COMPARATIVE STUDY OF STRUCTURE RESPONSE TO COAL MINE BLASTING

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ABSTRACT

Whole structure and mid-wall responses of 25 structures to surface coal mine blasting were characterized. Eighty-nine blasts were conducted at 11 mine sites throughout the U.S. to measure blast-generated dynamic response of atypical structures found in the proximity of surface coal mining. Atypical structures selected for this study include log-type, manufactured (single wide and double wide trailers), "mine camp"-type, adobe, and stone. Traditional acoustic microphones, tri-axial (ground) and single component (structure) velocity transducers were used to record airblast, ground motions, and structure response time histories with a common time base. The relative responses of selected "atypical" structures to blast vibrations and non-blasting causes of structural stress, including natural forces, environmental effects, and human habitation, are compared.

Data analyses for blast-induced motions were conducted to:

- compare vibration time histories in terms of velocity and calculated displacement within structures relative to ground excitations,
- evaluate the influence of air overpressures on structure response,
- evaluate response frequencies to determine natural frequencies and damping characteristics,
- determine structure response amplification of ground motions, and
- compute differential displacements of construction components and corner motions to estimate global or gross structure strains.

Corner and mid-wall motions from blasting were compared to motions induced by normal household activities and external forces such as wind. In addition, wall crack deformation responses to environmental changes, human-induced vibrations and blasting were measured in four of the structures in a parallel study.

Amplitudes of ground vibrations measured at structures ranged from 0.02 to 1.25 inches per second (in/sec). Scaled distances ranged from 22.9 to 340.0 ft/lb$^{1/2}$.

The amplifications of ground motions measured in upper structure corners varied by type of structure as well as for certain structures within each design type. Corner responses of log and wood-frame structures fell below values reported in U.S. Bureau of Mines RI 8507. For two structure designs (two-story log and two-story stone), amplifications greater than 4 were measured when excited by ground motions with predominant frequencies of 4 to 7 Hz.

Little difference in horizontal time histories between lower floor and ground motion responses were noted for all structure types with the exception of trailers without wood-frame add-ons. Single and double wide trailers produced wall base motions greater than exterior ground motions.

Trailer whole structure and mid-wall motions duplicated airblast time histories. Peak structure responses occurred within the airblast phase rather than within the ground motion phase, particularly when airblast exceeded 116 decibels. Mid-wall motions showed both high

frequency and low frequency characteristics for specific structures while trailer mid-walls tended to respond only at high frequencies. One-story camp and log structures and massive stone, concrete block and adobe structures did not respond to airblast.

Whole structure natural frequencies averaged 6.0 Hz. Mid-walls averaged 8.4 and 13.8 Hz in the transverse and radial walls, respectively. These values fell below those reported by the U.S. Bureau of Mines in RI 8507. Mid-wall motion frequencies duplicated low frequencies of the upper corner and also carried a high-frequency component. However, the range in data in this study corroborated U.S. Bureau of Mines findings.

Damping values fell well within the range reported in previous studies of 2 % to 10% of critical. Trailer transverse wall damping averaged 9.5% while log and trailer structures exhibited the highest whole structure (upper corner) radial damping of 9.7% and 9.6%, respectively. The least damped structure type was the two-story stone and measured 3.9% of critical.

Wall strains calculated from gross and mid-wall differential displacement were less than 20 μ-strains for wall bending. The maximum calculated in-plane tensile wall strain was 133.1 μ-strains and is well below cracking thresholds of 300 to 1000 μ-strains for plaster and wallboard.

Structure response to non-blasting events was measured. Human-induced whole structure responses up to 0.51 in/sec and mid-walls up to 2.14 in/sec were measured and are equivalent to ground vibration amplitudes of 0.28 in/sec for single wide trailers and 0.11 in/sec for double wide trailers and one-story adobe. Wind gusts generated air pressures that resulted in detectable levels of structure shaking and mid-wall responses in trailers up to 0.1 in/sec

Direct measurements of crack response were made for four structures. Addendum I is a report describing the measurement techniques and summarizing the long term (environmental) and transient (blast vibration) changes in crack width. Addendum II outlines protocols for implementing many of the measurement and analytical procedures described in this report.

INTRODUCTION

Explosives are used to break rock overlying a coal seam. The rock can be broken in place (conventional blasting) or broken and partially displaced into the adjacent pit (cast blasting). In any blast, the majority of energy is spent breaking rock. The balance of energy emanates from the site into the environment as either seismic or airblast energy. Once blasted, all the rock is moved to expose the coal for mining.

Ground vibrations and airblast leaving the mine eventually arrive at adjacent properties. The energy is then transmitted into the buildings. In turn the buildings respond or shake. If ground vibrations and /or airblast are strong enough, the building may be damaged. The Office of Surface Mining (OSM) and other regulatory agencies limit the amount of energy received at the building regardless of how blasting is being conducted at the mine.

Based on the research conducted to date, damage to buildings has never been observed below ground vibrations of 0.5 in/sec or airblasts of 140 decibels. Federal regulations allow limits up to a maximum vibration of 1.0 in/sec (between 301 to 5000 feet) and 134 decibels, respectively. At these limits, no damage is expected but we acknowledge that hairline cracking of plaster is possible under certain site or building conditions. The intent of the regulatory scheme, as outlined in the preamble to the federal rules and the development of a blasting plan, is for the coal mine permittee and the regulatory authority to tailor the allowable limits based on the site specific need to prevent damage to occupied dwellings. The regulatory authority is responsible for lowering the limits if necessary to prevent damage

People inside buildings can feel the structure shake and hear bric-a-brac rattle at ground vibrations and airblast as low as 0.04 in/s and 100 decibels, respectively. Citizens often begin noticing normal house changes, such as cracks in walls, and blame the changes on the vibrations they feel. To some, any type of environmental vibration is intrusive and disturbing. Since low level blasts will annoy some people, complaints are common.

The part of any residential structure most susceptible to blast induced vibrations is the superstructure or portion above ground level. Research over the years has defined the structure response characteristics of "typical" one and two story residential structures. OSM has built their regulations around this research since the majority of structures near coal mines are residential.

Occasionally, structures are found near the mine that do not fall into the "typical" category or may not have been included in the body of data on which the rules were founded. Such structures may include pre-fabricated houses, trailers, log homes, sub-code homes and adobe structures. This study measures the response characteristics of these "untypical" structures to blast induced ground vibration and airblast and compares motion characteristics to those of “typical” structures studied by the U.S. Bureau of Mines (U.S.B.M) and others in establishing the widely adopted safe level blast vibration criteria in the U.S. As such, field measurements and analyses were made to duplicate those conducted by past researchers. U.S.B.M. research primarily considered traditional wood frame housing. Therefore, it was the goal of this research to extend the understanding of similarities and differences in dynamic response between traditional wood-frame constructions and non-traditional type structures.

The motivation for this study began because of blast-related complaints from residences living near surface coal mines, despite an industry-wide adherence to safe blasting criteria prescribed for the coal mining industry. Limited investigations of blast complaints conducted by government officials revealed that certain structure types may respond to blasting vibrations in unique and unusual ways. Currently there is no uniform approach or guidelines available to investigate the uniqueness in structure response. Therefore this study was initiated to address two issues. The first was to characterize the response to blasting in various types of structures that are unlike those types that have been previously studied. The second was to develop a methodology to investigate and evaluate structures by placing traditional vibration instrumentation within structures in a manner to address uniqueness.

An important objective was to compare the responses of this study data to the data previously obtained by the U.S. Bureau of Mines as a measure of uniqueness for all structures studied. Finally, this study provided the opportunity for government personnel (GP) to take part in structure instrumentation and analysis of response data. This on-site training process is valuable to enhance understanding and confidence that GP require when investing blast-related complaints.

It is not the intent of this study to evaluate and compare the influence of blast design on ground motion and airblast excitations as a source of vibration response of structures. Furthermore, this study did not address wall cracking. No observations of crack extensions were made during structure response monitoring. Therefore, no conclusions have been made regarding the potential of specific ground motions and airbast excitations to induce cosmetic cracks in structures. Furthermore, there are no correlations of structure response with cracking potential.

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PROJECT APPROACH

Ground motion, airblast, and structural response data from surface coal mining blasting were collected at eleven mining sites. Structures instrumented in this study were selected to represent the range of structures found in the proximity of surface coal mining with focus on those not previously studied by the U.S. Bureau of Mines during structure response studies. These designs included pre-manufactured trailers, log, earth and stone, and mine "camp". Time-correlated measured responses include those of whole structure, mid-wall, and selected structural components. Responses include those from human activities, environmental effects, and surface mine blasting.

A crack response study, supported by Northwestern University, was conducted in parallel to the structure response study within structures possessing a representative hairline drywall, plaster or concrete block crack. Transient displacements of the crack from blasting were compared to static crack movement produced from long-term changes in environmental climate conditions. Results of this crack study are found as an Addendum I to this report and titled "Direct Measurement of Crack Response Study of Four OSM Study Structures". The monitoring of existing cracks within selected structures was neither part of the scope of work for this project nor was it required by the Office of Surface Mining. However, it was felt that a crack study, would provide another basis for understanding the manner in which structures respond to human habitation, environmental effects and blasting.

SITE AND STRUCTURE SELECTION

Eleven coal mining sites were selected by OSM based on recommendations of state personnel. These states included Virginia, Kentucky (two sites), West Virginia (two sites), Tennessee, Alabama, Ohio, Pennsylvania, New Mexico (representing Native American Indian lands), and Indiana. State blasting specialists nominated coal mines, based on structure uniqueness.

Criteria for the selection of structures had to satisfy study objectives and facilitate project tasks within limited time constraints and resources. These criteria included structure uniqueness, the proximity of the structures to the mine blasting site(s), willingness of home owners to cooperate on the project, and availability of a significant number and intensity (e.g. amplitudes of ground vibrations and airblast) of planned mine blasts to ensure measurable structure response, and the cooperation and assistance of the mine operators.

Specific selection criteria for structures included the following:

- Structure uniqueness

A minimum of one "atypical" structure was needed at each mine. At some sites, traditional wood-frame structures were selected based on availability and satisfaction of all other criteria. Incorporation of a limited number of traditional word-frame structures provided a basis of comparison responses with those of previous research and those of unique structures selected at the same mine site.

- Proximity to an active surface coal mining operation

To satisfy project objectives, sufficient blast-induced ground vibration and airblast energy was necessary to produce measurable vibrations and structure response. Therefore, the blast site distance to structures and the explosive charge weights (e.g. maximum charge weight detonated on one delay or within an eight millisecond, ms, delay interval) were important parameters to consider in site and structure selection. It was important that at least five blasts be detonated during the week monitoring to facilitate scheduling constraints and instrumentation requirements. Mine operations generating significant levels of ground vibrations (e.g., averaging 0.25 inches per second, or in/sec) and airblast (in excess of 115 dB) over a wide range of scaled distance factors were considered to be sufficient for the structure response study. Coordinating project logistics around five planned mine blasts one to two months ahead of site arrival provided challenges that were overcome by the cooperation of mine operators.

- Cooperation of the homeowner

Owners of structures that satisfied the criteria were provided written documentation describing the study. Home owners willing to participate were asked to sign a right of entry (required by OSM) and a release of claims (required by the contractor).

- Cooperation of the mine operator

Site scheduling was dependent on mine blasting activities near the homes. Mine operators were contacted by agency personnel and the contractor to coordinate study activities during specific weeks. Additionally, mine operators were requested to supply information on the location of blasts and proposed charge weights. In cases where five blasts were not possible during one week, an attempt was made to separate large blasts into smaller blasts or provide a few single hole detonations. In a few cases, less than five blasts were provided. However, redundancy in structure types among sites and greater numbers of blasts at other sites provided a sufficiently large data base to meet study objectives.

DESCRIPTION OF STRUCTURES

Structures were characterized and construction details were documented in a number of ways. Photographs were taken of each structure exterior and interior as well as the foundation (for non-slab foundations and where access was available). Specific attention was given to the type of foundation support. Laser-level surveys were conducted to establish floor elevations for all structures and room dimensions were measured with a laser rangefinder. This information was used to assess the overall condition of structures that might be a function of foundation support, distribution of structure load, as well as unusual structure loads or other construction details.

Appendix 1 provides detailed documentation of each structure. Included are scaled room layouts and photographs of various features. Room measurements were necessary to compute gross strains within structure walls.

Table 1 presents general construction details of all structures in this study. Structures are identified by state and location in the order in which they appear in Appendix I.

Structure designs include the following categories:

- pre-manufactured trailers constructed as single wide, double wide, and wood-frame add-on support by concrete masonry units (CMU, or cinder blocks),
- log structures – one and two story traditional natural log and two story prefabricated, manufactured log structures with vaulted ceiling living areas
- mine "camp dwellings" constructed of wood frames with diagonally sheathed walls and foundations of perimeter CMUs and interior log poles
- masonry and earth - construction includes CMU's, field stone and adobe, and traditional adobe
- traditional wood-frame structures - including one, two, and three story (cantilevered) designs

A brief description of each structure is given below. For clarity the following designations were used in identifying the structure category:

T – trailer	S – single-wide trailer
C – camp	SA – single-wide trailer with add-on
L – log	D – double wide trailer
E – masonry and earth	1S – one story
W – wood frame	2S – two story
	3S – three story

The designations following the structure category used to identify the states and mines (in alphabetical order) are:

AL - Alabama
IN - Indiana
KY1 – Kentucky site 1
KY2 – Kentucky site 2
NM – New Mexico
OH - Ohio
PA - Pennsylvania
TN - Tennessee
VA - Virginia
WV1 – West Virginia site 1
WV2 – West Virginia site 2

If two structures of the same category and design were selected, the following identifier was used:

A – first structure of category and design
B – second structure of same category and design

Pre-manufactured Trailer Structures

Pre-manufactured trailers ranged from small, single wide units 64 ft. long by 14 ft. wide to large double wide trailers 74 feet long by 28 feet wide. Single wide trailers with wood frame add-ons were 54 to 46 ft. in length and 24 to 26 ft. wide. All trailer interior walls, with the exception of one double wide, were constructed of wood fiberboard coated with a thin layer of plaster compound. All walls were covered with wallpaper or wood paneling.

One double wide trailer possessed a recently constructed wood frame and drywall interior wall separating the dining area from the kitchen parallel to the "marriage" wall (e.g. long trailer axis). This was the only trailer founded on a full basement.

All other trailers rested on piers of unmortared concrete blocks that were leveled with wood wedge shims. Pier support geometries for single wide and double wide trailers are shown in Figure 1. Some trailers were fastened to the ground using perimeter hurricane strapping shown in Figure 2. Concrete blocks were stacked singly or in pairs and placed beneath steel beams as shown. Wood shims were placed between the pier and trailer beams in all cases. Piers for one trailer were supported on poured concrete pads. The remaining trailer piers were founded directly on the soil.

A number of piers were tilted from a vertical line and not aligned normal to the steel beams. Tilting piers are shown in Appendix I for all trailers with the exception of TD-TN (Note, TD-PA is founded on a full basement). Tilting results from eccentric loading about the pier support.

<u>TS-KY2</u> is a single wide trailer with interior paneled walls. The single CMUs were configured as shown in Figure 1 (a). No hurricane strapping was used.

<u>TS-IN</u> is a single wide trailer with a small room addition at the east end founded on a stack of single CMUs configured as shown in Figure 1 (a). Hurricane strapping was used and all interior walls were paneled.

<u>TS-AL</u> is a single wide trailer with hurricane strapping. Double concrete piers support beams as shown in Figure 1 (a). Interior walls were either paneled or papered.

<u>TS-OH</u> is a single wide trailer with loose hurricane strapping. The trailer was built into a hillside and supported by varying pier heights ranging from a single half-block to a double set of five blocks in the configuration shown in Figure 1 (a). Interior walls were either paneled with wood or covered with wallpaper.

TSA-VA is a single wide with a wood frame add-on along the entire back of the house. The original trailer section is supported with double CMU piers while the wood frame add-on is supported by a conventional CMU perimeter wall. A one by eight sill plate supports floor joints and does not support the trailer section cross members. All interior walls have wallpaper covering or were paneled. No hurricane strapping was used.

TSA-KY2 is a single wide with a wood frame add-on along the entire front of the structure. A CMU wall exists around the entire perimeter. Beneath the trailer section, it serves as a skirt. Beneath the addition, it supports the frame. All interior piers were double concrete blocks. The wood-frame section is not supported with a perimeter wall and supported only with double concrete blocks. The support configuration is generalized in Figure 1 (b). No hurricane strapping was used.

TD-WV2 is a two-year old double-wide trailer. The support configuration is generalized in Figure 1 (c) with one single width stack of CMUs placed along the "marriage" wall beam. The piers were founded on poured concrete pads. Standing water from a bathroom water leak was noted under the northwest corner of the trailer. No hurricane strapping was used and all walls were covered with vinyl wall covering.

TD-TN is a two-year old double wide trailer with hurricane strapping. Double CMU piers were used in the corners and along the "marriage" wall beam. Single CMU piers used for all other beams along the perimeter as shown in the configuration of Figure 1 (b). All interior walls have wallpaper covering.

TD-PA is a double wide trailer with a full basement constructed of CMUs. The center steel beam carrying the "marriage" wall was cut to accommodate the stairway into the basement from the laundry room. This main beam is supported by steel posts, spaced on 12-foot centers along the trailer long axis. CM walls support cross-beams. All interior walls were wallpapered. The newly constructed wood-frame wall between the kitchen and the dining area is completed with drywall.

Mine Camp Structures

Mining camp houses ranged in age from 50 to 100 years old and construction widely varies. Exterior walls were constructed with two by fours placed at right angles to current wood frame construction practices. Shown in Figure 3, the four inch dimension of the studs is oriented parallel to the wall. Diagonal exterior boards complete the framing. Traditional camp houses in central Appalachia are supported on interior log poles, many of which are founded directly on bedrock. Others are supported on both logs and CMU piers. Floor joists rest on perimeter walls without sill plates and are randomly located rather than uniformly spaced. Other mine camp structures are supported on a mix of wood poles and concrete blocks or bricks. Perimeter foundations comprise a variety of fieldstone, CMUs, and poured concrete with rectangular wood post framing. A number of camp structures have been renovated by replacing stone foundations and adding modern wood-frame rooms.

C1S-AL is a one story mining camp structure approximately 55 years old. The frame construction rests on a CMU perimeter wall and interior piers of unmortared single concrete blocks or clay bricks. The interior walls of the house were paneled with a wood product. The living room has new sheet rock walls.

C1S-VA is a one-story structure built in 1945. The home is founded on bedrock using wood log posts. The exterior perimeter wall is constructed partly of field stones (front of the house) and cement block at the rear of the structure. Irregularly spaced floor joints do not form any particular pattern and rest directly on top of the perimeter concrete blocks with a sill plate formed of concrete. A number of log posts were found to be loose and not tied to the floor joists. All interior walls were paneled.

C2S-KY1A is a two-story camp home built in the early 1900's. Interior walls were plaster on lath covered with paneling throughout the house. Basement ceiling joists vary in spacing and were supported by log posts. Discontinuous two by eights were used to support the joists in many places. Basement walls were formed using field stone and mortar.

C2S-KY1B is a two-story camp home built in the 1950's with two additions. The rear addition forms the kitchen and a bathroom and a recent addition forms the living room. The older section of the structure is founded on a full basement while the additions are built upon a crawl space. The structure is supported with a perimeter concrete block wall and interior supports of many varieties. Interior supports include unmortared concrete block piers, wood posts, table legs, and a steel jack. Interior walls were newly constructed drywall or paneling.

Log Structures

The five log homes in this study were constructed of horizontally laid logs fitted together by one of the three techniques: the saddle lock-notch, notched and scribed, and butt-jointed. Figure 4 shows the three types of log fittings used to construct the homes. Four of the structures combine corner notching, either the saddle lock-notch or notched and scribed, and the log weight is used to form stable structures. The remaining house was built using butt-joints throughout the structure. At the structure corners, log ends were nailed perpendicular to each other. The butt-joint combined with the log weight formed a stable structure.

The logs with a saddle lock-notch were stacked such that they do not rest against each other except at the notch leaving a crack or "chink" of one inch or more visible between the logs. Chinks allow for warping and expanding. The chinks were filled or caulked with a plaster or mud material. Scribing a log is the terminology used to describe fitting the entire length of the log to match the shape of one log to another. Scribed logs were notched at each end and a tongue or groove is cut from notch-to-notch the length of the log. The tongue and groove serves as a means of tightly fitting the logs together. The butt-joint technique does not require notching to stabilize the logs. Two logs were joined by placing one log perpendicular to one end of the other log and nailing the two together. The normal stabilization method for butt-jointed logs involves drilling vertically through the stacked logs of a wall and driving rebar down through the drilled hole to stabilize the wall.

<u>L1S-OH</u> is a one story log cabin with a full CMU block wall basement. The structure is 40 years old. Walls comprise hand-crafted milled logs, approximately nine inches in diameter, were notched and scribed.

<u>L1S-WV1</u> is a one-story primitive handcrafted log cabin constructed more than 100 years. The construction is called primitive because the bark was not removed from the logs. The original part of the structure was built from hand-hewn logs that were saddle lock-notched and horizontally stacked. The chink was caulked with a mud or plaster type material. The logs were approximately six inches in thickness with additional six inches of framing on the inside for a total wall thickness of 12 in. Interior walls have a plaster finish. The original cabin sits on concrete piers at the corners. A concrete block foundation was added under the front porch of the cabin and under an addition at the rear.

<u>L2S-TN</u> is a two-story handcrafted cabin built using a butt-joint technique for the wall construction and corners. The logs were railroad cross ties cut six inch by six inch square and joined end-to-end with length of a wall with a two by six board nailed along the top of the joined cross ties. The cabin walls stand only under the weight of the logs. No vertical structural supports or ties (e.g., rebar) were used to vertically tie logs together. The foundation comprises a CMU perimeter wall and interior block piers forming a two to three foot crawl space founded directly on bedrock.

<u>L2S-OH</u> is a modern mill-log custom home designed and built by the owner. It is approximately 2 years old with a full cinder block basement. The vaulted ceiling in the living and dining rooms were constructed with roof trusses and exposed beams and rafters. A partial second floor is designed over one-half of the structure.

<u>L2S-WV2</u> is constructed from a log home kit with modern mill-logs, a vaulted ceiling with exposed breams, rafters and trusses. A partial second floor is constructed over one-half of the structure. The structure is founded on a crawl space with a cinder block perimeter wall and interior piers of concrete block. A single post supports a balcony and the roof beam overlooking the living area.

Masonry and Earth Structures

Masonry and earth structures include concrete block, stone, and adobe brick (stabilized from hardened soil blocks, baked in the sun) faced with stucco. Three structures falling in this category were located in New Mexico. Consistent with construction practices in the southwest, houses were founded on concrete slab or directly on the ground with stone perimeter beams supporting bearing walls.

<u>E1S-NMA</u> is a one-year old cinder block building founded on a reinforced eight-inch thick concrete slab.

<u>E2S-NM</u> is a two story stone (field rock with cement joint grout) structure built in 1880 with interior adobe walls. The stone exterior walls comprise two layers of sandstone block and mortar without wood framing or a bond beam to tie the exterior stone walls together. The mansard roof rafters rest on two, two by eight inch headers lying on top of the stone walls. There are no nailed connections between the roof and the structure wall. Interior walls on the first floor are covered with structural plaster. Exterior stone and interior adobe walls rest on a rock wall foundation.

E1S-NMB is a 17-year old single story traditional adobe structure. Exterior walls were covered with stucco while interior walls comprise exposed adobe bricks. The house is founded on a four inch concrete slab.

Wood-frame Structures

Wood-frame structures represent "typical" construction akin to structures previously selected by the U.S. Bureau of Mines. All wood-frame structures were founded on full basements.

W1S-IN is a one-story wood-frame structure with a full basement of CMU wall construction built in the 1950s.

W1S-PA is a newly constructed one-story wood-frame house with a full basement of CMUs.

W2S-IN is a house that was recently purchased by the mining company prior to mining through the property. It has a concrete block full basement and a partially completed attic. The structure age is unknown.

W3S-WV1 is a three-story structure founded on a concrete slab. The first story, constructed of CMUs, serves as a shop. The second and third stories were of wood-frame construction of perimeter dimensions four feet wider than the first floor.

INSTRUMENTATION

Whole structure and mid-wall responses were recorded with single axis velocity transducers attached to four-channel blasting seismographs manufactured by LARCOR, of Dallas, Texas. A connector interface box linked transducers to the seismograph, which allowed the air channel to be employed to record velocity. Three seismographs, one exterior (master) and two interior (slaves), were daisy-chained together to record ground and structure motions with a common time base. The master was set on trigger mode and the two slaves were set on manual mode. When triggered, the master unit sent a one-volt spike to the slave units to simultaneously start data recording. A tri-axial transducer buried in the ground and microphone recorded three components of ground motion and airblast at each structure exterior.

Interior transducer output was amplified by a factor of 2 (e.g., lowest detection level of 0.005 inches per second, in/sec All three seismographs were programmed to record 6 to 12 seconds of event time at a sample rate of 512 per second. The master unit was programmed to trigger at a ground particle velocity of 0.02 to 0.03 in/sec and the maximum range for all units varied from 2.5 to 10.0 in/sec depending on blast-to-structure distance and gain selected.

Polarity Testing of Velocity Geophones

Polarity was checked for each geophone prior to deploying instruments in the field. When evaluating differential motions between the ground and structures, it is important to document the polarity of the geophones. For instance, polarity for a vertical sensor normally produces a positive phase first motion. If the polarity of a structure-mounted vertical sensor is such that the first motion is negative while a ground motion sensor vertical component produces a positive first motion, it is likely that the structure sensor polarity is reversed.

Polarity becomes critical when measuring and comparing relative motions between the ground and upper portions of structures, particularly when differential displacements are to be calculated in order to estimate gross structure strains and in-plane wall strains. If sensors are mismatched, differential displacements may be over two times greater than displacements for a common polarity.

Sensor Locations within Structures

Typical instrumentation placements for many of the structures are shown in Figures 5 and 6. Horizontal sensor orientations for common polarity are found in Figure 7. The radial alignment of sensors placed in the ground and within structures was directed along the long axis of each structure. Efforts were made to place the ground R component in the same direction as positive (inward) wall and structure motions. Sometimes the position orientation of the radial ground sensor was placed in a direction opposite to that of the structure or mid-wall orientation. This opposite polarity was easily recognized and compensated during analysis.

Specific locations of exterior and interior geophones, and the seismograph unit serial number to which they were connected, are illustrated in the structure plans in Appendix II. Interior sensors S1 and S2 consisted of four single-component velocity transducers, three mounted to record horizontal motions and one mounted to record vertical motion. A sensor cluster (two horizontal and one vertical) was placed at the first floor structure corner base (S1) and a duplicate cluster (S2) was placed at the highest point of the same corner. Motions recorded at S1 and S2 were used to measure the whole structure response to blasting. Mid-wall response was measured using a third horizontal sensor, placed at or near the middle of each conjoined wall (shown as wall 1 and wall 2). At S1 and S2, the R sensor was aligned with the longest axis of the structure and T with the shortest axis, as shown in Figure 5 (b). The vertical, V, sensor was placed on either wall. Figure 6 shows a typical instrumentation set up for a one-story mining camp structure

Other instrumentation layouts, specific to a unique construction type, did not adhere to the typical layout shown in Figures 5 and 6. In most cases, the lower structure vertical response reflected the ground vertical vibrations. Therefore, in some structures the vertical component normally placed at the lower was placed on a ceiling or other more useful locations. Sometimes, motions between two or more construction components were monitored. Special layouts were used for double wide trailer TD-TN (where opposite sides of the “marriage” wall were instrumented), single wide trailers TS-AL and TS-OH (measuring torsional motions at opposite ends of the trailer), and between two different construction types in TSA-VA. Motions were also measured in log structures along the "great" wall at the end of a vaulted ceiling room by placing single transducers at the roof peak, L2S-TN, L2S-OH, and between the roof beam, rafters and center post, L2S-WV2. In two structures, the vertical motions of the ceiling were measure (E2S-NM, TS-IN) rather than wall vertical motions.

RESULTS

The focus of this study was to characterize the response of atypical structures to blasting vibrations and airblast generated from surface coal mines. The uniqueness of structure design was addressed by comparing vibration response characteristics with characteristics measured by the U.S. Bureau of Mines and others during previous studies using traditional design structures.

A total of 25 structures were selected for this study at 11 mine sites. Twenty-one structures represented non-traditional designs and four structures comprised traditional wood-frame construction. Ninety-nine mine blasts were conducted during response measurements and 2824 velocity time-histories were recorded and analyzed.

The results of this study are organized in two sections. The first section illustrates the characteristics in mine site blast vibration and airblast generation and attenuation. The second section provides the results of structure response, comparing the relative whole structure and mid-wall motions as well individual structure response relative to external ground vibrations and air overpressures. The response of structure motions relative to ground motions were evaluated in terms of amplification factor as defined by the U.S. Bureau of Mines (Siskind, et al, 1980a) and compared to amplification factors found for traditional structures. Fundamental (or natural) structure frequencies and damping characteristics were evaluated for structures only when significant ground motion and air overpressure intensities were generated. Maximum gross structure and wall strains were calculated based on whole structure differential displacements and mid-wall displacements integrated from velocity time histories. Lastly, the influence of airblast on certain airblast-sensitive structure designs was evaluated.

In each evaluation, data processing and analysis procedures are explained. Data are summarized in table format and selected data are plotted in figures for comparisons. All sensor records are available in electronic format

Summary tables for all sites are given in Appendix III. Data in these tables include the following:

- Blast date and time
- Maximum charge weight per delay and blast-to-structure distance
- Calculated scaled distance (square- and cube-root)
- Ground motion and airblast measurements
    - maximum velocity for each of the three components of ground motions
        - (T, transverse, V, vertical and R, radial)

peak particle velocity (PPV, in in/sec), the highest of three components
peak frequency (Hz) for three components (zero-crossing frequency)
Fast Fourier Transform (FFT) predominant frequency (Hz) for three components
airblast, in decibels (dB)

- Whole structure response, single components
  maximum velocity (in/sec), peak (zero crossing) frequency (Hz), and
  FFT frequency (Hz) for the R, V, and T components at either
    S1 (lower corner) and S2 (upper corner)
    S1 (lower corner) and S2 (upper peak or highest point in the structure)
    S1 (lower wall) and S2 (upper wall) for interior or exterior walls
    a variety of locations throughout the structure for conjoined components
- Mid-wall response, single components
  maximum velocity (in/sec), peak (zero crossing) frequency (Hz) and FFT
  frequency (Hz) for the radial (R) and transverse (T) walls

Mine Site Characteristics

Table 2 summarizes the ranges in values for blast-to-structure distances, maximum charge weight per delay and square root scaled distance factors. The total number of mine blasts and number of structures instrumented per site are given. Scaled distance factors ranged from 23 ft/lbs$^{1/2}$ in New Mexico to 340 ft/lbs$^{1/2}$ at Kentucky site 2. Blast-to-structure distances ranged from 570 ft. in Ohio to 9219 ft. in Indiana. The maximum charge weight detonated per delay among all sites was 13,047 lbs. in New Mexico while the smallest of 126 lbs. was used in Indiana and West Virginia site 1.

Ground Vibrations and Airblast Measurements

Ground Vibration Attenuation Plots

Attenuation plots of peak particle velocity versus square root scaled distance (SRSD) are shown in Figure 8 for all blast data. Figures 9 and 10 are attenuation plots for surface coal mine sites by state. Best-fit lines (50-percentiles) through site data with a sufficient range in scaled distance and a statistically significant data set to allow trend analysis are shown in Figure 9. Included in Figures 9 through 10 is the best-fit line given in Report of Investigation (RI) 8507 by the U.S. Bureau of Mines (Siskind, et al. 1980a) for the maximum horizontal component of ground motion for all coal mine data. Equations and correlation coefficients (R^2) for these lines are found in Table 3. The equations were fit to the PPV. Data for sites included in Figure 10 were not correlated. This is because either data represented a narrow range in blast-to-structure distances and charge weights, the data was highly scattered, or a limited number of blasts were conducted to produce a significant data set for correlation purposes.

Central Appalachia data in Figure 10 show a clustered set of similar scaled distances in Virginia and in West Virginia at site 2. Blasting at the remaining sites was conducted at various scaled distances in a number of different compass directions from structures. As such, data trends are not apparent and a narrow spread in ground motion values was recorded below 0.1 in/sec (98.5% of the data fell below 0.1 in/sec).

Interestingly, the New Mexico site generated data for both unconfined (casting) and highly confined (pre-split) as shown in Figure 9. The data fell with two distinct groups and the effects of greater confinement provided by pre-splitting blasting techniques resulted in far higher ground motion amplitudes compared to those produced from casting blast at a given scaled distance. Charge weights per delay for pre-splitting averaged 300 lbs/delay and for casting blasts, charge weights averaged 13,000 lbs/delay.

Equations describing the attenuation of ground motions, shown in Table 3, are compared with those provided by the U.S. Bureau of Mines for surface coal mines (Siskind, et al., 1980a). Site-specific data presented in the current study show a good degree of data correlation for the Alabama, Indiana, and New Mexico sites and scaled distance slope exponents (-b) ranging from –1.34 in Indiana to –2.22 in Alabama. The intercept or source term, ‘a’, varies from 64 in Indiana for highwall blasts with high relief (e.g. long delay periods along the face) to 5448 in New Mexico for highly confined pre-split blasts. The source term is a good indicator of explosive energy coupling at the blast site. Average values for data parameters ‘a’ and ‘b’ are slightly higher than values reported for coal mine data by the U.S. Bureau of Mines summarized in RI 8507, where ‘b’ is –1.52 and ‘a’ equal to 119 for all components of ground motion. This difference may indicate the presence of higher attenuating geologies at the current study sites in comparison with the U.S.B.M. sites.

Airblast

Airblast overpressure attenuation is given in Figure 11 for cube root scaled distance (CRSD) showing 50-percentile best-fit lines. Table 4 summarizes the best-fit equations in comparison with equations given by the U.S. Bureau of Mines (Siskind, et al, 1980b). The U.S. Bureau of Mines equation for highwalls shows a source term ‘a’ of 0.146 and ‘b’ equal to –0.823, R^2 of 0.77. The data for all sites compare favorably with past U.S. Bureau of Mines data.

Frequency Content of Ground Motions

Measuring Frequencies

Previous research has produced frequency-based velocity data without a clear definition of frequency or methods used to calculate frequencies. Frequency components of a vibration are equally important as the particle velocities. When the intent is to evaluate damage potential, the entire time history, or all frequency component, is an important factor to consider.

Frequency is most reliably computed by applying the Fourier frequency function, or FFT (Fast Fourier Transform), to transform the ground motion time histories (time domain) into the frequency domain. In this manner, the distribution of frequency content can be compared based on relative intensities of ground motion at specific frequencies, and predominant frequencies can be easily identified.

In contrast, the “zero-crossing” method has been widely adopted by industry for determining and reporting a single frequency value at the peak velocity of ground motions measured in three directions (R, T, and V), or the PPV. Current industry practices employ this "zero-crossing" frequency at the PPV to determine compliance with frequency-based limits (referred to henceforth as the peak frequency). A problem arises when the peak frequency occurs in a complex vibration time history containing a variety of frequencies and amplitudes. If the peak velocity occurs early in the time history within the high frequency components (e.g. above 20 to 30 Hz), the zero-crossing method may result in a frequency well above the natural frequency range of residential structures, even if the entire time history contains a strong low-frequency component. This peak frequency may not represent the frequency at which the maximum vibration energy is transferred into the structure. Most seismograph analysis software provides a means to plot the "zero-crossing" frequency for every peak contained within the time history. In this respect, the vibration energy contained over all frequencies can be evaluated with respect to potential structure response.

Measured Vibration Amplitudes and Frequencies

Peak particle velocity (PPV) data versus frequency are plotted in Figures 12 and 13. The upper bounds are shown for safe level blasting criteria recommendations reported in U.S. Bureau of Mines RI 8507 (Siskind, et al, 1980a) and Office of Surface Mining (1983). Frequency in Figure 12 is the peak frequency at the PPV while in Figure13, it is the predominant frequency calculated from the power spectrum of the Fast Fourier Transform (FFT).

Table 5 summarizes site-specific differences in frequency ranges calculated by the "zero-crossing" (Z.C.) and FFT methods. In all cases, with the exception of Tennessee, Z.C. frequencies at the PPV are higher at the upper end of the range compared with the FFT method. The change in the highest frequency within the range is most dramatic at five sites (Kentucky 1, New Mexico, Alabama, Kentucky-2, and Indiana) with upper Z.C. frequencies from 18 to 34 Hz and upper FFT frequencies less than 20 Hz. The remaining sites did not show such a large difference. The Tennessee site FFT frequencies actually increased over the Z.C. frequency. This increase is probably because the structure foundations rests directly on bedrock and measured ground motions were recorded within the thin, overlying soil layer where high frequencies were preserved.

Since the FFT method accounts for the entire wave train, it is preferred for structure response analysis. FFT is closely related to response spectra of ground motions and are employed to calculate structural natural frequencies and damping from structure motions.

Summary of Findings

These observations serve to illustrate a number of important points as follows:

- Different site characteristics, particularly structure site geology and blast-to-structure distance, produced different frequency content. Structure distances ranged from 570 ft. to 6280 ft. from the blasting. Certain structures such as those in Tennessee were founded directly on bedrock while others (in New Mexico) were founded on thick soils. Sites with different foundation materials produced a spread in ground motion frequencies while sites with similar geology produced a concentration of data within a narrow frequency range.

- At all but one mine site, FFT frequencies fell below "zero-crossing" frequencies and within the natural frequencies of structures for walls (12 to 20 Hz) and superstructures (5 to 10 Hz) reported by Dowding (1996).

- The Z.C. method employed to calculate frequencies were generally above those computed using the FFT method when only the peak velocities were analyzed.

- Frequencies calculated using the FFT method is prefer since they involve the full wave form and are a more conservative estimate of the dominant excitation frequency.

- Airblast attenuation was similar to that observed by the U.S. Bureau of Mines.

- Peak particle velocities for Appalachian coal mines were consistently below mean values predicated using scaled distance by the U.S. Bureau of Mines in RI 8507 for all coal mines with the exception of Pennsylvania. This is because mining in Appalachia is conducted at elevations higher than those of structures and well behind slope berms. As a result, PPV values are highly attenuated.

- Pre-split blasting consistently shows PPV values well above the mean for coal mining in RI 8507.

Structure Response

The measured response of structures to blasting vibrations and airblast are important to assess damage potential to individual components of the building. The amount of structure shaking is a function of the amplitude and frequency content of external ground velocity and airblast overpressure and the natural frequency and damping characteristics of the structure. Horizontal components of ground velocities are often amplified in structures while the highest structure velocities are measured when the ground frequency occurs at or within the structure's natural frequencies. The amplification of structure response relative to external ground vibrations is an important factor when assessing blast damage potential.

Two modes of structure vibrations occur during blasting and are referred to as mid-wall and whole structure responses. Mid-wall response is the motion of individual components such as wall, floors and ceilings, where motions are perpendicular to the plane of the building component. Mid-walls generally respond at high frequencies and tend to rattle windows and loose objects attached to walls. Resulting bending strains tend to be the greatest when the walls respond at their natural frequencies.

Whole structure response is vibration of the entire structure frame, measured at an outside corner, resulting in distortions, or racking, in walls. At low frequencies and high amplitudes of ground motions, whole structure deflections produce wall shear strains that, in turn, may be potentially damaging. Structure deflections are measured in terms of differential displacements between the upper and low (ground) corners in structures.

Time History Comparisons: Structure Response Relative to Ground Motions and Air Overpressures

Structural response (SR) to ground velocity and air pressure (airblast) are shown for (S2) upper structure corner locations or wall peaks, in rooms with vaulted ceilings and (S1) lower structure corners, at the base of the first floor wall, and mid-walls in Appendix IV. Ground velocities (GV) and air pressure (AP) are shown for comparisons. Peak values for velocities and airblast are provided. Superimposing excitation and structure response waveforms provides a visual means of evaluating the energy transferred into the structures over time. It further allows visual evaluation of structure or mid-wall free response after passage of the ground and air pressure pulses. Horizontal components of velocity were selected for comparisons. The maximum structure velocity in either the radial or transverse component is shown in Appendix IV figures, depending on the peak occurring within the structure.

Vertical components were only evaluated for manufactured (trailers) structures where structure response vertical motions were amplified. For all other structure designs, negligible differences among the lower and upper structure responses relative to ground vertical motions could be detected. Vertical structure motions within most structures duplicated ground vertical components in frequency, amplitude, and phase.

All vibrations are plotted in terms of velocity, in inches per second (in/sec). Vertical scales are not given and may vary between figures. However, among waveforms being compared in any one figure, constant vertical scales are used. Air pressure (AP) vertical scales are consistent among all plots.

Waveform time histories are expanded in time to illustrate similarities or differences in amplitudes, frequencies, and phases. Phase refers to the positive and negative pulse shapes forming the sinusoidal characteristics of a waveform. Vibrations of structures that are well-coupled to the ground may show good time history in-phase match with ground motions. However, when ground motion exhibit frequencies close to the natural frequency of the structure, structure vibrations are amplified and exhibit a near 90-degrees phase shift from the forcing or excitation motions.

Structure designs used for comparisons include manufactured (trailers), log, camp, earth, stone, and masonry. Responses of standard wood-frame structures are not shown as responses do not show uniqueness beyond what other structure studies show.

Figure IV-1 compares ground motions with those at the structure base (S1). Figure IV-2 shows comparisons between S1 and S2, the upper structure response. In Figures IV-3 through IV-6, ground and S2 motions are compared relative to air pressure time histories. Air pressure time histories are plotted with mid-wall and S2 structure responses in Figures IV-7 through IV-10 to show the airblast effects of whole structure and wall responses.

Ground motion versus lower structure response: Lower structure horizontal responses (S1) are generally equal to or lower in amplitude than the same component of ground motion for all structure design with the exception of trailers. Trailer structure base motions for single wide and double wide trailers shown in Figure IV-1(a) can exceed those of the ground except in the case of trailers with wood-frame add-ons (TSA-KY2). This is observed also for camp structures to a less extent in Figure IV-1(d). One-story traditional log structure base response given in Figure IV-1(b) and earth, stone, and masonry structures shown in Figure IV-1(c) often fell well below motions in the ground.

Vertical components of ground and S1 velocities are superimposed in Figure IV-1(e) to show the amplification of vertical motions in single and double wide trailers. Vertical trailer responses are amplified because trailers are not coupled to the ground and are free to bounce. Furthermore, the tendency of trailers to rotate around the long axis (radial direction) in the transverse directions can often translate a portion of this response in the vertical direction, resulting in higher vertical response than would be predicted by ground motions. This type of structure response is unique to trailers and was not measured in other structure designs.

Lower structure response versus upper structure response: Differential horizontal motions, or the difference between upper structure response, S2, and lower structure response, S1, induce whole structure strains in walls from racking distortions. Computing differential displacements, by first integrating the velocity time histories and subtracting S1 from S2 over time, allows the best estimation of strains.

A visual comparison of relative horizontal motions between the upper (S2) and lower (S1) walls of structures is shown in Figure IV-2. A good agreement of velocity time histories for most structure designs exists with the exception of log structures, shown in Figure IV-2 (b), and the two-story camp structure (C2S-KY1A) in Figure IV-2(d). All trailer motions showed good phase agreement (e.g. time history peaks and troughs matched in frequency). Motions in adobe (E1S-NMB) and concrete block (E1S-NMA) structures given in Figure IV-2 (c) show good phase agreement and amplification of S1 motions in the upper structure (at S2). The two-story stone structure (E2S-NM) did not show good phase matching.

Log structures, regardless of design, do not show similar upper and lower structure responses. Motions do not match in peaks while two-story designs show amplification of the upper response that is absent in one-story designs. This is to be expected because log structures are not constructed with a frame and the upper and lower horizontal log members move independently.

Ground and air pressure time histories relative to upper structure response: Upper structure (S2) response relative to ground motions and air pressure (or the pressure equivalent of airblast) are shown in Figures IV-3 through IV-6. Structures used to illustrate air pressure effects were subjected to airblast levels at or above 116 decibels (dB) (with the exception of camp structure C2S-KY1A). Single wide trailer responses (Figure IV-3) are less sensitive to ground vibrations than to airblast pressures. The airblast phase of structure response shows higher S2 amplitudes than for the ground motions phase for trailers TS-KY2, TS-IN, and TSA-KY2 with a wood-frame add-on. Airblast influence is not as apparent in double wide trailer TD-WV2 because the instruments used to measure whole structure response were placed along the interior center (marriage) wall. Note that the ground and S2 responses are approximately 90-degrees out of phase (where structure peaks lag behind peak in the ground motion) indicating that the deformation response of the structure is at a maximum.

Airblast excitation of whole structure response is apparent in the two-story log structures shown in Figure IV-4 (L2S-WV2 and L2S-TN) and is not as noticeable in one-story log, camp, earth, and masonry structures. The two-story stone structure E2S-NM, shown in Figure IV-5, was responding at the natural frequency by the time that the air pressure arrived and shows not additional response. This is evidence again by the phase shift in S2 response relative to the ground motion.

Mid-wall and upper structure response to air pressure: Mid-wall and upper structure (S2) motions shown in Figures IV-7 through IV-10 are compared with airblast arrival. Mid-wall motions show both high frequency and low frequency characteristics for log, camp, earth, stone, and masonry structures while trailer mid-walls responded only at high frequencies. Of the log structures for which mid-walls were measured, only L2S-OH mid-wall duplicated the low frequency peak S2 response. This is because the wall measured was the “great wall’ in the living room with a vaulted ceiling containing a massive stone chimney. Therefore, the mid-wall and upper peaks tended to move as one unit. This response was also observed in the two-story stone structure E2S-NM in Figure IV-9. The absence of high frequency components in the upper story mid-wall shows the strong influence of the whole structure motions on the massive stone mid-wall, indicating that the mid-wall moved in concert with the structure and not independently.

The one-story log structure L1S-WV1 did not show detectable mid-wall response to airblast (Figure IV-8). Similarly, the influence of air pressures is not significant for earth, stone and masonry mid-walls given in Figure IV-9. One-story adobe and concrete block structures also showed a correspondence in motions between upper structure and mid-walls. However E1S-NMB responded with both high and low frequencies.

Trailer mid-wall response is similar to the low frequency whole structure response with high frequencies superimposed. The large difference in exterior wall mid-wall response from S2 response for TD-WV2 given in Figure IV-7 is because S2 was measured on an interior wall and mid-wall response is shown for an exterior wall.

The mid-wall response of the one-story camp structure in Figure IV-10 is typical of motions for loose surface covering such as wood paneling in a thin-walled structure. In this case the mid-wall shows a large amplification over the upper structure response because of the loosely nailed paneling on this exterior wall to which the motion sensor was attached. The mid-wall response therefore is not necessarily true mid-wall response but rather the response of the material covering the wall. It is indicative, however, of rattling of objects on or adjacent to walls.

Summary of findings

- Lower corner horizontal responses for single wide and double wide trailers and camp structures exceeded ground velocities for similar components Single wide trailer with wood frame add-ons do not show this behavior.

- The lower horizontal corner response in log, earth, and masonry structures are equal or less than external ground motions.

- Trailers exhibited amplification of vertical ground velocities. Vertical structure response was less than external vertical vibration for all other structure designs.

- Upper (S2) and lower (S1) corners move in phase for trailers and one story camp, earth, stone, and masonry construction. Log structure corner motions are highly random and out of phase because they lack the frame support provided in other structure designs. Two story stone and camp structures show similar characteristics to log designs.

- The influence of airblast on whole structure response, for airblast of 116 dB and above, is clearly measured for trailers and two-story log structures. Earth, masonry and camp designs do not clearly show structure response to airblast.

- Mid-walls respond at high frequencies relative to whole structure responses. However, for log, camp, earth, stone, and masonry structures, mid-walls carried additional low frequencies associated with whole structure responses. Mid-walls did not respond to airblast in one-story log, earth, masonry, and two-story stone structures. Airblast effects are readily measured in mid-wall of all trailers, with both high and low frequency (whole structure) components, and camp structures.

- Loosely attached construction components and wall covering, such as paneling, can create high mid-wall motions that are not associated with structure response.

Correlating Structure Response to Ground Motions and Air Pressures

Whole structure (S2) and mid-wall responses were plotted against PPV and maximum airblast overpressure to compare the relative influences on structure response. Depending on structure design, the maximum structure responses will fall within the ground motion phase or airblast phase of structure response. For instance, trailer are sensitive to airblast and many of the peak velocities contained within the mid-wall time histories occur simultaneously with the airblast arrival (airblast phase) rather than during the passage of the ground motion wave (ground phase). Other structures show a greater sensitivity to ground motions and relatively little response to air pressures.

Maximum velocities within the upper structure (corner or peak measured at S2) and mid-wall time histories were plotted against the respective excitation driving the peak (e.g. peak air pressure or peak ground motion). Only horizontal components in the transverse, T, or radial, R, directions are considered.

Best-fit equations of structure response versus PPV for each structure design are presented in Table 6 to be consistent with RI 8507. Earlier discussions showed the importance of the entire excitation wave train. Thus these equations should not be used to predict structure response motion.

All equations were forced through the origin with a y-intercept value of ‘0’. A positive y-intercept at x = 0 is meaningless as it is not possible to measure a structure response without a positive driving force. A negative y-intercept is feasible in the case where a threshold force is necessary to measure a response. Although comparing this threshold among structures may be of interest, it was not a necessary component of response and therefore not measured. For comparisons with U.S. Bureau of Mines structure response equations given in RI 8507, positive y-intercepts were necessary to compute in some cases, but are not shown in Table 6.

Structure response to ground vibrations: Ground motion-induced peak structure responses are compared in Figures 14 and 15 for whole structures and mid-walls. Upper corner peak motions in Figure 14 show that only two structure designs (one-story log and earth, stone, and masonry) were subjected to peak ground motions greater than 0.40 in/sec. By comparing the data in Figure 14 with Figure 35 in RI 8507, it is apparent that atypical structure responses fall with the range of U.S. Bureau of Mines data.

However, the response of the two-story stone structure within a narrow range of ground motions from 0.21 to 0.45 in/sec shows amplifications above those exhibited by other structures within the same PPV range. The stone structure response can be explained by two factors. The unusual construction does not include an upper bond beam along the top of the walls. As such, the stone structure is free to respond without typical wall constraints. The second factor is that the ground frequency matched the natural frequency of the structure (about 4 Hz).

Mid-wall responses are shown for all structures in Figure 15. The mid-wall response of the stone structure is well above other structure designs. This is because the mid-walls did not move independently of the whole structure and amplified the 4 Hz ground vibrations. Mid-wall horizontal motions fall within the range of mid-wall responses reported in RI 8507 Figure 33.

Trailers are unique in that they have large ratios of transverse to radial wall dimensions. Figure 16 shows that the mid-wall responses in all trailers fall within two trends. Trailers tend to “rock’ along the long axis and whole structure responses are far larger in the transverse direction than in the radial direction. As stated previously, mid-walls carry the same motion carried by the whole structure. Hence, transverse mid-walls in trailers respond to this higher transverse motion.

Best-fit lines for one and two-story whole structure horizontal corner responses are given in Figure 17. Equations in Table 6 for these lines (given for all structures) show a large difference in slopes averaged for all structures. The one-story slope coefficient of 0.63 agrees with U.S. Bureau of Mines data fit for one-story wood frame structures (0.56 slope). Although the two-story slope of 1.43 falls above the 0.55 slope reported in RI 8507 for coal mine data, two-story whole structure responses fall within U.S. Bureau of Mines measurements when quarry and iron mine data are included.

Structure response to airblast overpressures: Airblast induced whole structure and mid-wall responses are shown in Figures 18 and 19. Earth, stone, and masonry structures did not respond to airblast over the ranges measured. All peak structure responses occurred strictly in the ground motion phase. Log structures exhibited little whole structure responses and no air-blast induced mid-wall responses**.**

The greatest airblast sensitivity existed in trailers for both mid-wall and whole structure responses. The large population of airblast-induced data for trailers indicates that the majority of the peak structure responses tended to fall within the airblast phase as opposed to the ground motion phase. Wood-frame and camp structures exhibited some sensitivity to airblast relative to ground motion. A comparison of mid-wall motions shows approximately 1.3, 1.8 and 2.9 times greater air-induced motions relative to ground-induced motions among trailers, wood-frame, and camp structure, respectively.

The unusual trailer and wood frame response to airblast (shown grouped within the ellipse in Figure 18) were recorded during an 11.6 Hz airblast pulse. The airblast frequency precisely matched the detonation time equal to the 67 ms front row delays plus the arrival time between holes spaced 21 feet apart, adding a 19 ms inter-hole travel time (21 ft. divided by the speed of sound in air around 1100 ft.). The inverse of 0.086 ms pulse beat is a strong 11.6 Hz that matched the power spectrum peak. This unusual airblast frequency is shown in Figure IV-7 for structure TS-IN and the response of the mid-walls and, to some degree, the whole structure, is evident.

Whole structure (racking) airblast responses in this study were very close to previous U.S. Bureau of Mines studies and recent measurements by Siskind (2002). The envelope of maximum response shown in Figure 18 is 77 in/sec/psi for well-confined blasts and 155 in/sec/psi for unusually high frequency airblasts. Historical U.S. Bureau of Mines and values provided by Siskind (2002) for equivalent type airblasts were 42 and 135 in/sec/psi, respectively. With the high variability of airblast characteristics and hence responses, these results can be considered equivalent and normal.

Airblast and vibration guidelines can be compared. The racking response maximum value of 155 in/sec/psi and regulatory limits of 132 dB for a 2-Hertz system (0.0129 psi), gives a maximum structure response of 2.06 in/sec.

In contrast to whole structure response, mid-wall responses to airblast shown in Figure 19 are higher than historical values, specifically for the trailer type structures. This study's worst case envelope for mid-wall responses was 442 in/sec/psi. The historical U.S. Bureau of Mines value was about 319 in/sec/psi, but did not include trailers. This study's results, exclusive of trailers, found a maximum of 266 in/sec/psi that is fairly close to the U.S. Bureau of Mine's value.

Summary of findings

- Whole structure and mid-wall peak responses induced by ground motions for all structures fell within data provided in U.S. Bureau of Mines RI 8507.

- Ground motion-induced whole structure response for one-story structures agrees with U.S. Bureau of Mines data fit for one-story wood frame structures. Two-story structure response falls above structure response reported in RI 8507 for coal mine data and within U.S. Bureau of Mines measurements when quarry and iron mine data are included.

- Earth, stone, and masonry structures did not response to airblast pressures while log structures produced measurable mid-wall responses and low whole structure responses.

- Trailers showed the highest whole structure and mid-wall responses to airblast with envelopes of 155 in/sec/psi and 442 in/sec/psi., respectively. Envelopes for other structures are 77 in/sec/psi and 266 in/sec/psi. These envelopes agree with historical U.S. Bureau of Mines data for non-trailer structures and are within normal ranges.

Fundamental Frequency Analysis: Natural Frequencies and Structure Damping

The natural frequency of each structure design was estimated using three methods. The first two methods were used to compute the natural frequencies during free response, when ground motions arrested, and during ground motion activity, when structure response peaks were 90-degrees out of phase with the ground motion peaks. The third method employed FFT analysis to calculate the predominant frequency of motion in structures when there was no free response. Calculating predominant frequencies using FFT analysis to estimate structure frequency response is desirable because blasting seismograph software easily accommodates this analysis. Isolating and computing natural frequencies over the response portion of structures that truly represents free response is often time consuming and requires experience. Therefore, a comparison of free response natural frequencies to FFT predominant frequencies is given herein to determine if using FFT analysis provides a good measure of structure free response.

Natural Frequency of Structures

Natural frequencies in structures can be observed either during free vibrations, when ground motions have ended, or during ground motions, producing a near-perfect sinusoid response, symmetrical about the time history x-axis and containing one single frequency. In the later case, structure vibration peaks will show a 90-degree phase angle shift from the ground motion (excitation) peaks, as described by Crum (1997) and predicted by theory (Harris, 2001). Examples of waveform time histories showing natural frequencies produced in the second floor upper corner and mid-wall during ground motions are given in Figures 20 (a) and (b). The ground motions are 90-degrees out of phase within the mid-wall and upper structure motions beyond the time marked by the vertical dashed lines. Just beyond this time the natural frequency can be measured. It should be pointed out that only two structures, TD-WV2 and E2S-NM, exhibited natural frequency response during ground motion activity.

Figure 20 (c) illustrates free response of an upper corner once ground motions have arrested and before arrival of the airblast. The structure response in this region, between 3.5 sec. and 6 sec., is 4.0 Hz. True free response measurements are often difficult to detect and analyze in the absence of ground motions and before the arrival of the airblast pulse. If the airblast arrives before ground motions arrest, free response may not be detected. The majority of structures exhibited this form of free response for natural frequency measurements. However, a sufficient number of structure responses in which ground motions could be isolated from airblast influence to obtain reliable free response measurements.

Table 7 shows the natural frequencies computed during the response phase shift from ground motions for E2S-NM (two-story stone structure) and TD-WV2 (double wide trailer). The 4.0 Hz stone structure radial and transverse mid-wall sensors were located on the first and second floors, respectively. The transverse S2 sensor placed in the 7.0 Hz double wide trailer was located along the marriage (center) wall and the radial sensor was placed on the outside wall, center at the structure peak. Both mid-walls were placed on outside walls. Within each structure, the frequency responses in mid-walls and the whole structure were identical, indicating that mid-walls do not respond independently but rather with the upper structure. Table 8 summarizes structure free response frequencies, calculated using the FFT of the time history after the ground motion has arrested. Data from structure response given in Table 3 from U.S.B.M RI 8507 for wood-frame structures are provide for comparison. Whole structure free response data for all structure and all sites compare well with U.S.B.M. data. Mid-wall response data may not compare because the U.S.B.M placed mid-wall sensors on the wall facing the blasts to capture air pressure effects. Therefore orientations could not be verified and mid-wall response data are averaged for both T and R directions.

Structure Response Based on Ground Motion FFT Analysis

Appendix V contains plots of relative amplitude from FFT analysis for S2 and MW as well as predominant frequencies of structure response compared to the dominant FFT frequencies of ground motions. Data are grouped by responses for radial, R, and transverse, T, walls to demonstrate that R and T frequencies are different for most structures.

Plotting structure response FFT frequencies based on relative amplitude from spectral analysis is a good means of identifying specific structures that respond at a unique and consistent frequency, regardless of ground motion amplitude and airblast levels. This further serves to illustrate how structures may amplify ground motions if the predominant ground frequency is close to the natural frequencies of the whole structure or mid-walls.

Figure V-1 through V-4 show relative amplitudes plotted against FFT predominant frequency at the upper structure (S2) and mid-walls (MW) for T and R walls. These peaks do not necessarily correlate with the averages given in Table 8 for all structures within each category as they represent the strong, dominating frequency for a single structure within the design category. For instance, in Figure V-1 (a), the single, strong peak at 3.8 Hz represents the predominant upper structure motion in TS-OH while all other single-wide trailers responded at higher frequencies. Whole structure double-wide trailer responses (TD-WV2 and TD-PA) shown in Figure V-1 (b) are centered at 7.2 Hz. Trailers with wood-frame add-ons responded at 4.4 Hz and 7.7 Hz.

Other dominating T frequencies are observed for all log structures, between 6.1 and 6.4 Hz, for designs with vaulted ceilings at 8.3 Hz, and earth, stone, and masonry structures, centered at 4.0 Hz. Camp and wood-frame structures show various amplitudes at a variety of frequencies that are not centered on one value.

Radial structure and wall motions show some predominance at 6.6 Hz for single-wide trailer TS-OH. Earth, stone, and masonry and log structures show central R frequencies similar to those in the T direction while camp and wood-frame structure show some focus between 6 to 7 Hz.

In Figures V-5 through V-8, T and R upper structure frequency responses are plotted against ground motions in terms of peak FFT frequencies. Data in Figure V-5 and V-7 indicate that single-wide and double-wide trailer structure frequencies do not correlate with ground motion frequencies for the same component. Response frequencies vary for whole structure and mid-walls. Wood-frame add-on trailers and log structures show a uniform behavior in response frequencies over a wide range of ground motion frequencies. Mid-walls tend to respond at frequencies higher than the upper structure. This is also observed for T walls for wood-frames structures in Figure V-6 (d).

Therefore, regardless of ground motions frequencies, structure frequencies were low and structures tended to respond at their natural frequencies. Trailers are an exception where structure frequencies highly varied.

Verification of Spectral Analysis Ability of Seismic Data Analysis Software

When using FFT methods to calculate frequency content, a question always arises regarding the computation schemes used in computing the power spectrum. The ability of computations to resolve the peak or predominant frequency in a spectral plot is a function of the number of data in the time history (record length) and sample rate (number of data points). The longer the record length, the more data are contained in the time history, and the frequency intervals become smaller. When only a small segment of the waveform (e.g. containing the natural frequency) is used in the FFT analysis, frequency intervals may become large, on the order of 0.5 to 1 Hz. Resolving the dominant frequency within $\pm$ 0.2 Hz may not be possible and the true peak may be missed.

Spectral plots using two software are compared in Figure 21 for the upper corner transverse response for TS-OH given in (a). Spectral plots using Seismograph Data Analysis 2000 v. 6.2.3 from White Industrial Seismology, Inc., and NUVIB (Huang, 1994) for various record length segments shown in Figure 21 (a) are given in Figures 21 (b) through 21 (d). Although the frequency intervals are not the same for each record length selected, the predominant frequencies calculated by each methods are in good agreement as follows:

	Predominant frequency in Hz White software	NUVIB
Entire waveform	3.75	3.72
Segment 1	4.00	4.00
Segment 2	3.75	3.72

Damping of Structure Motion

Structure damping near the natural frequency or during free responses was computed. Damping is the structure's resistance to movement and causes the structure to return to its resting position in a harmonic sinusoid. The harmonic vibration peaks decay in a well-defined exponential function from a maximum value, P_1, according to the following:

$$\beta = \frac{1}{2\pi m} \ln \frac{P_1}{P_{m+1}} \qquad (100\%) \qquad (1)$$

where β is the damping coefficient, P_1 and P_{m+1} are the successively peak amplitudes where $P_1 > P_{m+1}$ and P_1 is usually taken as the peak "free" response after the ground vibration has ceased. P_{m+1} is any peak following P_1, "m" cycles later in time. The damping coefficient is defined as the percentage of critical damping, where perfect damping is 100%. A perfectly damped system (such as a well-coupled geophone) is one that responds exactly the same as the driving force. On the other hand, at 0% damping, a structure would resonate and never stop vibrating. Values for successive damped peaks in the time history used to calculate β are illustrated in Figure 20 as P1 and P2.

Damping in structures is low as it takes many oscillations for a structure to complete moving. Dowding (1985) reports damping for residential structures in the range of 2 % to 10% of critical.

Damping terms were computed for structures that exhibited response peaks out of phase from ground motions, shown in Table 7, and for structures that exhibited free response after ground motions arrested, summarized in Table 9. Based on the data in Table 9, trailer transverse mid-walls showed the greatest damping (9.5% of critical). Log and trailer structures exhibited high damping in the radial structure peaks (9.7% and 9.6%, respectively). The least damped structure type was the earth, stone, and masonry structures with a 3.9% average damping term (the CMU block structure, E1S-NMA, did not show free response and therefore damping could not be computed). High damping in trailer and log structures can be explained by the unconstrained nature of construction components that do not effectively transmit frequencies. CMU piers supporting trailers are not mortared while logs are not nailed together to form a solid, supporting mass. Structure response amplitude may be high in such structures, but they quickly dampen due to the lack of structure bonding.

Summary of Findings

- Whole structure and mid-wall natural frequencies were determined for free response motions. Whole structures averaged 6.0 Hz and mid-wall averaged ranged from 8.4 to 13.8 Hz. U.S. Bureau of Mines whole structure natural frequencies range 7.1 to 7.8 Hz and mid-walls averaged 16.4 Hz.

- Average damping for all structure was 7.8% for whole structure vibrations and ranged between 7.3 % to 6.2 % for mid-walls. Average damaging values found by the U.S. Bureau of Mines ranged 4.4 % to 5% for whole structures and 1.8 % to 2.3 % for mid-walls.

- FFT methods are preferred to predict dominant frequencies because it takes into account the entire time history.

- Structures tended to respond at their natural frequencies with the exception of trailers. Structure response frequencies in trailers are highly varied and often are higher than the natural frequency.

- Log and trailer structures are more highly damped because of their lack of structure bonding.

Amplification Factors

Time-correlated amplifications of ground motions within structures were computed in terms of an amplification factor (AF) defined by Siskind et al. (1980a) and explained by Crum (1997). AF is defined as

$$AF = \frac{S2_{peak}}{V} \qquad (2)$$

where $S2_{peak}$ is the maximum velocity of the upper structure and V is the velocity of the ground motion for the same component at the corresponding moment of time or immediately preceding the time at the peak S2 motion. AF values were also computed using peak mid-wall responses relative to V in the ground.

Whole structure and mid-wall amplifications were determined from superimposed velocity time histories as shown in Figure 22 for the upper structure relative to ground velocity.

Plots of AF for whole structure responses are plotted for predominant FFT ground motion frequencies in Figures 23 through 27. For ground motion FFT frequencies greater than 7.1 Hz, the mean AF is 1.7 with a maximum of 3.3. At 7.1 Hz and below, the mean AF is 2.2 with a maximum of 5.0. Amplification factors greater than 3 were associated with ground motion frequencies between 4.0 and 7.1 Hz.

In U.S.B.M RI 8507, typical whole structure amplification factors are reported to be 1.5 with 4.0 being the highest value. The greatest values occurred at ground motion frequencies between 5 and 12 Hz. The U.S.B.M. study did not include sites with ground motion frequencies less than 5 Hz and included ground motions up to 85 Hz. In the current study, the average site ground motion frequency was 9.6 Hz with 28% of the sites exhibiting ground motion dominant frequencies of 5 Hz or less. It is reasonable to conclude that the U.S.B.M. data did not include AF greater than 4 because ground motion frequencies did not fall within the lower ranges of structure natural frequencies included in the current study.

Amplification plots by structure show that the two-story stone and two-story camp structures show the highest average amplification factors because structure natural frequencies matched those of the ground. The 4-Hz stone structure (E2S-NM) was subjected to six blasts with an average ground motion frequency of 4 Hz. One single two-story camp structure, with a natural frequency response of 6.1 Hz, was subjected to five blasts with ground motions averaging 6.4 Hz.

Summary of Findings

- Time correlated amplification factors (AF) ranged from 0.4 to 5. The U.S. Bureau of Mines calculated AF from 1.5 and 4.0.

- The highest AF values were observed for the two-story stone (4.6) and two-story camp structures (5.0) where the ground vibration frequencies matched the natural frequency of the structures. Log and one-story earth and masonry structures exhibited the lowest values of AF. Amplification factors in trailer were 4.0 and less.

- The highest amplification factors occurred when ground motion predominant frequencies matched structure natural frequencies.

Relative Displacements and Calculated Strains

Previous studies involving crack observations during blasting have shown that a strong correlation exists between peak particle velocity and blast-induced threshold wall damage (Nicholls, et al., 1971; Siskind, et al., 1980a; Stagg, et al., 1984). Studies that included dynamic strain gage instruments mounted on walls have produced limited insight to threshold strains that cause wall cracking. This is because changes in crack lengths and widths for blasting events are similar for time periods when no blasting took place. Furthermore, it is not possible to anticipate the wall locations that cracking will take place such that strain gages can be strategically placed.

Only two studies are notable. Wiss and Nicholls (1974) measured failure strains in gypsum wallboard during blasting and found new cracks formed during a maximum dynamic wall strain of 1010 μ-strains. Critical tensile failure strains in gypsum wallboard are given in RI 8507 by Siskind, et al., 1980a. Openings along butt joints and new cracks appeared during blasting events at failure strains in excess of 300 to 400 μ-strains. Strains associated with mortar joint cracking during blasting were measured in excess of 300 μ-strains (Edwards and Northwood, 1960; Northwood, et al., 1963).

Differential structure displacement time histories were computed by integrating velocity traces and used to compute the maximum differential whole structure strains. Peak or maximum differential displacements, $\Delta\delta_{max}$, between the upper and lower structure motions were used to determine global wall shear strains, γ, and maximum wall bending strains, ε. A schematic showing displacement and global shear strain is given in Figure 28. Note that the sensors mounted on the radial walls (the wall of the shortest overall structure lateral dimensions) measure gross structure motions in the transverse direction. Similarly, the transverse sensors measure motions in the radial walls.

Maximum differential displacements were computed by subtracting time-correlated displacement time histories measured at S1 (lower structure corner) from S2 time histories (upper structure corner). Since the polarity of the transducers was known, the resultant displacements were automatically accounted. Thus the relative displacement was obtained by simple subtraction. However only the absolute values are reported.

The maximum global structure shear strain of the wall, γ, is computed using the peak or maximum differential displacement divided by the wall height as follows:

$$\gamma = \frac{\Delta\delta_{\max}}{L} \tag{3}$$

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

The in-plane tensile wall strain, ε_L, is related to the gross structure shear strain for the same wall being affected by the motions. The maximum in-plane strain, $\varepsilon_{L(max)}$, is aligned along a 45 degree diagonal as shown in Figure 28, where $\theta = 45°$ is the direction of the maximum strain. The solution for in-plane tensile strains can be found in basic mechanics textbooks and $\varepsilon_{L(max)}$ is given as

$$\varepsilon_{L(max)} = \gamma_{max} \sin\theta \cos\theta \tag{4}$$

which reduces to

$$\varepsilon_{L(max)} = (0.5)\,\gamma_{max}$$

when $\theta = 45°$ for square walls and $\varepsilon_{L(max)}$ is one-half of the gross structure strain, γ. Global or overall in-plane tensile strains are critical to threshold wall cracking potential.

Calculations of wall bending strains with midwall motions is more challenging because it is necessary to estimate the bending mode shape. Dowding (1985) discusses this issue when relative upper corner superstructure displacements are known. The degree of fixity of the wall top and bottom controls the mode shape and thus the calculation of bending strains. Two of the possible mode shapes, fixed-free, and fixed-fixed, lead to following equations for maximum wall bending strain:

$$\varepsilon = \frac{6d\Delta\delta'_{\max}}{L^2} \qquad \text{(fixed-fixed)} \tag{5}$$

$$\varepsilon = \frac{3d\Delta\delta'_{\max}}{L^2} \qquad \text{(fixed-free)} \tag{6}$$

where d is the wall thickness divided by two, in inches, and ε is given as μ-in./in. or μ-strains. Even though the mode shapes and thus the coefficients to the strain equations can vary considerably as a result of the mode shape, a coefficient of 6 was employed in this study along with the maximum wall height for L.

The relative nature of the midwall displacements that are employed to calculate stains is also important. Where the midwall displacements, MW, are greater than those at either S1 or S2, it is difficult to know the relative midwall displacement with respect to displacements at the upper and lower corners. In this study, the average of the S1 and S2 was employed and the maximum wall displacement, $\Delta\delta'_{max}$, assumed to be located at the mid-wall, is calculated as

$$\Delta\delta'_{\max} = S_{mw} - \left(\frac{S_2 + S_1}{2}\right) \tag{7}$$

where S_{mw} is the peak mid-wall displacement and S2 and S1 are the time-correlated upper and lower corner displacements.

Calculated in-plane tensile strains and wall bending strains are summarized by structure design in Table 11. Average and maximum values are reported. Figures 29 (a) and (b) show examples of differential displacements (in terms of absolute values) calculations for the E2S-NM two-story stone structure in the radial and transverse directions, respectively. These displacements, given in inches, represent the average measurements for this structure during the study. Velocity time histories at the upper (S2) and lower (S1) structure corners were integrated and the resulting displacement time histories are subtracted (S2 - S1) to obtain the differential wall shear displacements. The absolute value of S2 – S1 is shown to readily display the maximum value of $\Delta\delta_{max}$.

Maximum calculated in-plane tensile strains and maximum calculated wall bending strains are shown in Figures 30 and 31 plotted against maximum ground motion for the same component. The largest in-plane tensile strains shown in Figure 30 were calculated from time-correlated differential displacements in the second story of the stone structure (E2S-NM). Motions in the radial direction resulted in a maximum calculated in-plane tensile strain of 113.1 μ-strain in the transverse wall. The second story transverse wall produced a maximum calculated bending strain of 46.6 μ-strain, assuming a fixed-free model of bending and is shown in Figure 31 at a PPV of 0.46 in/sec. The fixed-free model for structure E2S-NM is justified based on the absence of a top plate or beams affixed to the stone exterior walls to render the upper structure rigid. Calculated strains in the stone structure are below levels measured during previous research on mortar joint cracking during blasting.

One- and two-story log structures carry large strains due to their natural flexibility supplied by the individual wood members. Radial motions produced transverse wall peak strains of 95.5 and 66.6 μ-strain for one- and two-story log structures, respectively. Mid-wall strains were relative small for two-story structures and among the highest for one-story designs. Depending on the quality of wood, failure strains for logs can range from 7000 to 20,000 μ-strain (USDA, 1999). Therefore, calculated strains produced by blasting during this study are far below those strain levels that could possibly cause cracks in log walls.

Calculated strains produced in trailers, camp, wood-frame, concrete block, and adobe structures were as high as 12.5 μ-strains for gross structure shear (for which the highest was computed for wood frame types) and less than 9.2 μ-strains for all bending wall strains. Strains calculated for the one-story cinder block structure for radial and transverse in-plane strains fell below those calculated for wood frame structures. Cinder block wall strains are well below critical failure strains.

Summary of Findings

- Peak in-plane tensile strains calculated from whole structure differential displacements were 113.1 μ-strain in the two story stone structure. A value of 95.5 μ-strain was computed for a one-story log structure. For all other structures, whole structure wall strains were less then 40 μ-strain.

- Peak calculated mid-wall bending strains were the greatest in the two-story stone structure with a value of 46.4 μ-strains. Bending strains for all other structures were less than 26 μ-strain.

- In some structures, ground velocities may compare to structure response at S1. Therefore, ground velocities may be used to evaluate response in structures expect in the case of trailers where S1 does not match ground velocities.

Non-blasting Sources of Structure Vibrations

Household Activities

Structure responses to non-blasting events are shown in Table 12 for seven structures. A comparison of non-blasting event responses are shown in Table 13 compared with the maximum whole (upper) structure and mid-wall velocities recorded during blasting. It was not difficult to generate structure motions during normal household activities within trailers and wood frame structures. Structure responses from household activities were equal to those produced during blasting in the single wide trailer, TS-IN.

The more massive masonry and earth structures did not significantly respond during non-blasting influences. Therefore, responses shown in Table 12 are very low in amplitudes. Log and camp structures were not included in these tests.

Wind

Table 14 summarizes whole structure and mid-wall maximum velocities and strains for three trailers that responded to significant wind gusts traveling between 12 and 32 miles/hour. The maximum upper structure (S2) velocity and calculated whole structure strains (γ_{max}) are given for the T and R components or walls. Note that the upper structure response for the given component drives the shear strains in the opposing wall as previously described. For instance, the 0.055 in/sec maximum velocity recorded at S2 in the T direction produced an estimated 3.5 μ-strains of shear in the radial wall.

Upper structure transverse (S2) and mid-wall responses (both T and R walls) for air pressures (AP) from blasting and wind gusts are compared in Figure 32 for single wide trailer TS-KY2. Wind gusts are not efficient driving forces compared with blasting to excite significant structure responses. However wind gusts can generate air pressures that result in detectable levels of structure shaking and mid-wall responses up to 0.1 in/sec.

Summary of Findings

- Whole structure trailers motions from household activities were measured equal to motions induced from blasting. Mid-wall responses were general equal to or less than the responses from blasting. Structure responses from household activities in earth, stone and masonry structures were far lower and in some cases barely detectable in comparison with blasting responses.

- Trailer structure responses to wind gusts produced whole structure motions that were generally one-half of the motions generated during blasting.

CONCLUSIONS

1. Predominant frequencies of the ground motion time histories, as estimated from the Fast Fourier Transform power spectrum tended to be smaller than those computed using the zero-crossing method computed at the PPV. The frequency range with zero-crossing at the PPV was 16 to 32 Hz compared to a 7 to 20 Hz from the power spectrum. In all cases except one site, FFT frequencies fell below zero-crossing frequencies. The exception was the Tennessee site in which structure were founded directly on bedrock.

2. Fourier transforms and response spectra are preferable in structure response analysis to determine predominant excitation frequencies as the entire waveform is involved in the process.

3. Structure response relative to ground motions and airblast was evaluated by comparing horizontal time histories for the ground, lower structure (S1), upper structure (S2), and the mid-

walls. Differences between lower floor response and ground motions were small for all structure types with the exception of trailers in the vertical direction. Single and double wide trailers sustained wall base motions greater than exterior ground motions. In the case of trailers, wall base motions should to be instrumented in order to compute differential wall displacements. Although S1 measurements are preferred exterior ground motions may be used to estimate lower structure horizontal responses when foundations are coupled to the ground.

4. Whole structure motions, as indicated by the best-fit slope of upper structure response versus PPV, were the highest in the one two story stone (3.22) and camp (2.70) structures. Trailers, one-story wood frame, and log structures responded similarly with slopes of 1.29, 1.30, and 1.54, respectively. Other one story structures (log, earth and masonry) exhibited structure responses less than ground motions.

5. The greatest mid-wall responses, as indicated by the best-fit slope of mid-wall response versus PPV, were measured in log structures possessing "great walls" (2.98) and camp structures (2.58). Responses were similar for trailers (2.09) and wood frame (2.09) mid-walls

6. The influence of airblast over 116 decibels on the upper structure (S2) and mid-wall responses was observed for trailers. Whole structure and mid-wall motions duplicated airblast time histories and peak structure responses occurred within the airblast phase rather than within the ground motion phase. Mid-wall motions show both high frequency and low frequency characteristics for specific structures while trailer mid-walls tended to respond only at high frequencies. Upper (second story) mid-walls and upper structure corners move as one unit in most two story structures studied. In a number of cases, mid-wall responses duplicate airblast waveform signatures. Structure types that clearly did not show a response after the air pressure pulse arrival include one-story camp, log structures, and massive stone, concrete block and adobe structures.

7. Average values were determined for natural frequencies of mid-walls (8.4 Hz and 13.8 Hz) and whole structures (6.0 Hz) in both the radial and transverse directions. U.S. Bureau of Mines in RI 8507 reported average values of 16.4 Hz for mid-walls (no specific component) and 7.1 to 7.8 Hz for the whole structure. Dowding (1996) reported mid-wall frequencies between 12 to 20 Hz. Whole structure natural frequencies ranged 5 to 10 Hz. Data in this study corroborate these whole structure findings.

8. Damping characteristics during free response were evaluated for all structures. The greatest damping in mid-walls was found for the transverse direction in trailers equal to 9.5% of critical. Log and trailer structures exhibited the highest whole structure radial damping of 9.7% and 9.6%, respectively. The least damped structure type was the two-story stone that responded with an average damping of 3.9%. Values for damping fall well within those reported in the range of 2 % to 10% of critical by Dowding (1985).

9. Amplification factors varied by type of structure as well as for certain structures within each design type. These observations may be compared with those from U.S. Bureau of Mines RI 8507 where the maximum was 4 for structure corners. Corner responses of log and wood-frame structures fell below RI 8507 values. Out of this study of 25 atypical structures chosen for their

unusual character, only two structure designs displayed amplifications greater than 4. These included the two story stone and two story camp structures with upper structure motions amplified by 5.0 and 4.6, respectively. These values can be attributed to the fact that these structures were vibrated at or near their natural frequencies of 4 to 7 Hz.

10. In-plane tensile wall strains calculated from gross structure differential displacements were below cracking thresholds of 300 to 1000 μ-strains for plaster and wallboard. Calculated wall bending strains were less than 20 μ-strains.

11. Peak structure velocities induced in these atypical structures by occupant-induced motions were found to vary by structure type and distance between the source and measuring transducer. Habitation excitations that generated structure responses were primarily door and window closings. Those structures with low-mass walls (e.g., trailers) responded more than did structures with more massive walls to similar activities.

RECOMMENDATIONS

1. Time histories collected during this study of 25 atypical structures should be electronically archived for future access and analysis. They represent an unusually rich source of data that included ground motions as well as structural and crack responses.

2. The crack measurements presented in Addendum I to this study involved monitoring crack displacements, demonstrating that inexpensive techniques can be used to measure both long-term (environmental or weather-induced) and transient (blast induced) changes in crack widths, when conditions allow, to supplement traditional structure response techniques.

3. For atypical structures, time-correlated ground motion and structure velocity responses could be measured with systems similar to those employed in this study if conducted as outlined in Addendum II. Whole structure response motions should be measured at the top and bottom wall corners of uniform construction. Mid-wall response as well as crack deformations can be measured as additional options.

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