ground and the microphone was installed 10 to 15 cm above the ground surface.



**Figure 0: Organization of Velocity Transducers**

seconds. Each cluster measured the three components of gross structure response at either the bottom (S1) or top (S2) of the story containing the crack sensor and an adjacent mid-wall response. Differences in top and bottom corner displacement can be employed to calculate shear strain and differences between mid-wall and corner displacement can be employed to calculate wall bending strains.

Response of a horizontal exterior stucco crack to blasting and climate (temperature, humidity, and wind) was measured with Kaman, eddy current, micro-meter displacement sensors. The location of the crack and null sensors on the structure is shown in Figure 3. As has been the case with similar installations



**Figure 0: Photograph of Micro-meter Crack Displacement Sensors**

the same sensor is employed to monitor both long-term response to changes in climate and the dynamic response to blast induced excitation. The null sensor is affixed in the same fashion on adjacent uncracked wall material to measure wall and sensor material response. This wall-sensor response is then subtracted from the crack response to compensate for response of the sensor and wall itself. Although this sensor-wall material response is small, the null sensor is normally employed as a precautionary measure. Both crack and null sensor are attached to a Somat field computer (Siebert, 2002) to record response. Long-term response is obtained by sampling the crack and null sensors every hour, while dynamic response is obtained by sampling at 1000 sample per second for three

Structural response was measured with velocity transducers mounted perpendicular to walls in clusters as shown in Figure 2. Clusters were operated with a separate seismograph, which was triggered by the exterior unit. Trigger thresholds for ground motions and air overpressure were set at 0.8 millimeter per second (mm/s) and 125 decibels (dB) respectively. Velocity response time histories were digitized at a rate of 512 samples per second for 12

seconds upon triggering by the exterior seismograph. Temperature and relative humidity are recorded with a SUPCO data logger at a sample interval of 10 minutes.

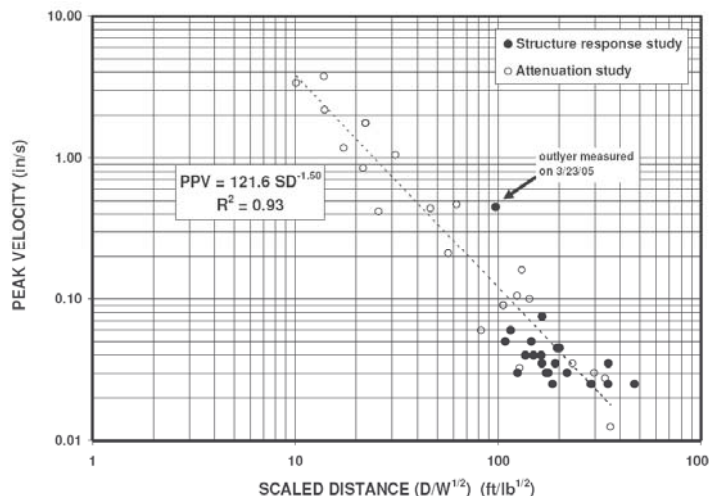
## BLASTING ENVIRONMENT

The ridges shown in Figure 1 are composed of andesite, tuff, and basalt flows with soil overburdens of 7 feet or less. Soil overburdens increase near the bottoms of the ridges. Water was not intersected in any of the geotechnical exploratory bore holes. Shear wave propagation velocities for the upper soils ranged between 450 and 1000 m/s while that for the upper rock ranged between 1400 and 3800 m/s.

**Table 1: Selected Blast Events, Resulting Ground Motions, Air Overpressures, and Crack Response**

Date	Time	Dist(D)	Charge(W) per delay	Scaled Distance	Peak Ground Velocity		Air Overpress.	Max Crack Disp.
		(m/feet)	kg/lb	R/W <sup>1/3</sup>	Radial mm/s	Trans. mm/s		
3/15/2005	10:00	354/1163	23/ 50	164	0.6	0.9	100	1.2/ 48
3/21/2005	11:07	1033/3389	136/ 300	195	1.0	1.1	100	1.5/ 62
3/22/2005	12:35	1341/4400	472/1040	136	1.0	1.0	100	1.1/ 46
3/23/2005	2:47	284/ 933	42/ 92	97	8.4	11.4	110	6.1/243
4/14/2005	4:00	1336/4385	200/ 439	209	0.9	1.0	nt	1.1/ 46

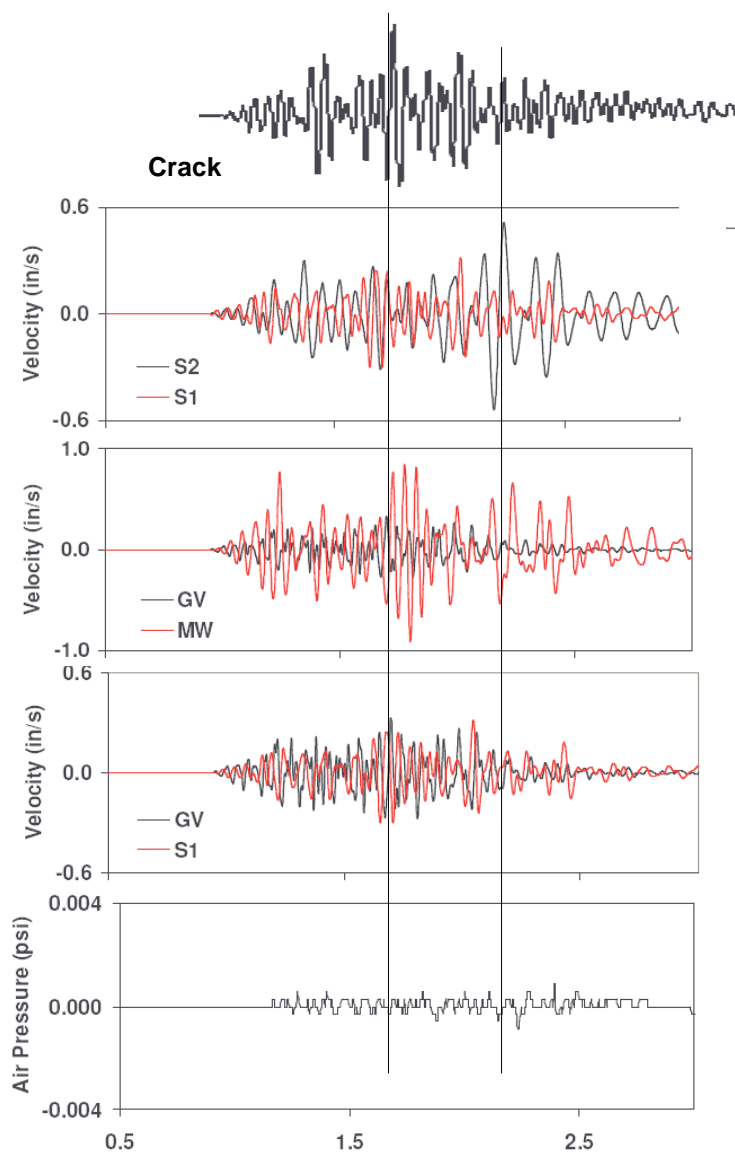
Construction blasting was undertaken to clear house pads and roadways as well as to excavate sewer trenches. Diameters of blast holes varied between 75 and 150 mm (3 and 6 inches). Ammonium nitrate fuel oil (ANFO) was used as the explosive, which was detonated by a 0.15 to 0.2 kg cast primer at the blast hole bottom. Some wet holes included an additional primer near the top to ensure initiation. Individual holes were detonated with non-electric detonation systems with separations of at least 8 milliseconds. On average the total weight of explosives detonated to fragment a specific volume of rock (powder factor) was approximately 0.5 kg/m<sup>3</sup> (or 1 lb/yd<sup>3</sup>).



**Figure 4: Attenuation of Peak Particle Velocity**

Shot design information concerning specific blasts described herein is presented in Table 1. As can be seen, blasts were typically designed to detonate a maximum charge in any one delay, which declined as the distance to the structure declined. Ground motions are controlled by designing blasts with a scaled

distance chart like that for this site in Figure 4 ( $25 \text{ mm/s} = 1 \text{ in/s}$ ). The desired peak particle velocity (PPV) is related to the required scaled distance, which is employed in design. Scaled distance is the distance between shot and closest structure divided by the square root of the maximum charge detonated in any instant (delay). Since the distance is always known, the desired maximum charge per delay can then be found. Since the scaled distances in these cases varied between 100 and 200, it can be seen that it was desired to reduce PPV's to less than 0.1 ips or 2.5 mm/s. Similar charts can be employed to control air over pressures. The 3/23 shot is an anomaly in that the measured ground motions exceeded those expected. As discussed by Aimone (2005) the reported maximum charge per delay may not be that which was detonated. To produce such a PPV, it is more likely that some 110 kg (240 lbs) were detonated in one delay.



**Figure 5: Comparative Timing of Crack Response, Ground Motions, Structure Response, and Air Over Pressure**

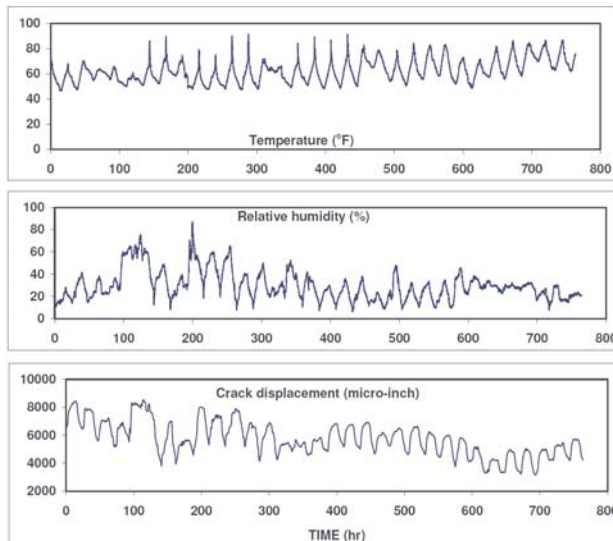
## HOUSE RESPONSE TO BLASTING EVENTS

Structural response to blast events is produced by both the ground motion and air overpressure as shown by the time histories from the 3/23 shot in Figure 5. ( $25 \text{ mm/s} = 1 \text{ in/s}$ ) This comparison of the timing of southwest wall responses (S2, S1 and MW) with parallel ground motions (GV) and the air over pressure demonstrates the dominant influences. The mid-wall responses occur during the maximum ground motions because their high natural frequency most closely matches the high dominant frequency of the ground motions. On the other hand the air overpressures produce the greatest corner or gross structure response as shown by the correspondence of the maximum S2 response with the later arriving air maximum over pressure



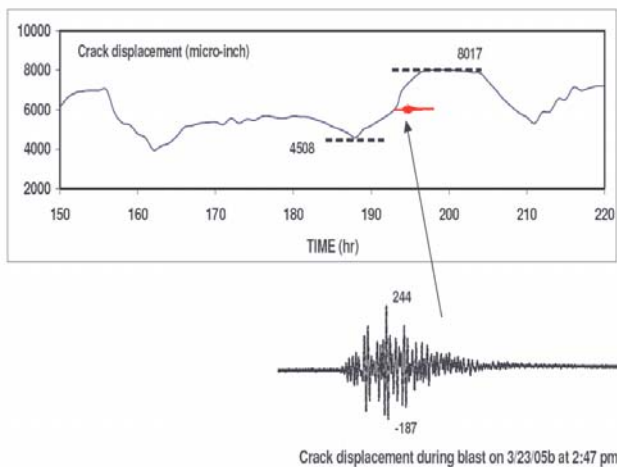
pulses. Free response of the corner motions (S2) at the end indicate that the natural frequency of the super structure is approximately 8 to 9 Hz, while the peak mid-wall responses indicate that the wall natural frequencies are some 16 Hz. Dominant frequency of the principal pulse of the ground motions varied between 21 and 32 Hz. Crack response peaks during the time of the highest amplitude ground motions.

## CRACK RESPONSE TO CHANGES IN TEMPERATURE & HUMIDITY



**Figure 6: Time History of Variation of Crack Response with Temperature and Humidity**

Long term response of the crack was obtained by recording its change in width every hour for some 30 days from mid March to mid April. This response is compared to changes in exterior temperature and humidity in Figure 6 ( $40 \mu \text{ in.} = 1 \mu \text{ m}$ ). The crack width changes in response to both temperature (daily) and humidity (with the passage of weather fronts). A longer monitoring period would have revealed a seasonal response as well.



**Figure 7: Comparison of Large Environmental Crack Response and Small Blast Response Even for PPV of 11.5 mm/s (0.45 in/s)**

Crack response to the largest blast event on 3/23 is compared with its response to daily weather effects in Figure 7 ( $40 \mu \text{ in.} = 1 \mu \text{ m}$ ). Even with a PPV of 11.4 mm/s (0.45 in/s) that is 90 % of the control limit, the blast induced crack response is only 1/7 th or 14 % of that of the maximum weather induced response over a four day period including 3/23. Specifically the zero to peak blast response was  $6 \mu \text{ m}$  ( $243 \mu \text{ in}$ ) while that day the zero to peak weather induced response was  $42 \mu \text{ m}$  ( $1750 \mu \text{ in}$ )

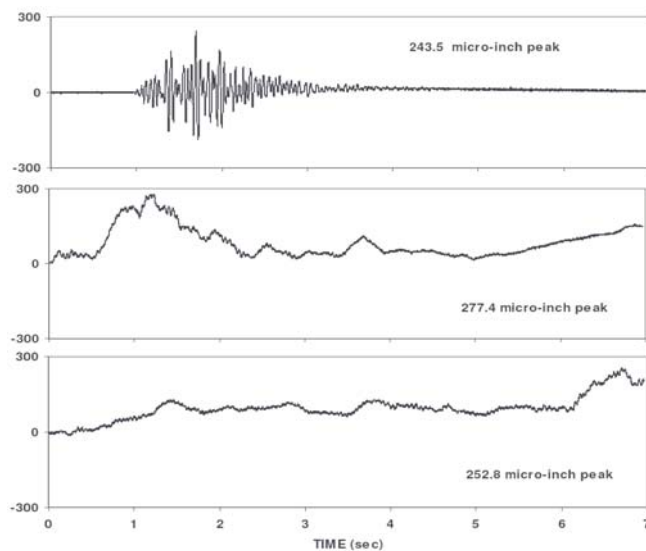
## CRACK RESPONSE TO WIND GUSTS

Wind response was obtained as the result of an unanticipated weather event. During the night of 22 March and early morning of 23 March a low pressure front moved quickly through Henderson and brought with it high wind gusts and high average wind speeds. Average wind speeds peaked during the early morning at the Henderson airport at 45 km/hr (28 mph) with gusts up to 64 km/hr (40 mph). Despite having set the trigger threshold at 125 dB to decrease the number of wind triggered events and locating the air pressure transducer near the ground where the wind velocity is low, three wind triggered events were measured that evening. After analyzing the responses and concluding that the events were in fact storm related, other similar anomalous events

**Table 2: Wind Gust Induced Crack Displacement with the Associated Wind Air Overpressure**

Date	Time	Maximum crack displacement	Air pressure	Equivalent airblast	Equivalent windspeed
		(micro-in)	(psi)	(dB)	(mph)
3/17/2005	9:08 PM	152.1	0.0057	125	17
3/17/2005	10:02 PM	169	0.007	127	19
3/22/2005	11:54 PM	252.8	0.0174	136	31
3/23/2005	2:15 AM	277.4	0.0212	137	34
3/23/2005	6:40 AM	37	0.0009	110	9

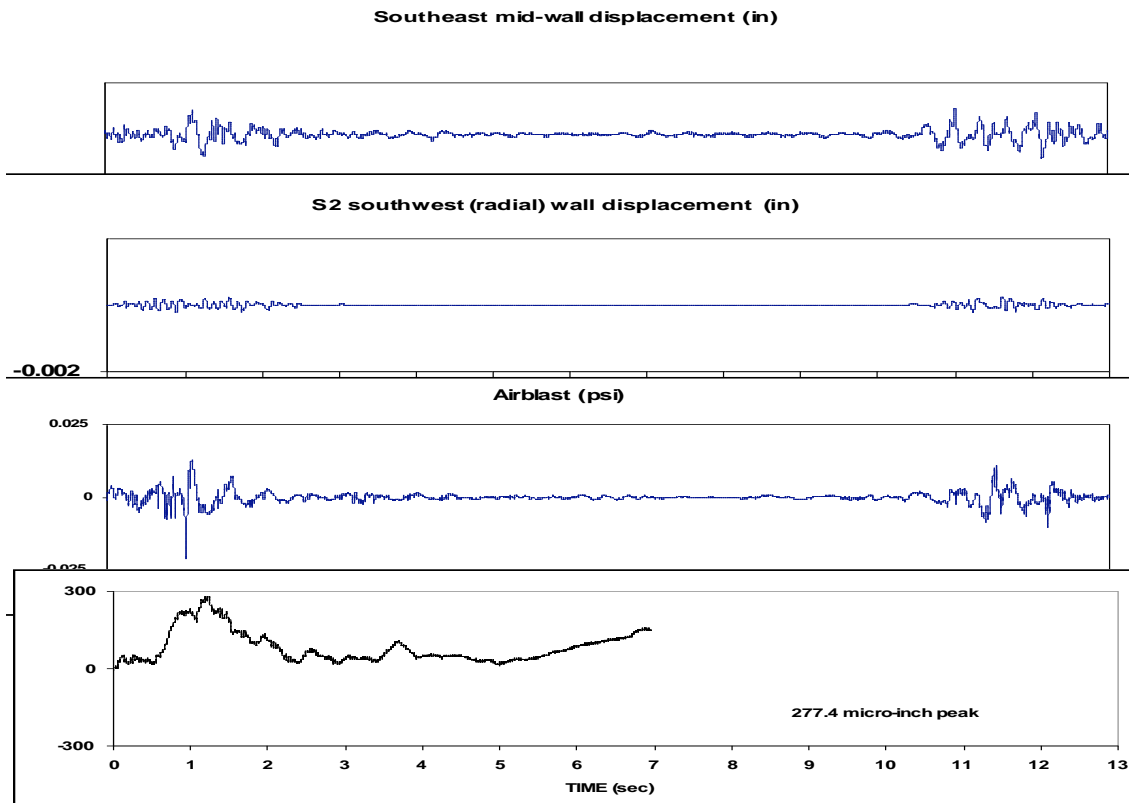
were reinterpreted. These five events are summarized in Table 2 ( 1 mph = 1.6 km/hr). The two largest crack responses were associated with back calculated wind gust speeds of 48 km/hr (30 mph) and greater. These equivalent local wind gust speeds were calculated from the maximum air overpressures (Aimone, 2005). This calculation may be low as gust velocities as high as 64 km/hr (40 mph) were reported at the local airport.



**Figure 8: Comparison of Low Frequency Gust Crack Response & High Frequency Blast Induced Response**

Time variation of crack response to wind deformation differs significantly from that for blast deformation from ground motions or air over pressure excitation. Response to the largest blast event is compared to wind response to the two largest gusts in Figure 8 ( $40 \mu \text{ in.} = 1 \mu \text{ m}$ ). The dominant response frequency to a blast event (10 to 20 Hz) is obviously significantly higher than that to wind gusts (0.3 Hz and lower).

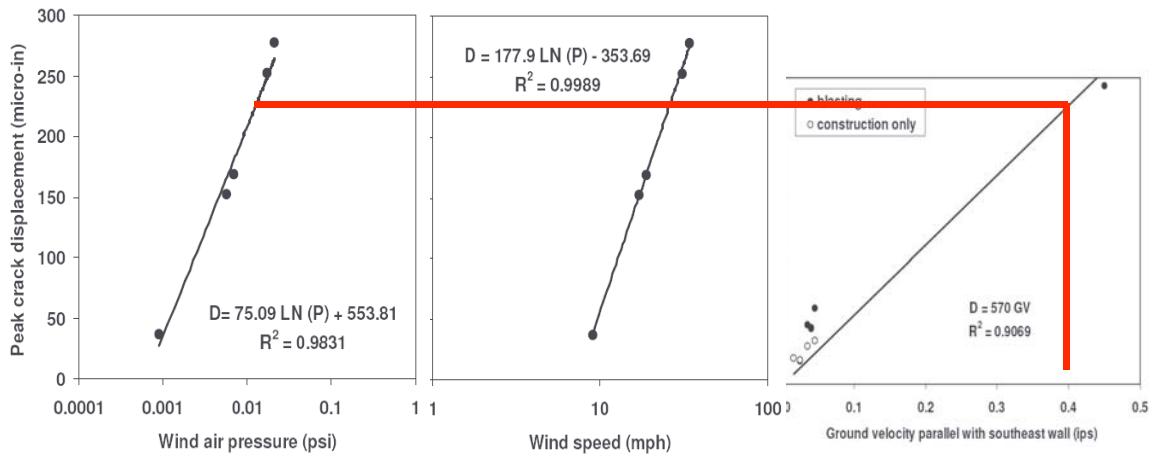
The low frequency ( $\sim 0.3$  Hz) or long period of the wind gust response shown in Figure 8 may be responsible for why gust response is not normally detected with typical blast vibration monitoring systems. Wind gusts produce little ground motion, so they can only be detected by triggering off the air overpressure transducer. Most investigators set the air over pressure trigger level high to avoid triggering off wind as there are normally too many events. If the system is triggered off velocity transducers on the superstructure, the frequency of wind drift ( $\sim 0.3$  Hz) is so low, that superstructure velocities fall below typical trigger levels of 0.02 to 0.04 ips. For example the large 3/23/05 wind gust that produced the large crack response only produced 0.02 and 0.03 ips in the upper structure, but did produce 0.1 ips at the midwall. Thus it appears that if systems cannot be triggered off the crack response itself, triggering velocity transducers should be placed at the midwalls, which do respond most to wind gusts as shown in Figure 9 below. Complicating all of these consideration is the fact that typical velocity transducers can only respond linearly down to 2 Hz.



**Figure 9: Comparison of displacement time histories of midwall and superstructure (top two graphs) and crack (bottom) during gust shown by airblast overpressure time history**



Graphical comparison of the sensitivity of this stucco crack to both blast events and wind gusts is helpful in describing the significance of wind events. Figure 10 (40  $\mu$  in. = 1  $\mu$  m) compares crack responses with wind generated air pressures, wind “gust” speeds and PPV levels to determine equivalencies. The relationships show that wind gusts of only some 48 km/hr (30 mph) generate the same crack response as a blast induced ground motion of PPV of 10 mm/s (0.4 in/s).



**Figure 10: Comparison of PPV Ground Motion and Wind Gust Speeds Necessary to Produce the Same Crack Response (1 psi = 7 kPa, 1 mph = 0.6 km/hr, 1 ips (in/s) = 25 mm/s)**

## COMPARISON OF WIND & BLAST INDUCED AIR PRESSURES

While 50 year design wind velocities are greater than 128 km/hr (80 mph) everywhere in the United States, what do historical weather records reveal about the reoccurrence of high, but not design, wind speeds from wind gusts. For example, consider a location in north central Tennessee. Between 1993 and 2003 weather data obtained at Nashville (since Nov 1996) and Clarksville (since Apr 2001) indicate on average during any one year, daily *maximum*, 5 second, wind gust velocities between 30-35 mph ( 1 mph = 1.6 km/hr) occurred 20 times and those between 36-40, 41-46 and 47-60 occurred 10, 3 and 1 times a year respectively, or 34 times a year wind gust velocities exceeded 30 mph.. Thus in last decade this area has been subjected to storms whose *maximum* wind gusts were from 30 to 40 mph -- 300 times; 40 to 45 mph -- 30 times; and 50 mph and greater -- 10 times. This count is not the number of such wind gusts. That count is much greater. This 34 times a year is the number of times a year that that maximum is recorded during the 24 hour period.

Simiu and Scanlan in their book *Wind Effects on Structures* (1986) indicate that maximum wind induced pressures can be estimated by the formula,  $P_{max} = 1/2 \rho V_{max}^2$ ; where P is the pressure in lb/ft<sup>2</sup> (psf),  $1/2 \rho$  is 0.00256 lb/ft<sup>2</sup>/(mph)<sup>2</sup> and  $V_{max}$  is the wind speed in mph. Thus a 48 km/hr (30 mph) wind gust would produce a wind pressure of 0.11 kPa ( 2.3 psf or 0.016 pound per square inch, psi), and a 64 & 80 km/hr (40 & 50 mph) gusts would produce wind pressures of 0.24 & 0.31 kPa (0.09 psf -0.028 psi-, and 6.4 psf -0.044 psi) respectively. Therefore over a 10 year period

wind storm gusts would produce *maximum* pressures > 0.1 kPa (0.016 psi) 300 times; > 0.2 kPa (0.028 psi) 30 times and > 0.3 kPa (0.044 psi) 10 times.

How do these wind gust air over pressures compare with current rock blasting control limits for air over pressure? Those employed by the Office of Surface Mining for coal production are 133 dB, which is equivalent to an over pressure of ~ 0.09 kPa (0.013 psi or 1.87 psf). Thus events that produce air over pressures at these control limits produce air pressures that are exceeded by the *maximum* daily wind storm gust pressures on average 34 times a year. As was seen in this case study by the occurrence of multiple events per day, the number is actually larger.

## CONCLUSIONS

Crack response to blasting vibrations at 90 % of the control limit or 11.4 mm/s (0.45 ips) were also induced by wind gusts as slow as 48 km/hr (30 mph). While house response depends on structure cross section, it is likely that wind will produce similar significant responses in other homes.

Even the largest wind and ground motion crack responses are small compared to long-term climatologically induced crack deformation.

Wind gusts of 48 km/hr (30 mph) or greater are likely to occur more than 30 to 40 times a year.

Wind gust response has been under reported in blasting and geotechnical literature because its low frequency induces response velocities that are undetectable with typical velocity transducers

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