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OFFICE OF SURFACE MINING
RECLAMATION AND ENFORCEMENT
TECHNICAL REPORT/1994

INVESTIGATION OF DAMAGE TO STRUCTURES IN THE McCUTCHANVILLE-DAYLIGHT AREA OF SOUTHWESTERN INDIANA

Volume 2 of 3

Part II: Geologic and Unconsolidated Materials in the McCutchanville-Daylight Area.
Part III: Blast Design Effects on Ground Vibrations in McCutchanville and Daylight, Indiana from Blasting at the AMAX, Ayrshire Mine.
Part IV: Vibration Environment and Damage Characterization for Houses in McCutchanville and Daylight, Indiana.
Part V: Racking Response of Large Structures from Airblast, A Case Study.
Part VI: Investigation of Building Damage in the McCutchanville-Daylight, Indiana Area.

US Department of Interior
Office of Surface Mining
Reclamation and Enforcement

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U.S. Department of the Interior

Office of Surface Mining Reclamation and Enforcement

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Vol 1

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TECHNICAL REPORT/1994

INVESTIGATION OF DAMAGE TO STRUCTURES IN THE M^c^CUTCHANVILLE-DAYLIGHT AREA OF SOUTHWESTERN INDIANA

Volume 1 of 3

Part I: Composite Report

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Office of Surface Mining
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PREFACE

This report is a result of the combined efforts of the following agencies: U.S. Department of the Interior, Bureau of Mines (USBM), Geological Survey (USGS), and Office of Surface Mining Reclamation and Enforcement (OSM); the U.S. Army Corps of Engineers, South Atlantic Division Laboratory (SADL) and Waterways Experiment Station (WES); and the Indiana Department of Natural Resources, Geological Survey (IGS). The report consists of nine parts described as follows:

Part I, Composite Report. Integration of all other parts of this report. Also includes: the scope of work of the study; historical background and characteristics of the study area; review of the conclusions from all facets of the study; and OSM findings and conclusions.

Part II, Geologic and Unconsolidated Materials in the McCutchanville-Daylight Area. OSM review of the 1989 IGS report describing the geologic and unconsolidated materials in the study area.

Part III, Blast Design Effects on Ground Vibrations in McCutchanville and Daylight, Indiana from Blasting at the AMAX, Ayrshire Mine. OSM analysis of citizens' complaints, historical blasting activity, and blast design influences on ground vibrations.

Part IV, Vibration Environment and Damage Characterization For Houses in McCutchanville and Daylight, Indiana. USBM 1989-90 investigation: monitoring and analysis of ground vibrations, airblasts, and residential structure responses associated with Ayrshire Mine blasting; and an evaluation of observed damages in the study area.

Part V, Racking Response of Large Structures from Airblast, A Case Study. USBM 1991-1992 investigation: monitoring and analysis of airblast propagation through the study area; and evaluation of structure responses of large buildings to airblast from Ayrshire Mine blasting.

Part VI, Investigation of Building Damage in the McCutchanville-Daylight, Indiana Area. USGS 1991-92 investigation: structural engineering inspections of residential buildings in the study area and in a "remote" area unaffected by blast vibrations; risk assessment for potential damage from earthquake-induced ground vibrations; geophysical testing; ground vibration monitoring and structure response measurements; and analysis of "site response" characteristics within the study area.

Part VII, Experimental and Analytical Studies of the Vibration Response of Residential Structures Due to Surface Mine Blasting. WES, Structures Laboratory 1991-92 investigation: monitoring of ground vibrations, airblasts, and structure response; development of finite-element (FE), multi-degree-of-freedom model for "typical" one- and two-story residential structures; estimation of failure potential of structural materials from measured and predicted ground vibration amplitudes; and static analyses of potential basement floor and wall failure from observed settlements and estimated loadings.

Part VIII, Dynamic Soil Property Testing and Analysis of Soil Properties, Daylight and McCutchanville, Indiana. WES, Geotechnical Laboratory 1991-92 investigation: testing of soils obtained from the study area; assessment of consolidation and shear-strength reduction tendencies of foundation soils under cyclic loadings; evaluation of soil swell potential; estimation of soil settlement potential under static loadings from residential structures; and estimation of static horizontal soil pressures against basement walls.

Part IX: Environmental Conditions Related to Geology, Soils, and Precipitation, McCutchanville and Daylight, Vanderburgh County, IN. OSM overview of geologic, soils, and precipitation data from the study and remote areas; and assessment of geologic and soil conditions affecting buildings.

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INVESTIGATION OF DAMAGE TO STRUCTURES
IN THE MCCUTCHANVILLE-DAYLIGHT
AREA OF SOUTHWESTERN INDIANA
FINAL REPORT

EXECUTIVE SUMMARY

In March 1989, the Indiana Department of Natural Resources (IDNR) requested assistance from the Office of Surface Mining Reclamation and Enforcement (OSM) in determining whether there was a relationship between vibrations from blasting at coal mines and damage occurring to nearby homes. OSM entered into an agreement with the U.S. Bureau of Mines (USBM) to monitor blast-induced ground vibrations, airblasts, and structure response at selected homes and to assist in determining whether blasting was responsible for the observed damage in the study area. A number of outstanding issues remained after the completion of the USBM 1990 report. To resolve these issues, OSM authorized additional field investigations in the study area beginning in October 1991. The work was conducted by an interagency team made up of staff scientists and engineers from USBM; the U.S. Geological Survey (USGS); the U.S. Army, Corps of Engineers, Waterways Experiment Station (WES); and OSM. Final reports from each agency were submitted by March 1994 for review by OSM and the principal investigators. OSM developed this composite report to draw together the work and findings presented in the individual agency reports.

Findings

- The effects of both natural and man-made stresses were evaluated to identify the most probable causes of damages. The results of the study do not substantiate claims that damage to structures within the study area was caused by blasting. The evidence supports natural stresses as the principal cause of the observed damage.

- The focal point of this study was blast-induced ground vibrations and airblasts. Blast-induced ground vibrations in the study area were not found to propagate abnormally or to be unusual relative to the results of other studies.

- The natural frequencies of the soils closely matched those of the structures. When the two natural frequencies match, structure amplification of the ground vibrations and the potential for damage also increase. However, this study did not produce any empirical data indicating that there was sufficient structure response for damage to occur. Peak structure response to blast vibrations in both McCutchanville and Daylight were significantly lower than the 0.5 inches per second (ips) ground vibration required to crack the weakest construction material, plaster.

- Normal household activities recorded in a structure in McCutchanville generated structure response comparable to that caused by blast vibrations during the same monitoring period. Vehicular traffic, in particular aircraft landings and takeoffs, recorded in McCutchanville documented structure response similar to those observed from many of the monitored blast vibrations.

- The available evidence suggests that the use of different blast designs did affect peak amplitudes of ground vibrations in the study area. From all of the ground vibration data available to OSM, the maximum recorded peak particle velocity (PPV) values were 0.2 ips in Daylight and 0.06 ips in McCutchanville.

- The linear-elastic, FE model analysis of one- and two-story structures estimated the structural stresses in building materials from predicted theoretical "worst-case" ground vibrations. The results of the model analysis indicate that damages to wallboard and plaster walls in structures with normal foundation or superstructural conditions should not have resulted from amplitudes below 1.0 ips. Cracking in brick veneer mortar joints should not have occurred below 0.4 ips.

- The available evidence suggests that fatigue failure of building materials from the cumulative effects of mine blasting was not responsible for damages in the study area.

- Evidence from vibratory and triaxial soil tests indicates that blast-induced ground vibrations have neither consolidated nor destabilized foundation soils enough to generate structure distress.

- There is insufficient evidence that airblast caused enough structure response to induce damage. Atmospheric conditions strongly affect airblast propagation and prohibit reliable prediction of overpressure levels during unmonitored periods.

- Natural stresses result from soil conditions, moisture availability, temperature fluctuations, humidity fluctuations, and earthquakes. Geologic and cultural conditions also affect the soils and hydrology around a structure.

- Earthquakes periodically affect the study area. Coal company seismographs in the Daylight area recorded the ground vibrations of a June 10, 1987 earthquake with amplitudes ranging from 0.13 to 0.44 ips. Earthquake-induced vibrations were recorded at one of the complainant structures in McCutchanville at 0.06 ips on September 26, 1990.

- Structural inspections of homes in the remote area, which served as a control, showed similar types of damages as those observed in the study area.

- Several natural processes unrelated to ground vibration are the primary causes of damage to structures in the study area. These processes are related to foundation problems associated with soil movement and water; and construction practices. The evidence suggests the other potential mechanisms resulting in damage includes: foundation settlements; soil erosion; down-slope movement of soils; inadequate drainage control around foundations; shrink-swell of shallow loessial soils; clay swell potential along the soil bedrock interface; excessive lateral earth pressure; and thermal and humidity fluctuations. The degree to which

these processes have taken place at a specific structure depends on a variety of structure-type and environmental factors.

- Construction practices strongly influence a structure's ability to resist loads. Some construction practices observed in the study area have made damages more likely. Examples include: (1) the use of short dowels for connecting the superstructure to unreinforced masonry wall block, thus creating a zone of weakness in the top course of block; (2) structural additions; and (3) lack of reinforcement in foundations, basement walls, or concrete floors. Distress in joists and beam supports resulted from the use of improper materials or from excessive loading from the superstructure.

Background

In 1973, the AMAX Coal Company opened the Ayrshire Mine east of Evansville. During the life of the mine, over 10 square miles were mined with a nearly 2.5-mile long north-south trending pit advancing westward toward the communities of Daylight and McCutchanville in Vanderburgh County. In February 1988, AMAX added cast blasting to its mining program. Cast blasting uses additional explosive energy, rather than machinery, to move a portion of the overburden. This change in the use of explosives was noticed by homeowners in the surrounding communities. The number and frequency of complaints to both AMAX and IDNR increased significantly. The citizens maintained that vibrations from blasting were causing damage to their homes.

In late 1988, IDNR and AMAX began a joint study to determine whether any changes should be made to the allowed blasting limits. The study evaluated 75 blast events at eleven sites including the mine's normal compliance stations and additional sites in the concerned communities. The vibrations recorded had low frequencies, long duration, and peak particle velocities from below 0.02 to 1.22 ips. The study was inconclusive in terms of evaluating the cause of damage and recommended maintaining existing vibration limits.

The complaints continued, and in March 1989, the Director of IDNR requested assistance from OSM in determining the relationship between the damage occurring to the homes and vibrations from blasting at nearby coal mines. OSM/IDNR survey teams visited 107 structures in the McCutchanville-Daylight area. The teams recorded information about the location, age, construction materials, alleged damages, and residents' perceptions of blast severity and photographed each structure.

Interagency Investigation

After observing the damage, OSM negotiated an agreement with USBM to monitor blast-induced ground vibrations, airblasts, and structure response at a selected number of homes and to assist in answering the following questions: Is blasting responsible for the observed damage in the study area? Are the recorded vibrations unusual in any way? Do those vibrations produce an increased risk of damage? A second agreement was reached with the Indiana Geological Survey (IGS) to characterize the soils and the geology in the area

and to drill and sample soils at the homes being monitored by USBM. IGS sent soil samples to the U.S. Army Corps of Engineers South Atlantic Division Laboratory for testing of the engineering properties of the soils and clay mineral identification.

USBM selected six sites from the OSM survey records for measurement of ground vibrations and airblast. Two structures were monitored for response to those vibrations. USBM recorded maximum ground vibrations of 0.1 ips in Daylight and 0.06 ips in McCutchanville. USBM Report of Investigation (RI) 8507 indicates that vibrations of more than 0.5 ips are needed to crack the weakest construction material, plaster. For the cracks observed in some of the foundations and basement walls to occur, vibrations at least ten times greater than those recorded would be required. The maximum airblast recorded during the study was 121 decibels (dB), well below the damage threshold established in RI 8485. Structure response was typical of residential structures and was lower than the response to human activity in one home during this period. USBM also measured existing cracks to determine if they changed because of any blast vibrations. No crack movements were observed in response to blast vibrations, but a number of cracks opened and closed in response to changes in weather.

The IGS investigation documented that the study area is underlain by Pennsylvanian bedrock consisting mostly of unnamed shale and sandstone units with thin beds of limestone, clay, and coal. The land surfaces are divided into the upland, the middle surface or sideslope, and a lower historic lake basin. The unconsolidated materials are between 10 feet thick at some upland and middle surface locations to greater than 80 feet in the lake basin. The upper and middle surfaces of the eastern half of the area contain modern soils with a hard impermeable layer or fragipan at 2 to 3 feet below the surface. Portions of this soil profile restrict drainage at the fragipan, the buried soil horizons and the bedrock interface. Expansive clay minerals and associated mixed layer clays were identified within the soil profile. Swelling soils may exert lateral and uplift pressures on foundations and basement walls when alternately saturated and dried.

Drought conditions were recorded between February and May 1988 followed by heavy rain in July. In February 1988, the first month of the drought, AMAX introduced cast blasting. In July, precipitation jumped above the 11 year average, rehydrating the dried soils. When rehydration occurs, the expansive characteristics of the clays are activated.

Based upon evidence available in 1990, it was established that: 1) significant and widespread occurrences of damage to houses in the study area had been documented; 2) blast-related ground vibrations and/or airblasts from the Ayrshire Mine were discernable to the complainants at distances in excess of four miles; 3) no correlation was made between blasting and cracks in the studied structures; 4) the maximum amplitudes of ground vibration and structure response were well below the established thresholds for damage; and 5) analysis of drilling and testing data indicated the widespread presence of expansive clays in the study area. However, in-house and interagency reviews of the OSM investigation up to and including the USBM study identified the following technical issues:

1) To what extent does blast design (both conventional and cast blasting) alter the effects of blast vibrations in the study area?

2) To what degree do geology, soil, and topography influence ground wave propagation; site response amplification; and the amplitude, frequency, and duration of waves?

3) Are there ground vibrations at very low frequencies [down to 0.5 Hertz (Hz)] that are capable of causing damage?

4) Do airblasts produce adverse structure response in the study area?

5) Certain types of damages, observed by some investigators, appear to have been caused by lateral forces. If so, what are the relative contributions of blast-induced ground vibrations/airblasts, earthquakes, and wind to this force?

6) Can observed damage be ascribed to fatigue induced by the repetitive exposure of structures to ground vibrations and/or airblasts?

7) Is there a potential for collapse of the structure of unsaturated soils or pore-pressure rise in saturated soils in the study area due to ground vibration?

8) Are there comparable damages in a remote area (unaffected by blasting) with similar geology, soils, and topography?

9) Do alternative mechanisms (inadequate foundations, slope/soil movement) contribute to the observed damages?

To resolve these issues, OSM authorized additional field investigations in the study area. The work was conducted by an interagency team made up of staff scientists and engineers from USBM, USGS, WES, and OSM. Each agency's responsibilities, detailed in interagency agreements, are summarized as follows:

1) USBM monitored structure response to airblasts induced by blasting at the Ayrshire Mine and other sources during a time period of 7 months at 5 locations in the study area. Monitored sites were structures characterized as having the best possibility of airblast-induced response due to high surface area exposure, proximity to source of airblasts, etc. Possible effects of weather-induced focusing of airblasts were also assessed.

2) USGS: a) conducted structural engineering inspections in 13 "complainant" residences, 20 "non-complainant" residences in the study area and 19 residences in a "remote" area of similar topography, geology, soil types, and construction types but unaffected by surface mine blasting; b) monitored and analyzed ground vibrations induced by blasting and other sources in the general frequency bandwidth of 1-30 Hz.; c) monitored 2 to 4 residences over a time period of 3 months for structure response to ground vibrations from blasting and other sources;

d) conducted surface refraction and down-hole shear/compression wave measurements at selected monitoring sites and at sites which indicate potential anomalies in induced-vibration response, structure type, or material-failure patterns; e) identified geotechnical variations between sites that may cause different ground/structure responses; and f) estimated past and future earthquake magnitudes and intensities in the study area.

3) WES: a) conducted engineering analyses on typical structures to (i) estimate vertical wall loads on footings, (ii) determine probable extent of foundation settlement from estimated static wall loads, and (iii) determine differential settlements required to cause yield line cracking in unreinforced basement floor slabs; b) conducted a lateral load analysis for unreinforced basement walls in a typical structure; c) monitored free-field and near-structure ground vibrations, airblast, and structure response from blasting and other sources of cyclic loading; d) conducted modal tests to identify overall and component dynamic properties (including all natural frequencies) of a selected structure; e) performed FE analyses using structural models (one-story and two-story) based on information obtained above; f) evaluated the potential of fatigue failure; g) tested soil samples for consolidation under induced cyclic loading; and h) conducted undrained cyclic triaxial tests and companion static undrained triaxial tests on saturated specimens from the study area.

4) OSM: a) provided funding for each participating agency's program; b) contracted for and managed support drilling, soil sampling, and laboratory testing services required to facilitate the team's investigations; c) obtained rights-of-entry from cooperating landowners to permit access for monitoring, inspection and/or drilling activities; d) provided personnel for direct field support to principal investigators; e) conducted an assessment of the historical blasting records of the Ayrshire Mine in order to determine effects of various blast designs on vibrations and to establish theoretical worst-case vibration levels; f) conducted field reconnaissance work to establish geology of both the study and remote areas; g) conducted the assessment of impacts of soils, geological and environmental factors in the project area; and h) drafted the composite interagency team report.

INVESTIGATION OF DAMAGE TO STRUCTURES
IN THE MCCUTCHANVILLE-DAYLIGHT
AREA OF SOUTHWESTERN INDIANA
FINAL REPORT

PART I: COMPOSITE REPORT

1.0 INTRODUCTION

1.1 Purpose of Study

This report presents the results of a study by the U.S. Department of the Interior, Office of Surface Mining Reclamation and Enforcement (OSM) into citizens' complaints in McCutchanville, Daylight, and other communities in and near Vanderburgh County, Indiana. The complainants contend that damages to homes and other structures have been caused by surface mine blasting operations of the AMAX Coal Company (AMAX) Ayrshire Mine. This study was initiated at the request of the Indiana Department of Natural Resources (IDNR).

The purpose of this study was to determine if AMAX blasting has caused the alleged damages to structures in the McCutchanville-Daylight, Indiana area (hereafter referred to as the "study area"). The project has been structured around two central questions:

1) Have buildings in the study area been affected by surface mine blasts which produced ground vibrations or airblasts of such amplitude, frequency and/or duration as to cause damage?

2) Have the damages in the study area resulted from causes other than blasting?

1.2 Scope of Work

OSM initiated this study in response to a March 22, 1989, request from IDNR. The study has included several phases of activity as follows:

- Collection of background information: Through July 1989, OSM, with the assistance of IDNR, collected information on the physiography and cultural development of the study area from available literature, maps, and records. OSM also identified and conducted a preliminary analysis into potential causes of damages. The results were used in planning other phases of the investigation.

- Survey and analysis of damages and past structure-response events: In June 1989, OSM and IDNR personnel visited 107 properties in the study area to document the condition of the structures and review the residents' experience of past vibrations. OSM evaluated the complainant data, Ayrshire Mine blasting data (date/time of blasts and blast design parameters), and data on past weather conditions. From August to November 1989, preliminary analyses were made of (1) the range of damage

severity existing among the homes and buildings of complainants in the study area and (2) the complainants' perception of vibrations relative to a variety of blast-design and weather factors.

- Characterization and analysis of ground vibrations, airblasts, and structure responses: USBM performed on-site monitoring of blast-induced ground vibrations, airblasts, and structure responses for selected houses in the study area. The monitoring took place between November 1989 and January 1990. The results from this investigation are presented in Part IV of this report.

- Characterization and analysis of the physical setting in the study area: Information on bedrock geology, ground water, and soils in the study area was gathered via drilling, geophysical logging, and rock/soil sample analysis by IGS in November and December 1989 under an interagency agreement (IA) with OSM. The results of this work are presented in Part II of this report. Data pertaining to the engineering properties of foundation soils in the study area were also obtained from laboratory testing performed by SADL in February and March 1990 and by the IGS. This information is referenced in Parts VIII and IX; and has been used by WES and OSM to assess the potential effects of soil behavior on structures in the study area.

- Interim evaluation: In-house and interagency review of work accomplished during the previous phases of the study took place from February 1990 to February 1991. The review process included two interagency site visits: one taking place in December 1990 and involving the U.S. Department of Agriculture, Soil Conservation Service (SCS), IGS, IDNR, and OSM; the other taking place in February 1991 and involving the SCS, WES, USGS, USBM, IDNR, and OSM. The purpose of these site visits was to collect additional field information on soil conditions and their potential effects on structures; obtain structural and geotechnical engineering assessments of damages; and utilize the expertise and perspectives of scientists and engineers not previously involved in the study. This process resulted in identification of nine technical issues discussed in Section 1.3 and in the submission of proposals from several of the agencies involved.

- Structural and soil engineering analyses; site response analysis; earthquake risk assessment; and airblast monitoring: Planning for this final phase of the study took place from February to September 1991. Negotiations were held with USGS, WES, and USBM to formulate scopes of work. A series of briefings were held with concerned citizens, IDNR, AMAX, and Congressional staff. Upon receipt of comments from these sources, the scopes of work were finalized. IAs were entered into with USGS, WES, and USBM. Additionally, OSM undertook two independent investigations at the request of the interagency team. The investigations were designed so that the study, in its entirety, would address all of the issues.

  Field work took place from October 1991 to December 1992. The scope of work and results of the IA investigations are contained in Parts II, IV,

V, VI, VII, and VIII of this report. The OSM investigations are presented in Parts III and IX. Excerpts and summaries of the investigations are provided in the context of the nine technical issues in Part I, Section 3.

1.3 Technical Issues

The technical issues that have steered the planning and performance of this study are as follows:

1) To what extent does blast design (both conventional and cast blasting) alter the effects of blast vibrations? The possibility existed that blast design affected the character of ground vibrations and airblasts, even at large distances from the source. This was significant considering the limited time periods of the ground vibration, airblast, and structure-response monitoring relative to the duration of mine operations. Consequently, a historical assessment of blasting was conducted by OSM and the report is presented in Part III.

2) To what degree do geology, soil, and topography influence the propagation of ground waves; site response amplification; and the amplitude, frequency, and duration of the waves? An in-depth understanding was needed of the influence of the geology, soils, and topography of the study site through which the vibrations travel. Of particular interest was the potential for soils and topographic highs to amplify vibrations. This was addressed in the USGS investigation (Part VI). The propagation characteristics of blast-induced vibrations were analyzed by USBM (Part IV).

3) Are there significant ground vibrations occurring at frequencies below 1 Hz? Ground vibration frequencies monitored during the USBM Part IV investigation ranged from 1 to 5,000 Hz. The possibility could not be dismissed that frequencies below the measured lower bound existed; and had sufficient amplitude and enough harmonics with buildings to induce damage. Frequencies below 1 Hz were monitored by WES (Part VII).

4) Are airblasts causing damage in the study area? The lack of historical airblast data was documented in the USBM Part IV investigation. Consequently, USBM conducted additional measurements of airblast and structure response (Part V). Particular attention was given to the response of buildings with large exposures to the airblast wave front.

5) What is the source and nature of the "lateral force" postulated to be a causative factor of certain damages such as horizontal cracks? The interagency site review in February 1991 identified damages that could not be readily ascribed to soil or foundation problems. This included long horizontal wall cracks near or above ground level and other signs of strain resulting from an unidentified lateral force. This lateral force, if present, could have acted on structures from below ground through ground vibrations or from above ground through airblasts. The USBM 1989-90 investigation had monitored blast-induced vibrations, airblasts, and response and had concluded that mine blasting should not

have caused damages. This potential contradiction highlighted the need for additional analysis. USGS and WES contributed to the resolution of this issue (Parts VI and VII, respectively).

6) Can observed damage be ascribed to fatigue induced by the repetitive exposure to ground vibrations and/or airblasts? The 1989-90 USBM investigation in this study was limited to immediate effects of blasting. During the interim evaluation, reviewers raised the possibility of fatigue-induced damages from repetitive blast events. The cumulative effects of ground vibrations and airblasts on building materials had been studied by USBM near the Ayrshire Mine (RI 8896). This research included a thorough analysis of the response of one recently built residential structure to cyclic loadings. Additional examination of the fatigue question was performed by WES (Part VII).

7) Are there synergistic effects occurring between ground vibration, soil movement, and structure distress? Work accomplished up to the final phase of this study was limited to the direct effects of blast-induced ground vibrations and airblasts on structures. The interim evaluation raised the possibility that repetitive, low level vibrations from blasting affected structures by dynamically loading the foundation soils. Soil samples from the study and remote areas were tested for potential consolidation or strength reduction under cyclic load by WES (Part VIII).

8) Are there comparable damages in a remote area with similar geology, soils, and topography? Damages to buildings can result from many different factors. A comparison of structures between the study area and a remote area was undertaken to assist investigators in identifying causes of damage. A structural engineering survey of homes in the study area and the remote area was undertaken by USGS (Part VI).

9) To what extent do alternative mechanisms (expansive soils, hydrology, inadequate foundations, slope/soil movement, piping, etc.) contribute to the observed damages? The identification of causes of damage other than blasting in the study area was necessary for the completeness of the study. Contributions to the resolution of this question have been provided by all four agencies that have participated in the final phase of the study (see Parts IV, VI, VII, VIII, and IX).

2.0 BACKGROUND

2.1 Location and Description of Study Area

The study area is approximately 8 miles north of Evansville in southwestern Indiana (Figure 1) and covers an area of approximately 60 square miles. The approximate coordinates on the Evansville North and Daylight, Indiana, USGS quadrangles range from 38 degrees, 3-7 minutes North and 87 degrees, 28-32 minutes West. The area is bounded by Base Line Road to the north; the intersection of U.S. 41 and State Route 57, and the extension of Millersburg Road to the south; U.S. 41 to the west; and County Line Road to the east. The topography is mostly flat with gentle hills to the west, where the relief is approximately 200 feet. Pennsylvania Age bedrock formations containing shale, sandstone, limestone, and coal underlie the area and dip westward at about 25 ft/mi. into the Illinois Basin. The rock is overlain by Pleistocene loess and lacustrine deposits.

The study area includes the western section of the Ayrshire Mine permit; the communities of McCutchanville and Daylight; and the Evansville-Dress Regional Airport. Based on the USGS quadrangles, which were photo-revised in 1986, there are approximately 1,800 structures in the study area. The land use includes low-density residential neighborhoods, agriculture, light industry, and mining.

In 1989 there were 23 active surface mines (including the Ayrshire Mine) in Warrick County and one underground mine in Vanderburgh County. The surface mines are all east and within 19 miles of the study area. Four or five mines have been reclaimed and are pending bond release. Surface mining in Vanderburgh County is prohibited by county law.

2.2 History of the Ayrshire Mine

The Ayrshire Mine (Permit No. S-00004) began operations in the Danville and Hymera coal beds in 1973. The highwall of this surface mine progressed from the eastern boundary of the permit westward to within 4.5 miles northeast of McCutchanville and 1.5 miles east of Daylight. Over 10 square miles were mined. The final pit was approximately 2.5 miles long, 120 feet wide. The mine advanced westward at a rate of about 1/4 mi./yr. Overburden thickness above the lower coal bed (the Hymera) averaged 84 feet within the boundary of the Ayrshire Mine permit.

In February 1988, the Ayrshire Mine adopted cast blasting for the northern portion of the pit for more effective and economical removal of bedrock overburden. Cast blasting is the use of explosives for the breakage and horizontal displacement of overburden material into the adjacent pit. Generally, cast blasts are designed to move between 10 and 50 percent of the overburden into the pit. More explosives per unit volume of rock are used in this type of blast than with conventional blasts.

Throughout the summer and fall of 1988, complaints about blasting at the Ayrshire Mine increased. IDNR received 1 complaint in June, 1 in August, 6 in October, and 25 in November. The majority of the residents who were

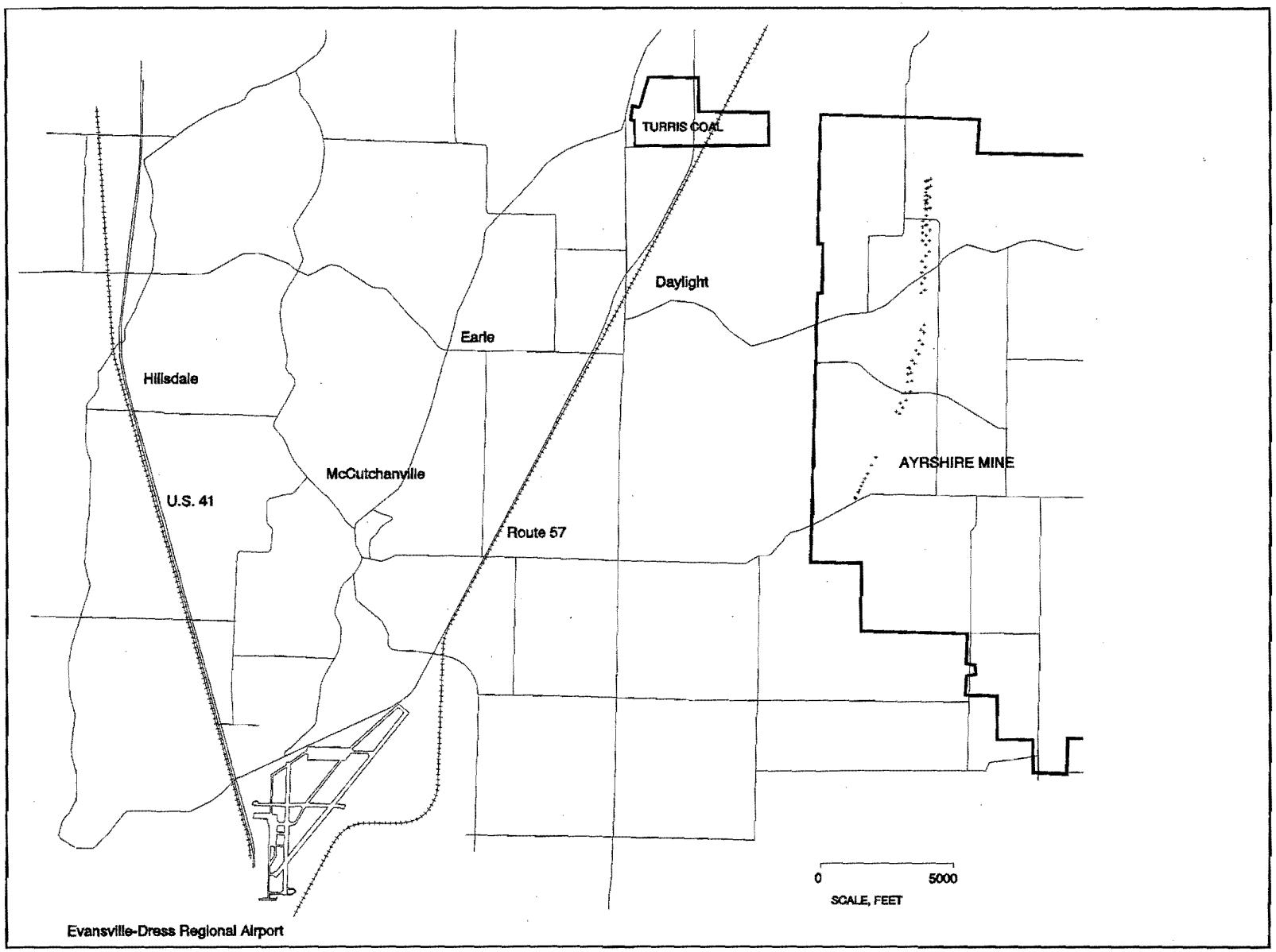

Figure 1. McCutchanville-Daylight study area, near Evansville, Indiana.

submitting complaints to IDNR lived between 3 and 5 miles from the mine, with some as far away as 10 miles. The greatest concentration of complaints came from the southwestern portion of the study area in and around McCutchanville. The other complainants lived in more sparsely populated areas.

2.3 The IDNR Investigation

The first government investigation in response to the complaints in the study area was conducted by IDNR between November 1988 and June 1989. IDNR reviewed blast records and citizen complaints; and coordinated a ground vibration and airblast monitoring program with AMAX. The monitoring program recorded vibrations and airblasts at up to 11 locations between December 5, 1988 and the end of February 1989; and included 75 blasts during two full cuts along the Ayrshire Mine pit. The findings of the IDNR investigation included the following:

1) Between September 1, 1988 and May 30, 1989, Ayrshire Mine detonated 296 blasts. One blast on December 13, 1988, at 1:39 PM exceeded the allowable vibration amplitude limit under the Indiana regulations at the closest permit compliance station. All other blast vibrations were measured to be within the regulatory limits and most of them had amplitudes far below predetermined damage thresholds.

2) During the same September 1988-May 1989 period, IDNR received reports of 191 structure-response events that shook and/or allegedly damaged homes. IDNR determined that 123 of the 191 of the reported events (64 percent) occurred within + or - 15 minutes of the AMAX blasts and thus were determined to correlate with those blasts.

3) The amplitudes of the 24 blast vibrations selected for detailed analysis were very low, ranging from 0.01 tc 0.05 ips. However, these vibrations were also characterized by low frequencies (4 to 12 Hz) and long durations (5 to 11 seconds). Based on these characteristics and the results of previous research, IDNR determined that "some of the vibrations recorded would be '<u>distinctly perceptible</u>'."

4) All measured airblast levels were within the limits of the regulations. However, several airblasts exceeded 120 dB at stations more than two miles from the blast sites. Normally, only quarterly airblast data from compliance stations are required. The report recommended that continuous recordings be made of Ayrshire Mine airblasts.

Although the IDNR study indicated that at least some of the AMAX blasts were discernable to residents of the study area, the measured blast-induced ground vibrations and airblasts were below predetermined thresholds of damage. However, it was also clear that the scope of this investigation did not address the central issue, which is whether or not damages in the study area were caused by the blasting. The possibility that damages could be resulting from blasts below regulatory limits was not ruled out.

2.4 Initial OSM Involvement

After receiving the assistance request from Indiana, OSM initiated a background-collection phase for its own investigation. This included the development of a list of 115 homes allegedly damaged by blasting. The list was prepared from a log of letters and telephone calls provided by IDNR (76 complainants), an intent-to-sue letter to AMAX (30 additional complainants), and a list of structures surveyed by AMAX (9 additional complainants). OSM also reviewed the ground vibration and airblast data obtained during the 75 blasts of the IDNR Two-Cut Study. The peak velocities of individual blast events recorded by the seismographs were analyzed to identify patterns of ground vibration propagation. Initial indications were that ground vibrations propagated more efficiently towards the northwest.

An important part of the background development was the preliminary assessment of potential causes of damage other than blasting at the Ayrshire Mine. The results of the assessment are summarized as follows:

- Mine Subsidence: Based on coal mine maps obtained from the Indiana Department of Conservation and the OSM Mine Map Repository, none of the complainants' homes appears to have been undermined. Two abandoned deep mines exist along the northern border of the Daylight USGS quadrangle, but do not extend into the study area.

- Cultural Features: There are several cultural sources of ground vibrations and airblasts in the study area. Many structures are in the flight path of the Evansville-Dress Regional Airport. A four-lane highway (State Route 41), two railroads, and a railroad switchyard are also within the study area. Interstate I-164 was under construction in the area around the time of the preliminary assessment, although no blasting was used. Trends between the locations of complainant structures and cultural features were not observed. However, the possibility that cultural activity had some effect on nearby structures could not be ruled out.

Other identified potential causes included earthquakes, soil consolidation, slope movement, inadequate foundation design, ground-water activity, and blasting at other surface mines. None of these factors could be readily eliminated.

Overview of Damages in the Study Area

OSM conducted a survey of complainant homes in the study area between June 19 and 29, 1989. During this two-week period, 60 of the 115 structures on the OSM complaint list and 47 additional structures were visited. A numerical identification system was developed for the surveyed structures (this identification system was used through the remainder of the study and is used in this report). Four field teams comprised of OSM and IDNR employees interviewed people residing or working in the homes and buildings; and recorded the condition of the structures. The teams gathered general information (names, addresses, telephone numbers, etc.); topographic map location; dates and times of vibration events; and data on building

construction, property development, and damages. The survey was not intended to be a representative sample of structures in the area, but rather an overview of the type and extent of damages.

The information from the standard survey forms was later examined to determine the range of damage among the sampled structures and to identify workable criteria for a damage ranking system. The rankings and their criteria are presented in the table below:

Damage Rankings for Surveyed Structures

Rank	Criteria
0	No reported damages on or within structure.
1	Damages limited to short, thin cracks or displacements that are concentrated at window and door frame corners and a few popped nails.
2	Extensive thin cracks/displacements and popped nails. A few areas of cracks and displacements greater than 1/8 inch or floor/wall bulges. Long horizontal foundation crack(s).
3	Several floor/wall bulges and areas of cracks/ displacements greater than 1/8 inch. Dense occurrence of thin cracks/displacements. Long horizontal foundation crack(s). Integrity of structural parts in jeopardy.

The rankings and other items of information from the survey were used at various points in the study as reference material for site selection and building damage analysis (see Part IX). Rankings 1, 2, and 3 closely resemble descriptions in the literature of "threshold," "minor," and "major" damages (Table 1).

2.5 Prior Research

Past research established guidelines recognized by the current Federal and IDNR regulations for safe ground vibration and airblast levels. Some of the research included blast monitoring within or near the current study area. Two central activities associated with this research were:

- Analysis of ground vibration and airblast characteristics in relation to site specific conditions and blast designs; and
- Analysis of dynamic structure response and damage thresholds in relation to blast-induced ground vibrations and airblasts.

Extensive work on the effects of surface coal mine blasting on structures has been accomplished by USBM. USBM recommended safe ground vibration and airblast levels as low as 0.5 ips and 134 dB, respectively. None of the research observed damage at ground vibration levels below 0.5 ips. Safe airblast levels were based on comparable structure response to ground

Description	Study	Uniform Classification
Loosening of paint Small plaster cracks at joints between construction elements Lengthening of old cracks	Threshold Dvorak (1962) Edwards and Northwood 1960) Northwood et al. (1963) Minor Thoenen and Windes (1942)	Threshold
Loosening and falling of plaster Cracks in masonry around openings near partitions Hairline to 3-mm (0-1/8 in.) cracks Fall of loose mortar	Minor Dvorak (1962) Edwards and Northwood (1960) Northwood et al. (1963) Jensen and Rietman (1978) Langfors et al. (1958) Major Thoenen and Windes (1942)	Minor
Cracks of several millimeters in walls Rupture of opening vaults Structural weakening Fall of masonry (e.g. chimneys) Load support ability affected	Major Dvorak (1962) Edwards and Northwood (1960) Northwood et al. (1963) Langfors et al. (1958)	Major

Table 1. Comparison of Damage Classifications, (Dowding 1985). From Part VII, Table 1.1.

vibrations (e.g., 134 dB generated the same structure response as 0.5 ips). Subsequent to 1980, USBM research related to:

- The potential of multiple blasts causing damages to single family houses through structural fatigue;

- The effects of various blast designs (especially millisecond delays between charges) on ground vibrations and airblasts; and

- The influence of underground mine workings and thick layers of unconsolidated sediments on the propagation of blast-generated ground vibrations.

Much of the above work was conducted at the Ayrshire Mine. This research confirmed past findings and the current guidelines on safe vibration and airblast levels. However, studies published in 1987 and 1989 did raise some concern about relatively high amplitudes and long durations associated with low frequency (4-12 Hz) ground vibrations near the Ayrshire Mine and other locations in Indiana. These studies, however, did not include structure response analyses.

Other work in southern Indiana included a statistical evaluation of damages in the study area by Barnes (1977, citation in Part III). This investigation did not include any monitoring of blast-related vibrations or airblasts. Barnes' conclusion that damages in the area were primarily results of AMAX blasting was based on an apparent correlation between homeowners' reports of damage and distance from the mine.

Braile et al. (1982, citation in Part III) monitored ground vibrations resulting from blasting at the Wright Mine in Warrick County, Indiana. Their evaluation included propagation patterns of the vibrations relative to distance and azimuth from the blast sites. Like Siskind et al. five and seven years later, Braile et al. expressed concern over low frequency, long duration vibrations occurring at far distances from the mine. This work also did not include a structure response analysis.

Ground Vibrations

The USBM research summarized above and earlier work at other locations resulted in concepts that are central to current regulatory standards on blast-induced ground vibrations employed by OSM and IDNR. These are as follows:

- Particle velocity (millimeters or inches per second) is the best parameter for monitoring ground vibrations and limiting amplitudes to levels below damage thresholds.

- Ground vibration amplitudes from blasting are best predicted by charge weight of explosives per delay in the blast design and distance from the blast site. Some effect may also result from the degree of charge confinement (i.e., room to which fractured rock may expand as it changes

volume) and the time delays between the detonation of individual charges within a blast.

- Damage thresholds are frequency dependent. Generally, damage threshold in peak particle velocity (PPV) decreases with lower frequencies. Frequency tends to decrease with distance from the blast site.

Frequency influences damage thresholds because the resonant or natural frequency range of one- and two-story homes, is low, between 4 and 12 Hz. Such structures respond more to ground vibrations in this frequency range. When the ground vibration frequency matches the natural frequency of the structure, the structure resonates.

Since 1982, the potentially damaging effect of low frequency, long duration waves have been recognized. Prior to this study, USBM did not observe damages from ground vibrations below 10 Hz, although other researchers had. Furthermore, the dominance of low frequencies relatively distant from a blast was found to result from the filtering effects of the geologic materials, soils, and in some cases, the influence of geologic structure. The frequency content of the vibrations are independent of blast design in the far field (one mile from the blast site and beyond). However, there has been uncertainty whether changes in blast designs can effectively limit the amplitude of vibrations in the far field.

Airblasts

USBM recommended safe airblast levels for the prevention of mine blast-related damages. Blasting produces airborne energy called airblast, overpressure, or impulsive sound. Charge size per delay and distance are important parameters for predicting airblast levels. Also, the degree of charge confinement is far more important for airblast than it is for ground vibration. High airblast levels (measured in decibels) are expected to result from blow-out (venting of explosion energy into the atmosphere). Airblast is also influenced by weather conditions, particularly wind direction and temperature inversions. For these reasons, airblast prediction is difficult for a given charge and distance and can vary by two orders of magnitude (a factor of 100). Airblasts travel more slowly than ground vibrations. Propagation speed is approximately 1,100 ft/s.

3.0 SUMMARY OF INVESTIGATIONS AND FINDINGS

3.1 To what extent does blast design alter the effects of blast vibrations?

Study participants recognized the possibility that certain aspects of blast design, in addition to explosives per delay and distance, may have been affecting the character of ground vibrations and airblasts. To assess the potential effects of various attributes of surface mine blasting on the environment, OSM performed an assessment of blast design, ground vibration, and complaint records. This investigation is summarized below.

3.1.1 OSM Investigation

Data for this work included the Ayrshire Mine blast logs (January 1, 1986, to April 15, 1992), IDNR and OSM complaint logs (January 1, 1988 to June 15, 1989), the IDNR study monitoring data (December 5, 1988 to February 27, 1989), and the USBM monitoring data (November 1, 1989, to January 3, 1990). The methods and results of specific analyses performed are as follows:

Classification of Blast Designs

Blast designs were identified and classified. They were divided into five "series," representing: conventional (100 series); cast (200 series); pre-split (300 series); box-cut (400 series); and parting (500 series) blasts (Figure 2). The series were subdivided into "patterns" which defined the blast hole layout (rectangular or staggered) and the initiation sequence (delay intervals between the rows and columns of holes). A final layer of subdivision pertained to the dominant number of explosive decks per hole per blast.

Historical Trends in Blast Design

Total pounds of explosives (total pounds) and pounds of explosives per delay (pounds per delay) were plotted against time (1986-92). Trends were compared with the monitoring periods of the 1988-89 IDNR study, 1989-90 USBM Part IV investigation, and 1991-92 interagency joint investigation (JI).

Total pounds did not exceed 100,000 lbs in 1986 and 1987 (Figure 3). In March 1988, with the advent of cast blasting, the range of total pounds began increasing and ultimately peaked in 1989, with a maximum of 411,688 lbs. The same trend applies to pounds per delay, which rarely exceeded 2,000 lbs in 1986 and 1987 and peaked at 8,500 lbs in 1989 (Figure 4). Although cast blasting continued to occur through the period of this study, both total pounds and pounds per delay steadily declined. The peak values fall between the monitoring periods of the IDNR study and the USBM Part IV investigation. The maximum total pounds and pounds per delay covered by these monitoring periods were significantly greater than those covered by the JI monitoring period.

A few conventional blast patterns were frequently used from the beginning of 1988 to the early months of 1992 (Figure 5). The occurrences of five cast

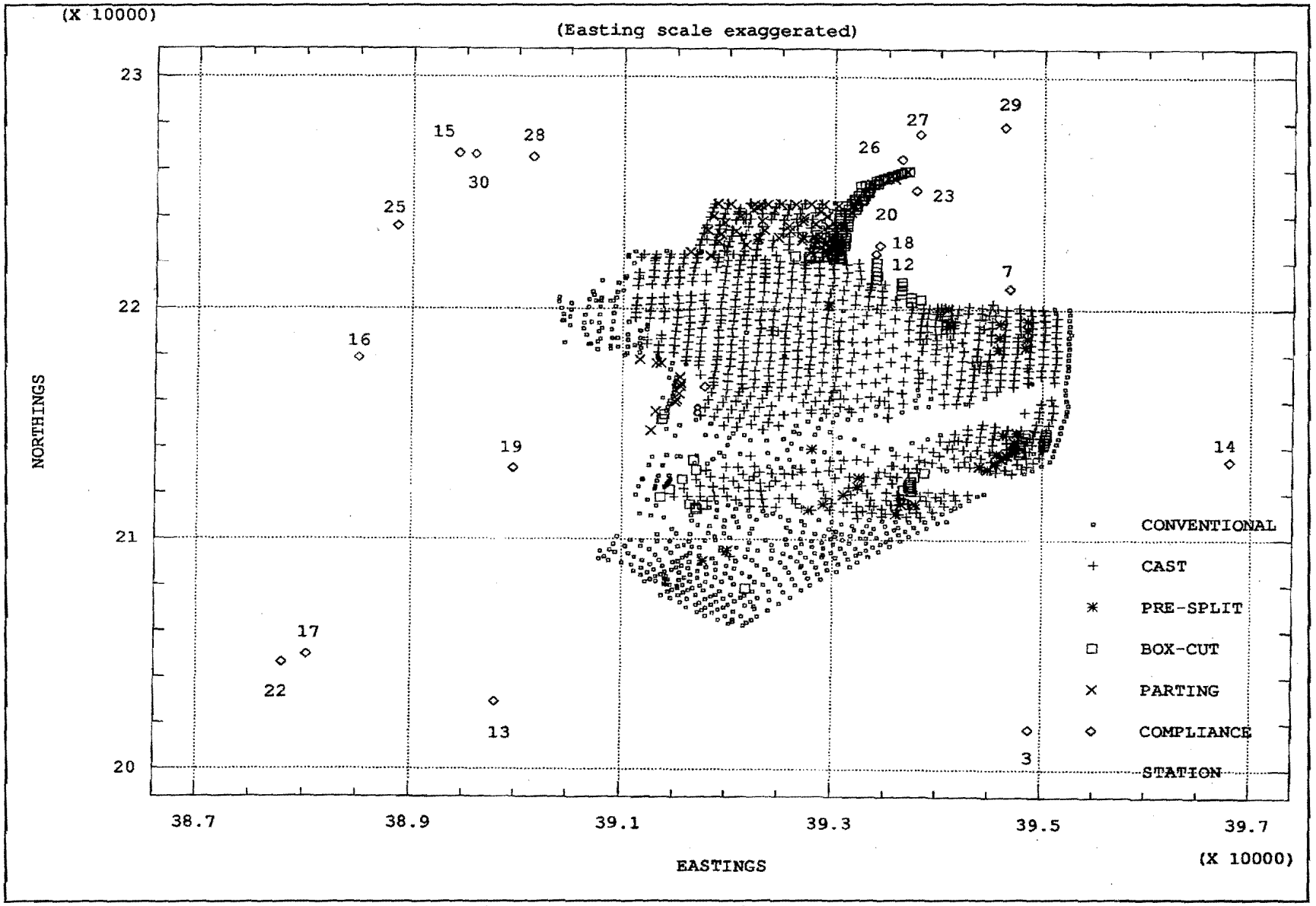

Figure 2. Blast location by pattern series. From Part III, Figure 10.

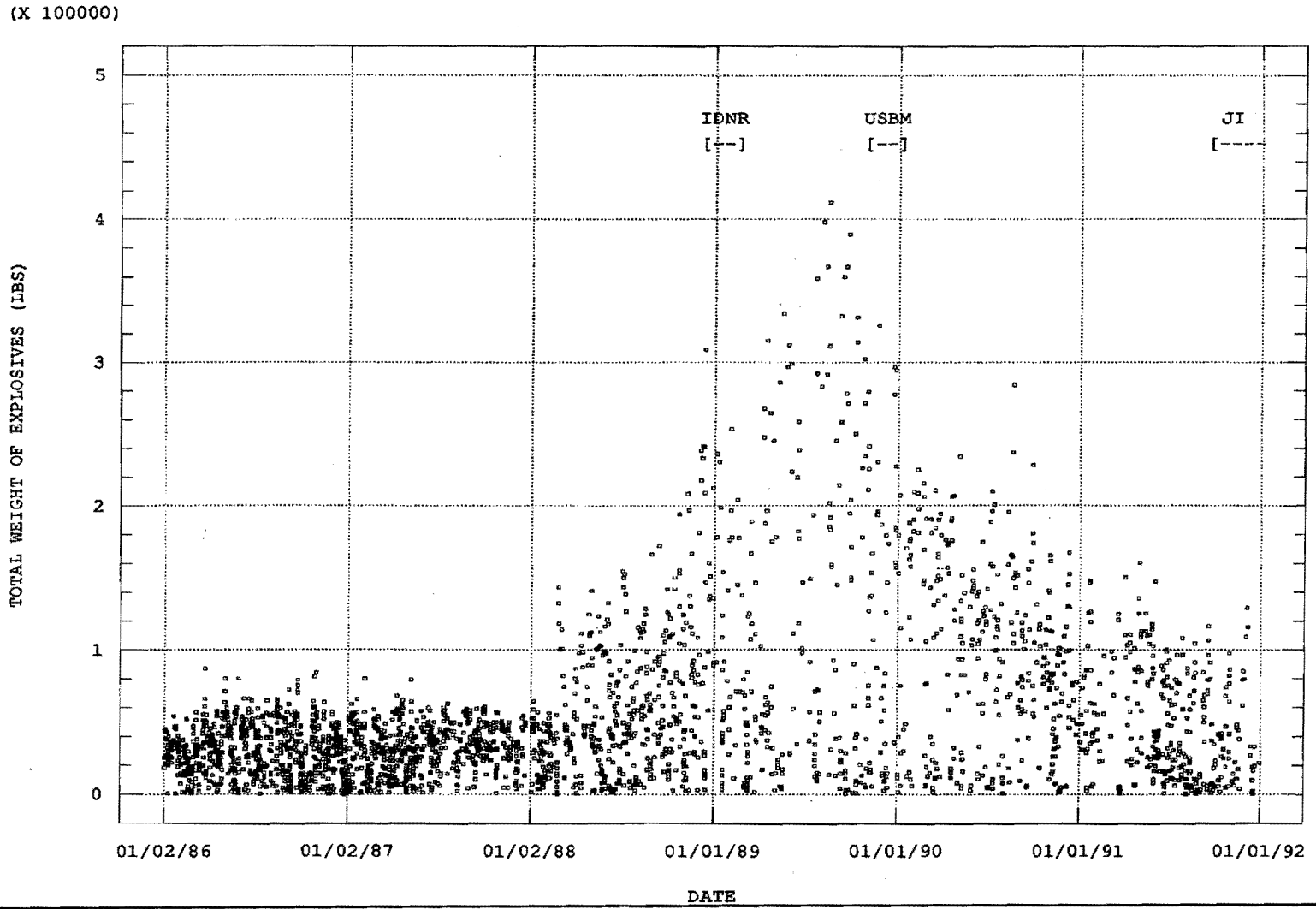

Figure 3. Ayrshire mine total weight of explosives per blast 1986 through 1991. From Part III, Figure 4.

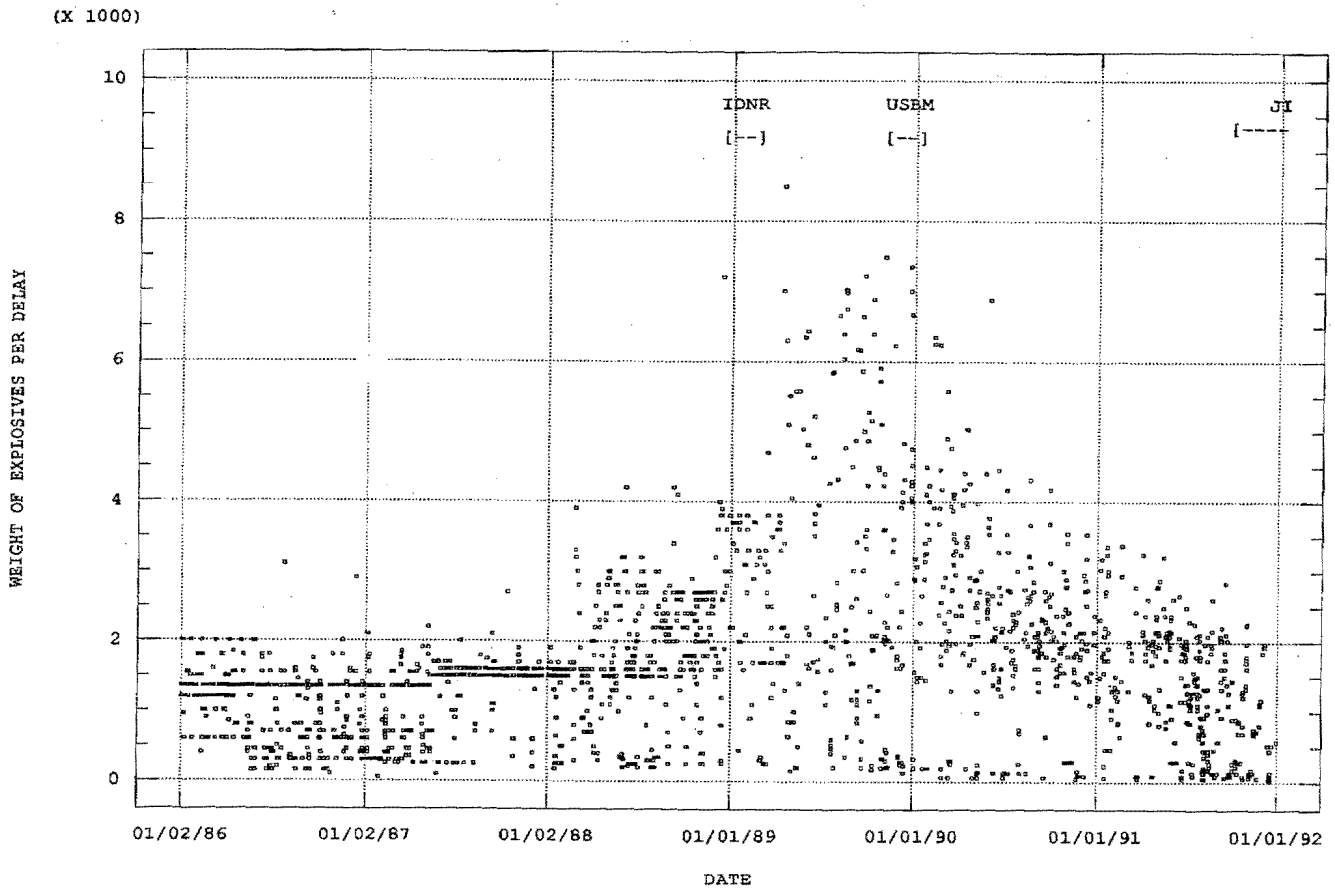

Figure 4. Ayrshire mine weight of explosives per delay per blast 1986 through 1991. From Part III, Figure 5.

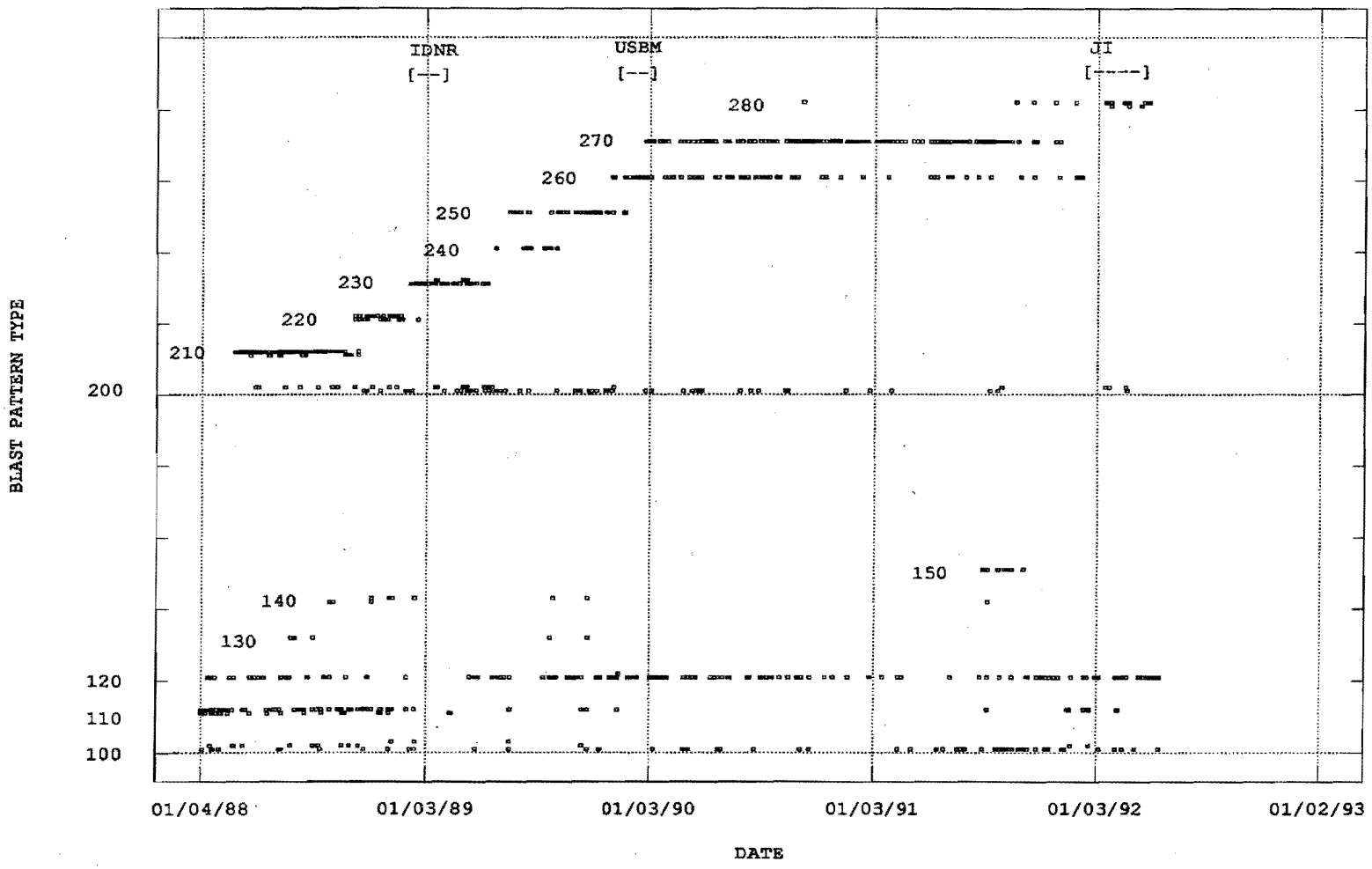

Figure 5. Time distribution of 100 and 200 blast patterns, January 1988 through April 1992. From Part III, Figure 16.

blast patterns (210-250) were each concentrated in approximately four to eight month periods. Collectively, these periods spanned 1988 and 1989. These were replaced by three other patterns (260-280) in 1990 and 1991. Altogether, the IDNR, USBM, and JI monitoring periods covered all cast blast patterns but 240. Pattern 240 was used between the IDNR and USBM monitoring periods. No individual monitoring period encompassed all of the cast blast patterns used during the course of the mine operation.

Complaints

The dates and times of events identified in citizens' blasting complaints were compared with the Ayrshire Mine blast dates and times for 1988. The frequency of complaints per blast were compared with blast patterns for January 1988 to June 1989. In a preliminary analysis, several other variables were also correlated with complaints for the IDNR monitoring period. These variables included total pounds, pounds of explosives, the duration of the shot sequence, the average depth of the blast hole, the powder factor, and various weather-related factors.

Approximately 80 percent of the 1988 complaints fell within +/-15 minutes of blasts at the Ayrshire Mine. This and strong correlations between blast design variables and complaint frequency supported the hypothesis that blast-induced ground vibrations and/or airblasts have been clearly discernable to many residents in the study area. Total pounds and pounds per delay were strongly correlated with complaint frequency for the IDNR study period. This agreed with the finding that, in 1988 and the first half of 1989, cast blast patterns resulted in significantly more complaints per blast than other patterns. The patterns with the three highest complaint/blast ratios were, from high to low, 230, 220, and 240 (Figure 6).

Ground Vibrations

Peak amplitudes of blast-induced ground vibrations were plotted against the square root scaled distance [SD=distance/(lbs. of explosives/delay)$^{1/2}$] for blasts between 1986 and 1992. Regression analysis of the IDNR study data for cast blast pattern 230, exclusive of Station 14, yielded the equation, $PPV=55(SD)^{-1.19}$, with a correlation coefficient of 73 percent. A two-standard deviation was applied to the spread of data for cast blasts to determine an upper bound. This upper bound is defined by the equation, $PPV=137(SD)^{-1.19}$ (Figure 7). Data from Compliance Station 14 was separated out because the station was located in spoil and east of the highwall.

Data point distributions were compared to the two lines for the following: (1) IDNR vs. USBM data; (2) cast vs. conventional blast data; and (3) cast blast patterns. Theoretical worst-case ground vibration amplitudes for the cast blasts in Daylight and McCutchanville were based on these comparisons. The mean line was used to predict worst-case amplitudes for patterns 250-280 and the upper bound was used for patterns 210-240.

Theoretical worst-case ground vibrations for Daylight and McCutchanville were estimated to be 0.38 and 0.17 ips, respectively. This generally agrees with

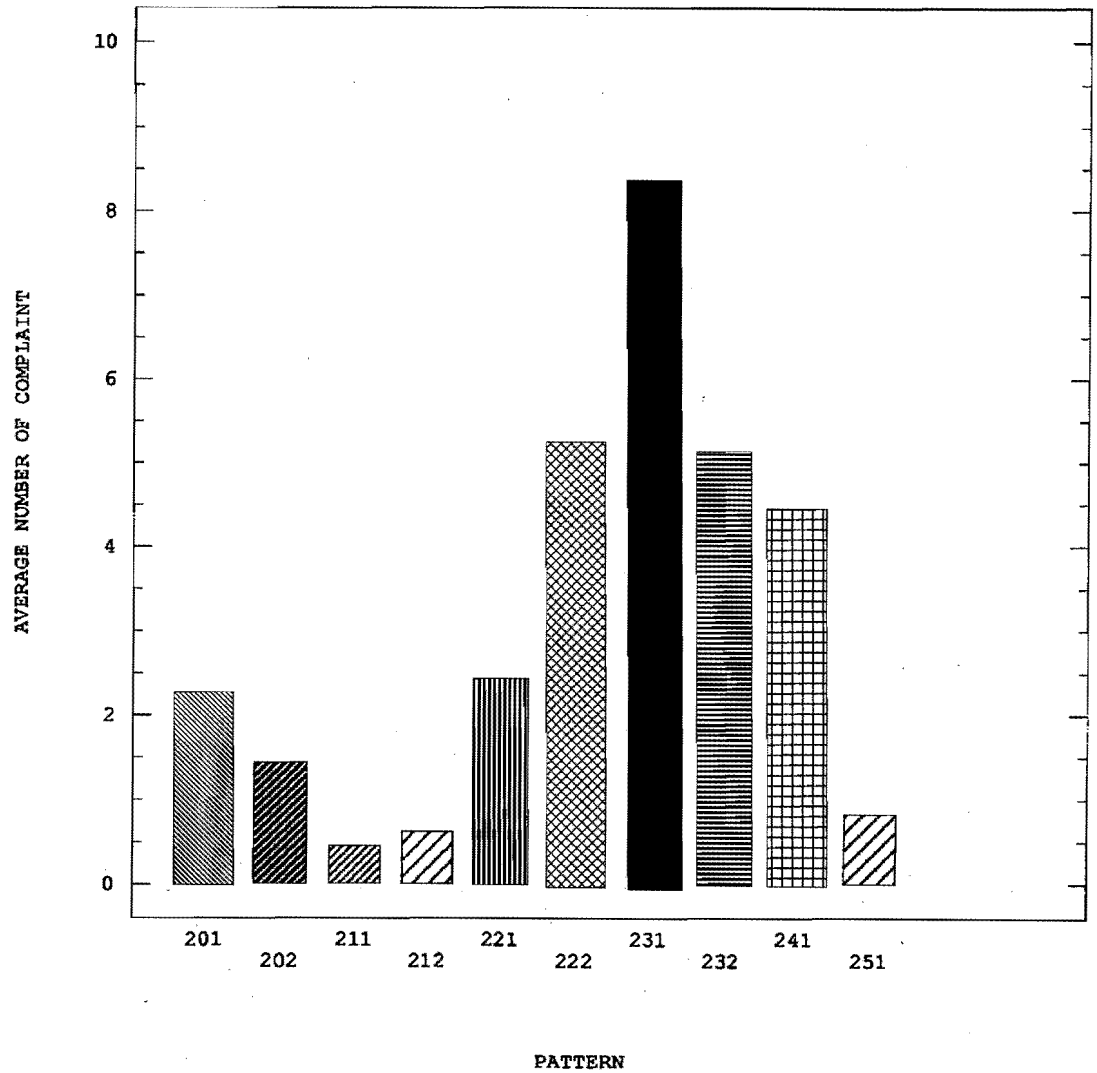

Figure 6. Average complaints per cast blast pattern. From Part III, Figure 15.

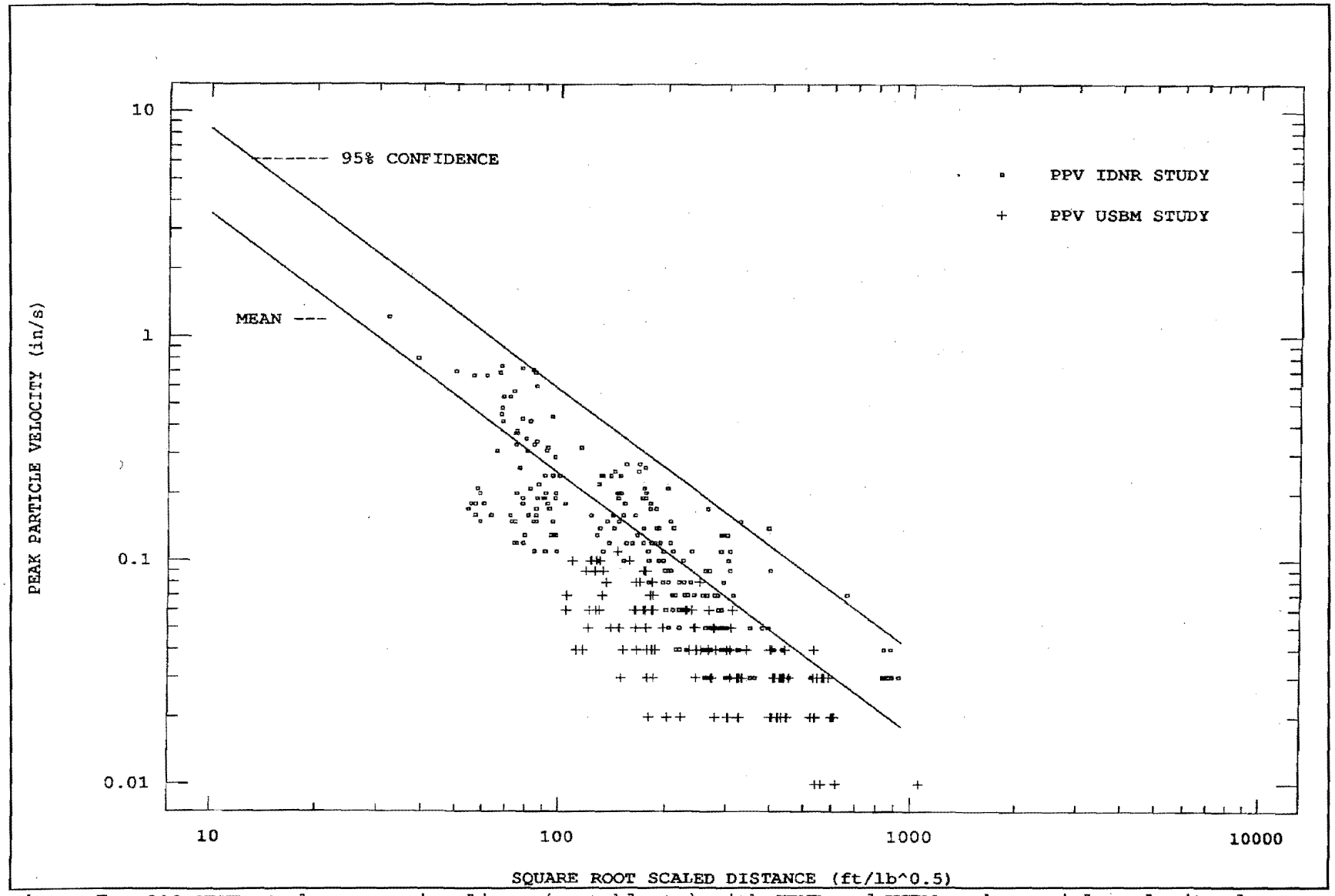

Figure 7. 1989 IDNR study regression lines (cast blasts) with IDNR and USBM peak particle velocity data. From Part III, Figure 18.

earlier estimations of 0.5 and 0.2 ips presented in the WES Geotechnical Laboratory report for the February 1991 field inspection (see Part VIII).

Taken individually, the range of ground-vibration propagation data points for cast blast patterns 210-230 approach the upper bound (Figure 8). The same can be said for pattern 240--although more cautiously since there are fewer data points to observe (Figure 9). The point spreads for patterns 200 and 250-280 do not approach the upper bound (Figure 10), with the exception of three points (for 200 and 270) which are clearly separate from the clustered data. It appears, therefore, that the blasts of 210-230 resulted in generally stronger ground vibrations than the other patterns.

OSM and USBM (see Section 3.2) investigators observed that ground vibration data from the USBM monitoring period are almost consistently lower than the IDNR data for similar scaled distances. The IDNR monitoring period covered patterns 200, 220, and 230; and the USBM period included 200 and 250-270. These two data sets are not restricted to compliance station recordings, but their relative distributions still correspond to the blast pattern relationships discussed in the previous paragraph.

3.1.2 Findings

The evidence suggests that the use of different cast blast designs did affect peak amplitudes of ground vibrations in the study area. The evidence also indicates that there may have been blast-induced vibrations in Daylight and McCutchanville that exceeded ground vibration recordings during the IDNR monitoring period and especially during both monitoring periods of the OSM study. The maximum calculated amplitudes predicted in Daylight and McCutchanville are 0.38 ips and 0.17 ips, respectively. The application of worst-case vibrations to a FE model used to predict structure response (see Section 3.5 and Part VII) is justified.

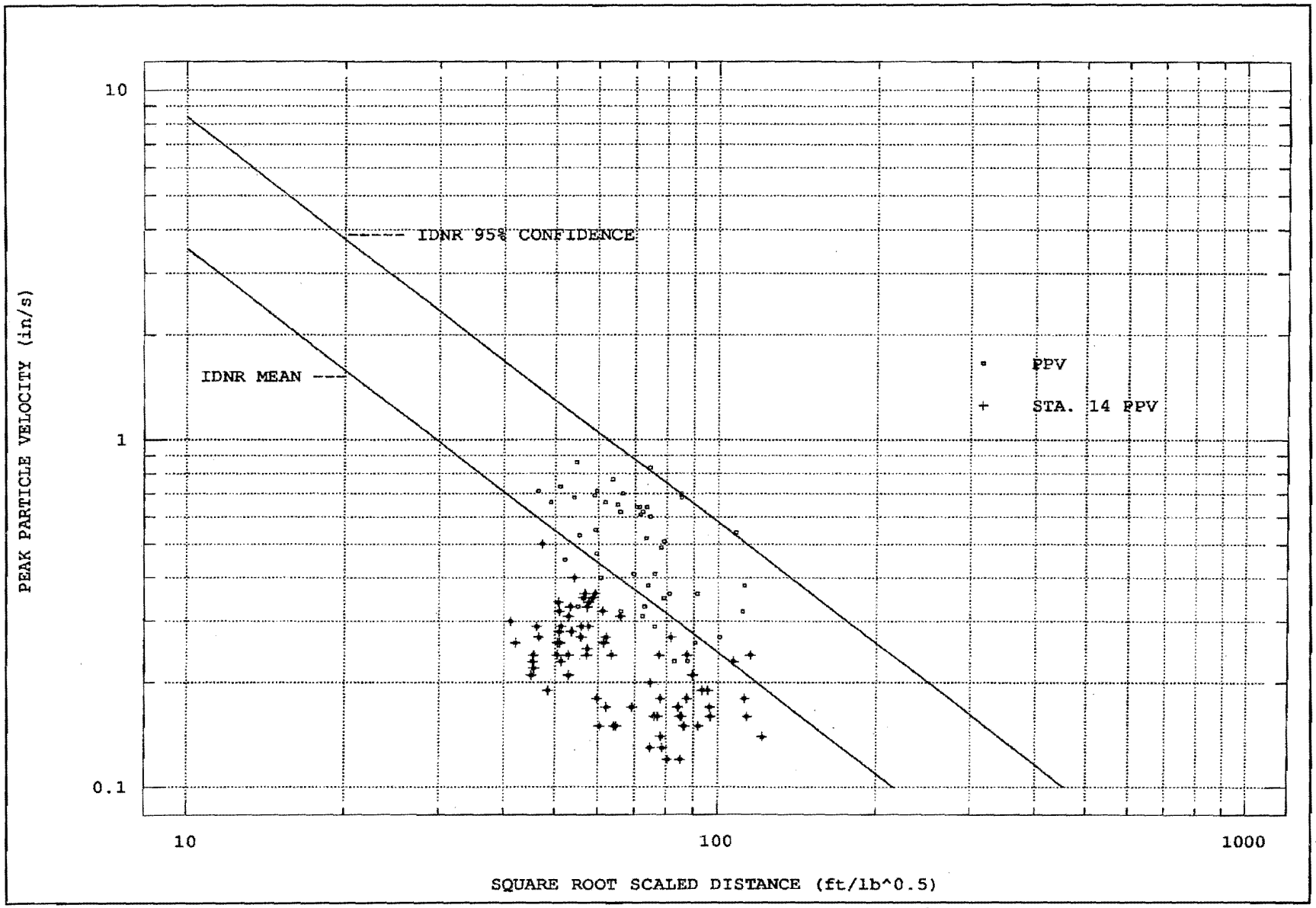

Figure 8. Compliance station data, cast blast pattern 210. From Part III, Figure 28.

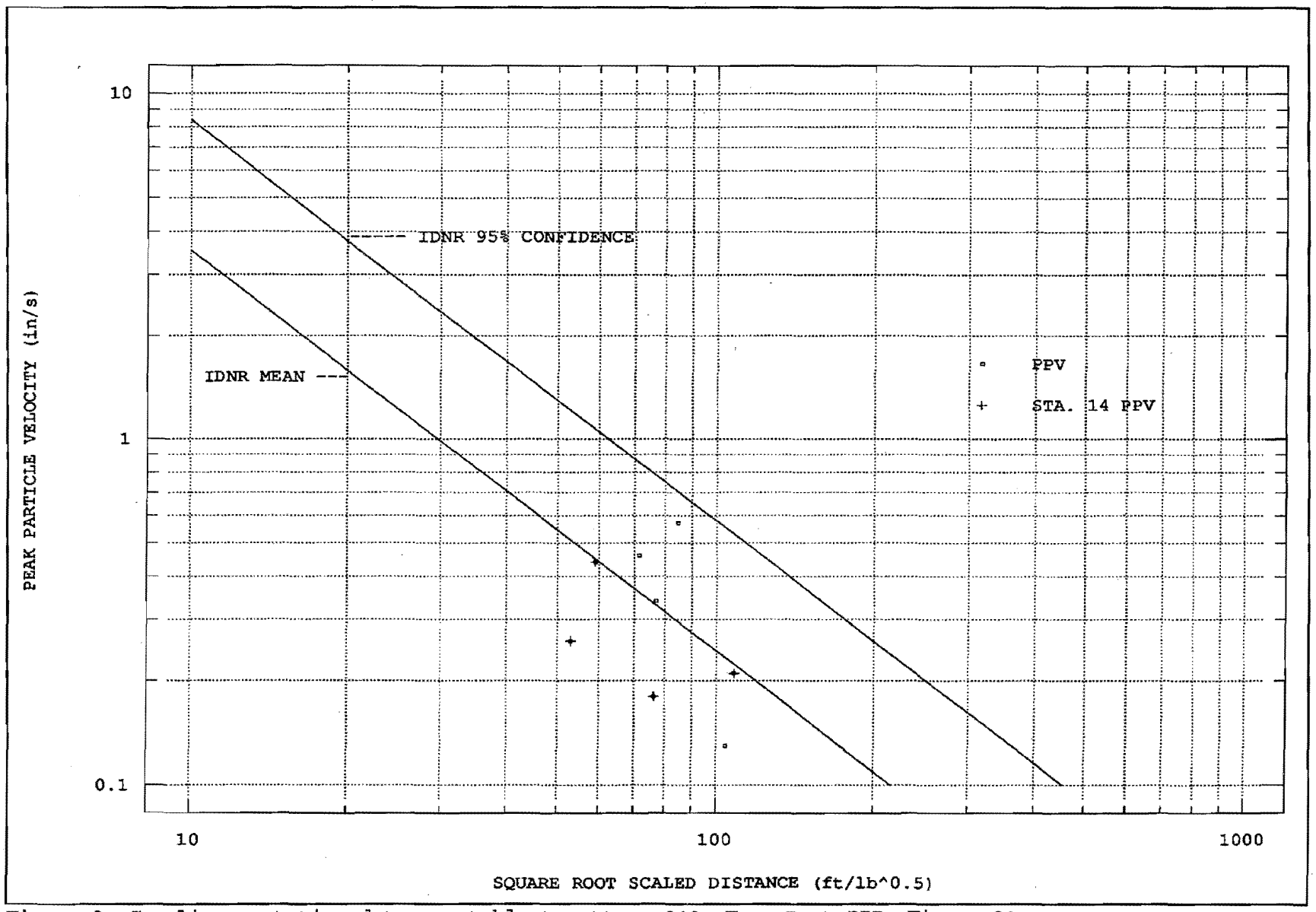

Figure 9. Compliance station data, cast blast pattern 240. From Part III, Figure 31.

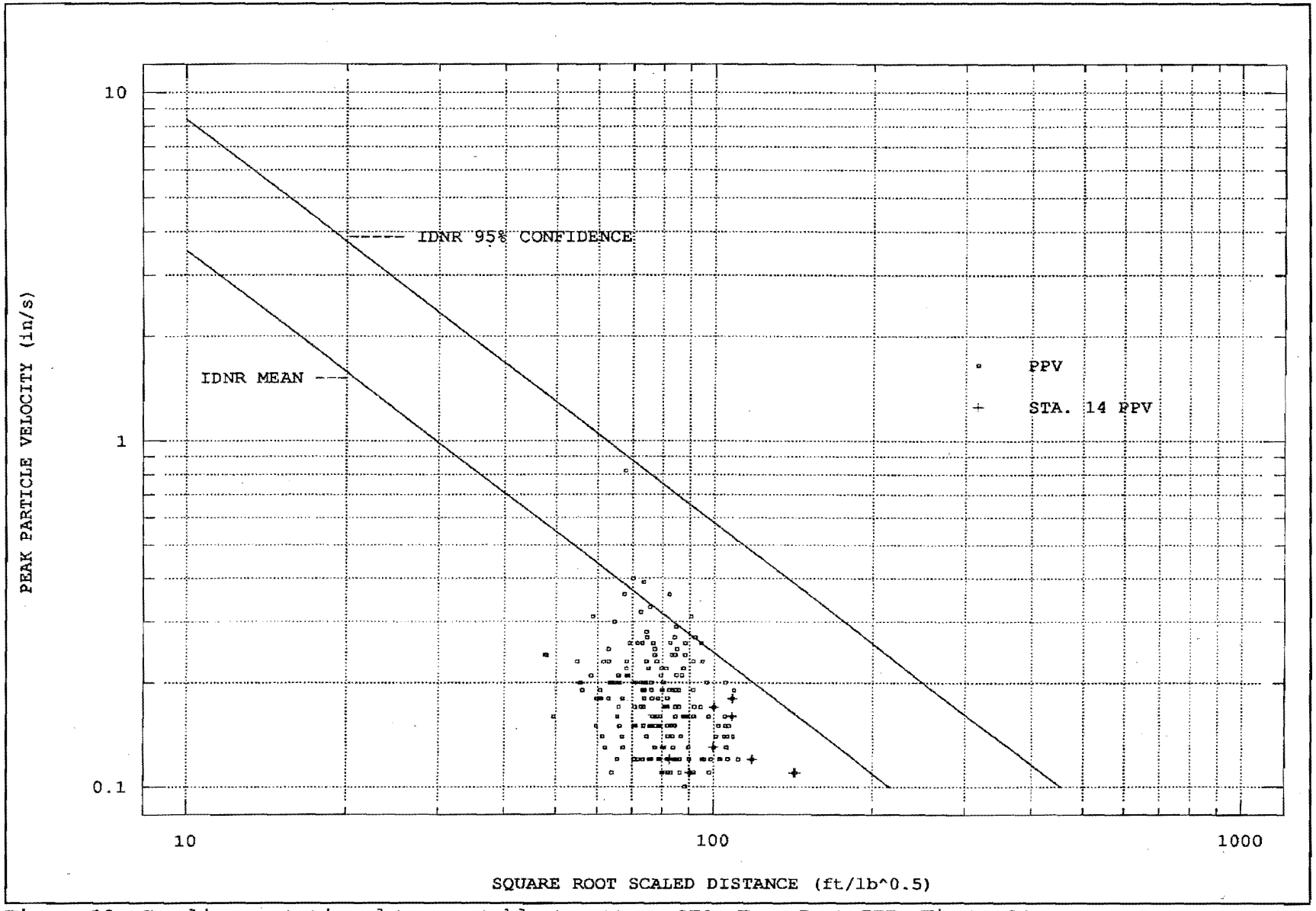

Figure 10. Compliance station data, cast blast pattern 270. From Part III, Figure 34.

3.2 To what degree do geology, soil, and topography influence the propagation of ground waves; site response amplification; and the amplitude, frequency, and duration of the waves?

The study participants recognized the need for an in-depth understanding of the influence of the geology, soils, and topography through which ground vibrations travel. Of particular interest was the potential for soils and topographic highs to amplify blast-induced vibrations and increase the risk of damage. This was addressed by USGS (Part VI). The propagation characteristics of blast-induced vibrations were analyzed by USBM in its 1989-90 investigation (Part IV). The investigation reports are excerpted and summarized in chronological order as follows:

3.2.1 The USBM 1989-90 Investigation

USBM evaluated the characteristics of ground vibrations as they propagated from the site of the mine blast. The methods and results of the evaluation are summarized as follows:

Monitoring

USBM placed seismographs with airblast channels at three houses in McCutchanville, two in Daylight, and one along Base Line Road (Figure 11). The houses were selected to represent various observed damage levels and various locations. The ground vibration sensors were aligned eastward, in the direction of the mine, but were not realigned to adjust for changing shot locations along the north-south trending highwall. The long distances between the shot and the recording stations mitigated imprecise directional alignment of the transducers and did not greatly affect peak-level measurements.

Ground Vibration Amplitudes

Historical data on peak ground vibration amplitudes were obtained by USBM from the IDNR study (December 1988-February 1989) and from recordings at structure 108 (February-December 1989). These were used in conjunction with the USBM field recordings (November 1, 1989 to January 3, 1990) to construct propagation plots in three directions from the mine: southwest towards McCutchanville (Figure 12); west towards Daylight; and northwest along Base Line Road. This enabled a direct comparison between the USBM measurements and earlier data and a determination of the seismic propagation characteristics of the area.

A seismic propagation line [$PPV=51(SD)^{-1.16}$] developed in the USBM Report of Investigation (RI) 9226 is a least squares regression fit to peak-particle-velocity amplitudes measured near the Ayrshire Mine. The amplitudes had been recorded from an east-west array of seismic stations that extended from close-in to the blast to about 6,000 feet west of the highwall in the Daylight direction. The new propagation plots in the McCutchanville and Daylight directions showed very good correlation with the RI 9226 propagation line. Because of the narrow range of scaled distances involved, the data are clustered, but where scaled distances overlap, the PPV levels are similar.

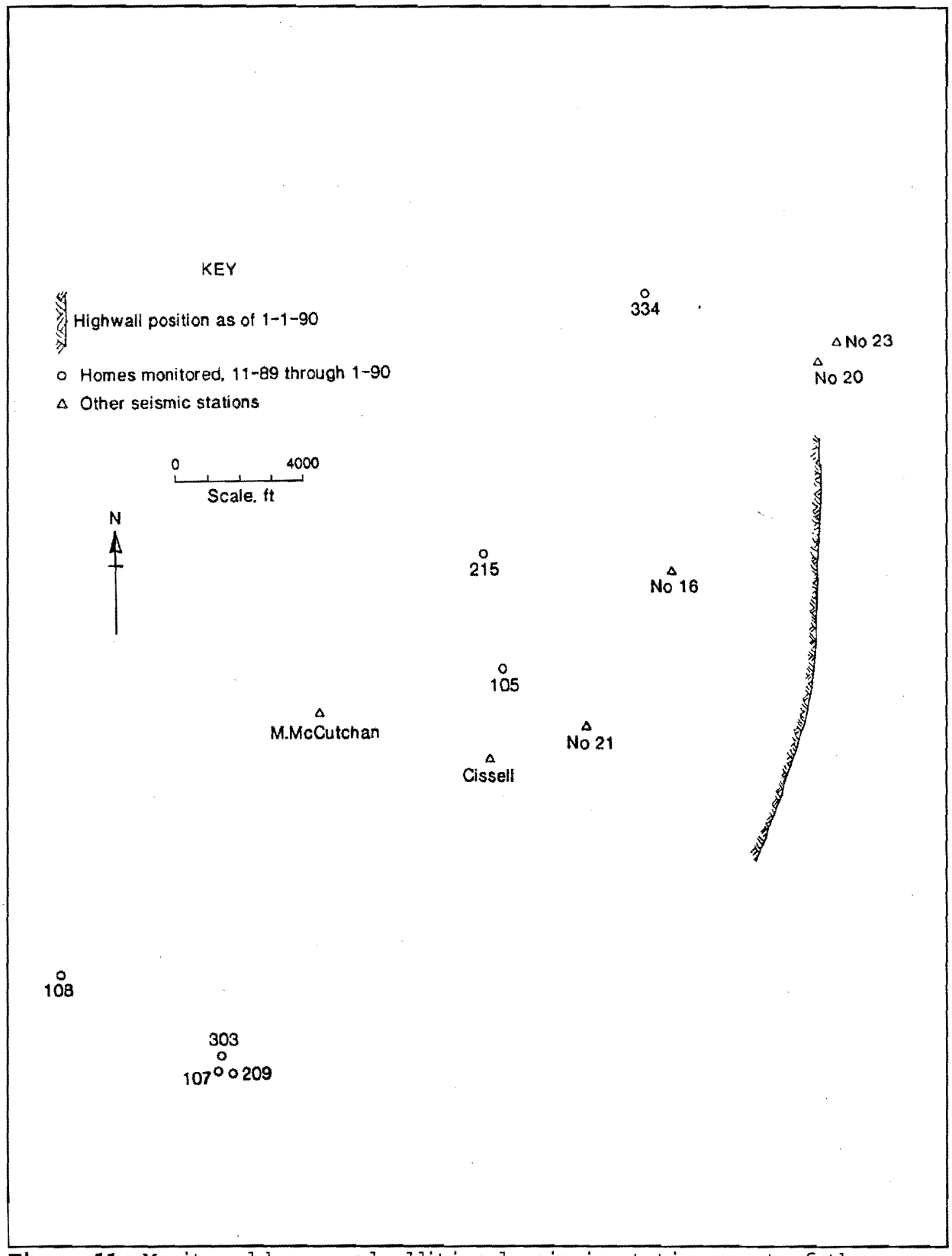

Figure 11. Monitored homes and additional seismic stations west of the Ayrshire mine highwall. From Part IV, Figure 3.

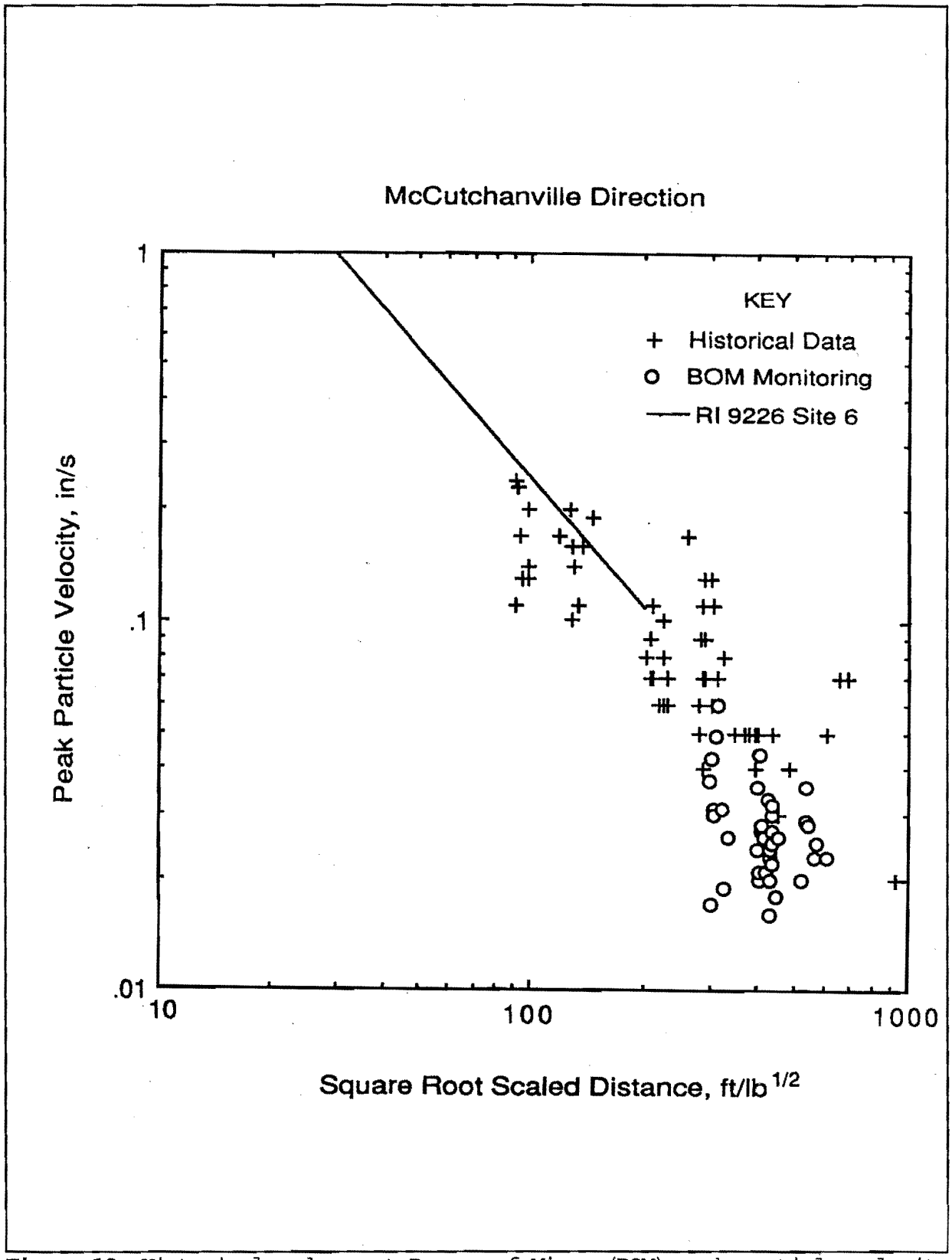

Figure 12. Historical and recent Bureau of Mines (BOM) peak particle velocity data in the McCutchanville (southwest) direction. From Part IV, Figure 16.

The USBM data showed consistently lower PPVs in both directions compared to the IDNR data at similar scaled distances.

The Base Line Road plot of historical data suggested a more efficient seismic propagation from those of the Daylight and McCutchanville directions. The Base Line Road amplitudes were higher at common scaled distances. Many of the blasts related to the Base Line Road and McCutchanville plots had the same or similar designs. Variations in blast design did not appear to be responsible for the amplitude differences.

For all three directions, the PPVs recorded during the USBM monitoring were generally lower than the IDNR data for the same scaled distance; and were near or lower than those predicted in the RI 9226 regression line. The maximum ground vibration recorded in Daylight was 0.1 ips and in McCutchanville was 0.06 ips. USBM concluded that distance from a blast and changes in site characteristics most likely accounted for the slight amplitude differences at different locations.

Ground Vibration Frequencies

The ground vibrations measured by USBM in McCutchanville had a narrow frequency range between 4 to 8 Hz. The highest amplitudes (0.03 to 0.06 ips) occurred at about 5 Hz in houses 209 and 107. Based on the distribution of peak amplitude frequencies, the characteristic frequency of the ground in McCutchanville appeared to be 5 Hz. Homes in Daylight experienced a frequency range from about 3 to 20 Hz, which is broader than in McCutchanville. Peak velocity levels of 0.1 ips occurred at about 5 Hz for house 105 and about 11 Hz for houses 215 and 334.

The homes in McCutchanville appeared to be experiencing a greater amount of narrow-band, low frequency vibrations than in Daylight. USBM hypothesized that this resulted from the longer distances from the blasts and the influence of the local geology and possibly topography. The vibration frequencies concentrated near 5 Hz were near the natural frequency of the homes. This and the influence of long durations made the effects of the ground vibrations more noticeable. However, the recorded ground vibration amplitudes were far below established threshold damage levels. The investigators further concluded that there were no conceivable blast design changes that could have raised the vibrations to damage threshold levels.

3.2.2 The USGS Investigation

USGS studied specific geologic, topographic, and soil effects on blast-induced ground vibrations. This included: (1) determining the attenuation (reduction in amplitude) of ground vibrations propagating in bedrock; (2) evaluating potential topographic enhancement effects by comparing vibrations recorded in upper and lower parts of the topography; (3) comparing ground vibrations of bedrock sites and soil sites to determine the effects of soils (or "site response" effects); (4) downhole geophysical testing; and (5) comparing ground vibrations among different soil sites. Ground vibrations from mine blasts were recorded at a total of 24 locations (Figure 13).

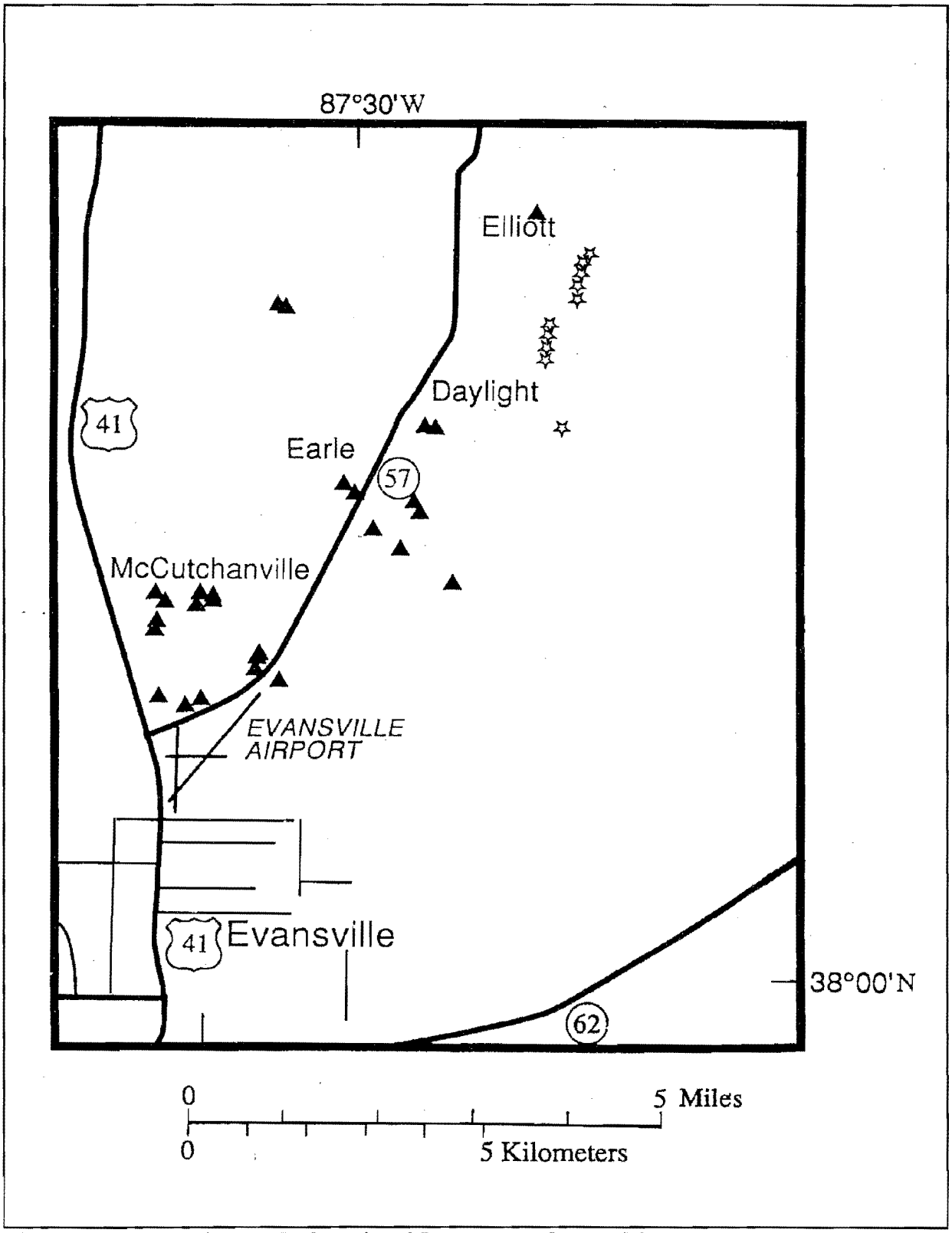

Figure 13. Locations of the mine blasts are denoted by open stars, locations of seismic recording sites are denoted by solid triangles. From Part VI, Figure 11.

Between two and six seismic systems were installed at temporary locations to record vibrations from specific blast events.

Ground Vibration Attenuation

The principal objective of this analysis was to compare the attenuation characteristics of the study area with USGS experience in another region of the country. In USGS usage, attenuation is the decrease in amplitude of vibrations as measured in bedrock based upon distance from the mine. The ground vibration amplitude difference at sites located on bedrock, at similar azimuths from the source, and at different distances from the source, are mainly due to geometric spreading of the waves and energy absorption by the propagation materials. USGS determined the attenuation rate from the mine to McCutchanville using an array of two bedrock sites. The sites were located on limestone and were approximately five and eight kilometers (three and five miles) from the mine.

Using a power-law function, an attenuation exponent of -2.04 was derived for the recorded PPVs from mine blasts in the frequency bandwidth under study (0.5-18 Hz) (Figure 14). This was slightly higher attenuation than a similarly derived exponent of -1.7 from mine blasts in a previous USGS study at Centralia, Washington. Ground vibration attenuation within frequencies similar to the natural frequencies of specific sites or buildings was of greater concern than the attenuation of PPV. Therefore, the attenuation exponents for narrow bandwidths (with increments of 1-2 Hz) were also derived. The amplitude of the frequency bandwidth near the natural frequencies of the houses attenuated more rapidly than the ground vibration PPV (Figure 15).

Topographic Effects on Ground Vibrations

Three instrument arrays were deployed to examine the possible topographic effects of hills in McCutchanville on ground vibrations. Each array utilized two to four seismographs to record vibrations on a selected hill and neighboring valley or lowland, using locations approximately equidistant from the mine blasts. Elevation differences between upland and lowland locations ranged from approximately 55 to 130 feet. Two ratios were used to compare ground vibrations of the two topographic positions. One is the PPV ratio, which is calculated by dividing the horizontal PPV of a lowland recording into the horizontal PPV of an upland recording (of the same blast event). The other is the spectral ratio, obtained by dividing the spectra (amplitudes of narrow frequency bandwidth increments) of a lowland recording into the upland spectra.

The PPV and spectral ratios represented differences in lowland and upland ground vibrations due to a summation of effects from topography and site response. The PPV ratios ranged from 1.2 to 2.5 (Table 2). The spectral ratios ranged from 2.1 to 3.6. They showed a greater amount of spectral energy in the 4-6, 6-8, and 16-18 frequency bandwidths at the upland sites.

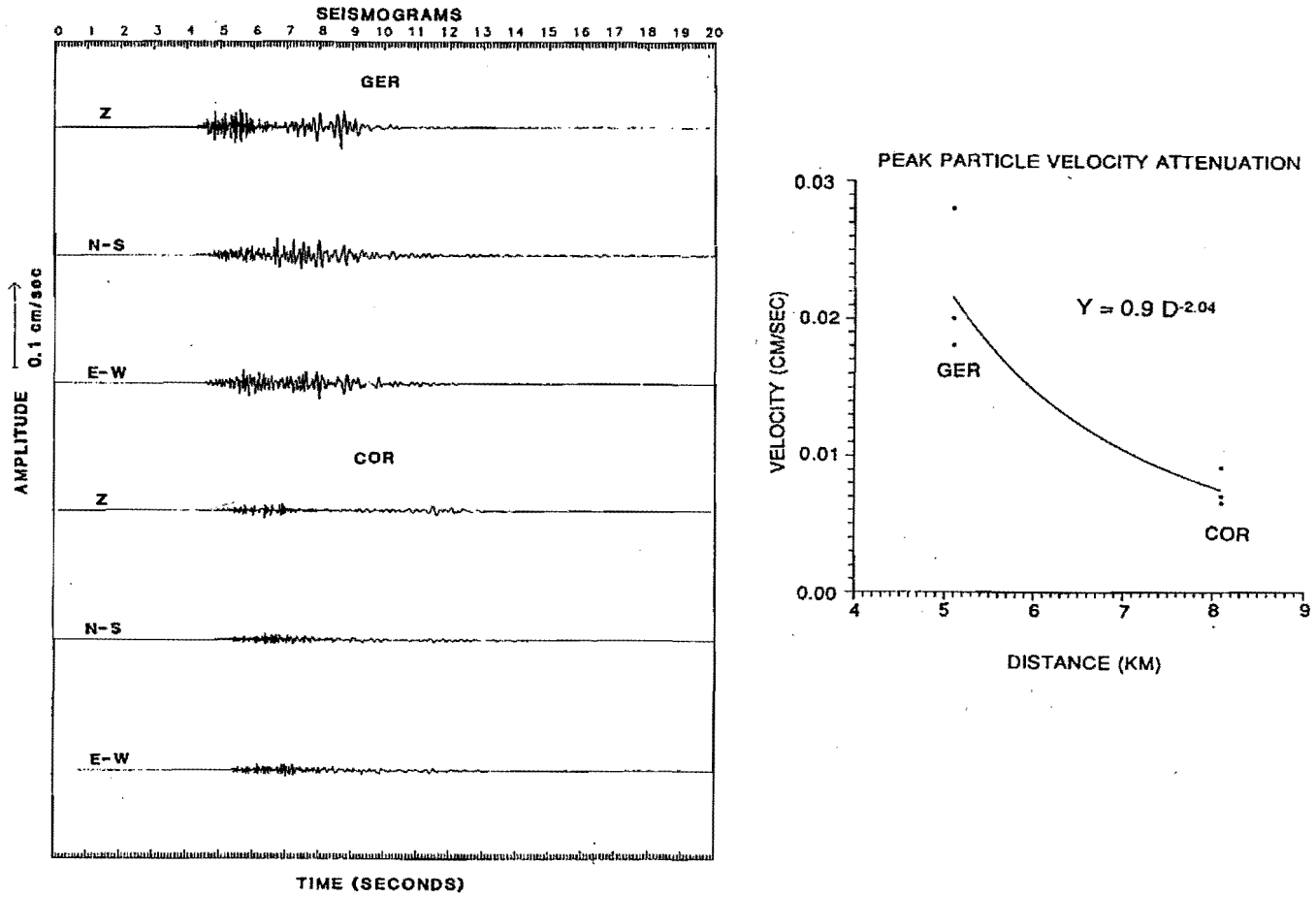

Figure 14. Seismograms and graph showing ground motion attenuation. From Part VI, Figure 13.

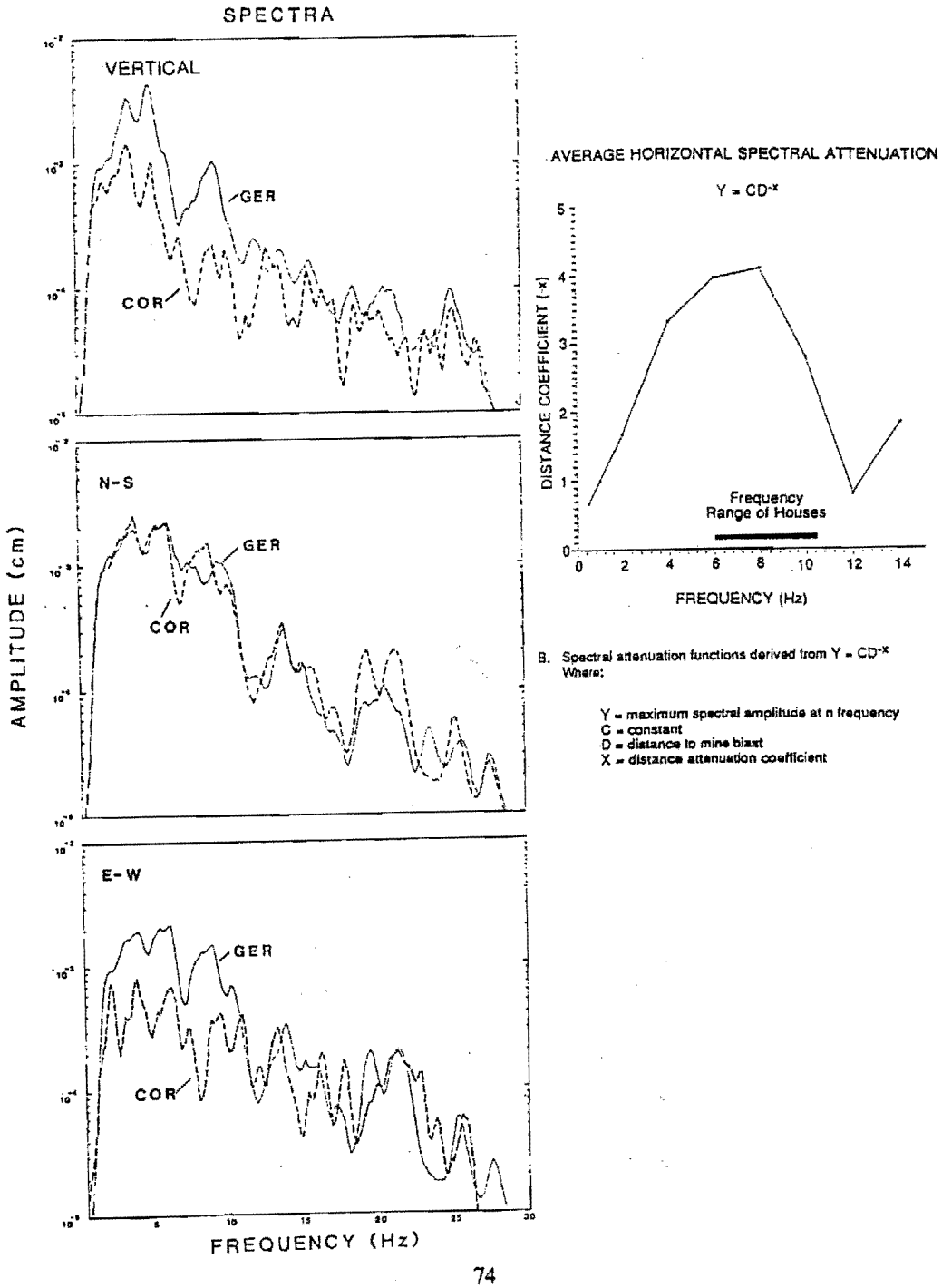

Figure 15. Spectra and spectral attenuation functions for Array No. 1. The horizontal bar in Figure B shows the bandwidth of the natural frequencies of the houses. From Part VI, Figure 14.

STATION CODE	ELEV. (feet)	DIST. (km)	PEAK-PARTICLE VELOCITY RATIO [1, 2]			HORIZONTAL SPECTRAL RATIO [3]	
			Vertical Component	Horizontal North-South Component	Horizontal East-West Component	Ratio[1]	Frequency Range, Hz
ARRAY NO. 2							
STA	380	4.1	Valley Reference Site			Valley Reference Site	
HAD	510	4.1	2.5	1.6	2.5	3.4	10 - 12
						2.5	6 - 8
ARRAY NO. 3							
HEI	390	8.2	Valley Reference Site			Valley Reference Site	
GRE	465	8.0	(1.2)	(1.2)	(1.2)	3.6	6 - 8
MCC	440	7.7	(1.7)	(1.3)	(1.3)	2.1	6 - 8
ARN	480	7.9	(2.1)	(1.4)	(1.4)	2.5	4 - 6
ARRAY NO. 4							
AIR	410	7.4	Valley Reference Site			Valley Reference Site	
FRI	465	7.4	2.1	1.5	2.5	3.2	16 - 18
						2.1	4 - 6

[1] Ratios are relative to the appropriate valley reference site.
[2] Numbers in ( ) have been corrected by the distance attenuation factor.
[3] Spectral ratios are based on the average of the two horizontal components prior to computing the ratio. Spectral ratios are not corrected for the slight differences in distance to mine blasts.

Table 2. Summary of topographic data. From Part VI, Table 5.

Site Response

Site response analysis consists of deriving PPV and spectral ratios using ground vibration time histories recorded at soil sites and a reference site 2 located on bedrock. The bedrock recordings serve as a standard for the evaluation of site specific soil effects on vibrations. The ratios are calculated by dividing the standard site PPV and spectra into the soil-site PPV and spectra. If the source, distance, azimuth, and general topographic position can be held constant, then the primary cause of difference in ground vibrations between the sites is related to the soil site's subsurface conditions.

Peak amplitude and spectral ratios were determined for a total of 11 soil sites, using 2 standard bedrock locations. The site response investigation indicated that the sites underlain by soil had a PPV amplification factor of approximately 2-4 over the ground vibration on rock (Table 3). Most of the frequencies of the higher response spectral ratios were in the 6-8 or 8-10 Hz bandwidth. The frequency of the larger values of the spectral ratios indicate the natural frequency of the soil column. The natural frequencies indicated from the spectral ratios agree with the natural frequencies determined from downhole geophysical tests (discussed below).

The site response investigation at the same sites evaluated for topographic enhancement indicated that the soil column had greater influence on ground vibrations than the topographic effect. This conclusion of USGS was reinforced when one of the rock sites was compared to a soil site located lower in topographic position. The lower soil location showed greater response.

Downhole Geophysical Testing

Nineteen holes were augered and used for geophysical logging of the soil profile. Thirteen of the holes were augered to bedrock. Eleven of these were located at complainant houses, one in a valley and one on a hill. Eight holes were located at non-complainant houses. These eight holes were augered to depths of about 2.5 feet below the bottom of the foundations. Where possible, samples of bedrock (shale or limestone) were recovered from the 13 holes augered to bedrock by split-spoon sampling at the bottom of the hole. A standard penetration test was also made at the bottom of the hole to confirm resistance. The auger holes were then cased and grouted in preparation for geophysical testing. The tests included natural gamma logging and both compressional and shear wave velocities.

The natural gamma logging helped to define the top of bedrock and concentrations of clay in the soil profile. The compressional wave and shear wave velocities helped to define the site response characteristics of the materials. Previous investigations (cited in Part VI) had found that the site amplification of ground vibration increases as the shear wave velocity decreases and that the amplification usually occurs at sites that are underlain by the thick sequence of material that has a shear wave velocity below approximately 150 m/s (142 ft/s). However, the test results did

STATION CODE	ELEV. (feet)	DIST. (km)	PEAK-PARTICLE VELOCITY RATIO [1,2]			HORIZONTAL SPECTRAL RATIO [3]	
			Vertical Component	Horizontal North-South Component	Horizontal East-West Component	Ratio[1]	Frequency Range, Hz
ARRAY NO. 5							
COR	490	7.0	Rock Reference Site			Rock Reference Site	
GRE	465	8.0	(1.3)	(3.4)	(3.5)	6.7	6 - 8
WOF	430	8.0	(1.3)	(3.2)	(3.0)	3.3	6 - 8
KIN	425	8.2	(1.4)	(3.0)	(3.3)	3.3	6 - 8
MCC	440	7.7	(1.2)	(2.6)	(3.7)	4.3	6 - 8
ARN	480	7.9	(1.3)	(3.4)	(3.8)	4.1	6 - 8
EFF	435	8.3	(1.4)	(3.7)	(3.2)	4.7	6 - 8
FRI	465	7.1	(1.0)	(2.6)	(2.6)	5.0	8 - 10
FIN	470	7.1	(0.9)	(1.3)	(2.1)	9.2	8 - 10
ARRAY NO. 6							
GER	520	4.1	Rock Reference Site			Rock Reference Site	
STA	380	4.1	0.4	1.6	0.9	2.2	5, 13
ENG	395	4.1	1.2	1.6	1.7	4.9	12 - 14
HAD	510	4.1	1.1	2.6	2.3	6.8	12 - 14

[1] Ratios are relative to the appropriate rock reference site.
[2] Numbers in ( ) have been corrected by the distance attenuation factor.
[3] Spectral ratios are based on the average of the two horizontal components prior to computing the ratio. Spectral ratios are not corrected for the slight differences in distance to mine blasts.

Table 3. Summary of site response data. From Part VI, Table 6.

not find any of the materials to have shear wave velocities as low as this value.

The natural frequency of the soil was calculated from the shear wave velocities and the depth of the soil profile. The natural frequencies ranged from 4.8 Hz at the site of deepest low velocity material (i.e., soil) to 8.3 Hz at the site of shallowest low velocity material. The range of soil natural frequencies closely corresponds with a natural frequency range of 5.6 to 10.5 Hz for all of the 21 structures that were tested by USGS (see Section 3.5.2 for further discussion on this subject).

Site Comparisons

Six pairs of sites were used in the comparison studies. Each pair consisted of (1) a site at which the home owner had made an official complaint due to suspected vibration damage and (2) a house that was proximate to the complainant house and at which the resident had not made an official damage complaint (see Table 4 for a list of complainant and non-complainant structures used in the USGS investigation). For each pair, an attempt was made to select a non-complainant house within a few hundred meters (a few hundred yards) of the complainant house to minimize differences in geology and topographic position. For each site seismographs were located 15-20 feet from the structures in order to measure "free-field" ground vibrations, i.e., ground vibrations unaffected by structure response.

The time histories from two closely-spaced ground sites will have certain variations because of slight differences in the seismic recording systems, differences in the coupling between the seismometer and the ground, variances in the readout, and differences in the geophysical properties of the soil. USGS had estimated the maximum spectral ratio variation due to the first three differences to be approximately 15 percent. The duration of blast-induced vibrations were compared by measuring the time during which their amplitudes exceed those of ambient (non-blast-related) vibrations by at least 40 percent. For all site pairs the duration difference was less than one second. PPV ratios were obtained by dividing the companion non-complainant PPV into the complainant PPV. Most of the differences were less than 30 percent. USGS did not consider those differences significant. Larger differences were recorded for two complainant sites, with a PPV ratio of 1.6 at house 301 and a ratio of 1.5 at house 107.

3.2.3 Findings

USBM confirmed the general blast vibration propagation characteristics of the study area which were initially reported in RI 9226. Relatively efficient seismic propagation occurred in a northwestern direction along Baseline Road.

Recordings made by USBM in November 1989-January 1990 were almost consistently lower in amplitude than the IDNR data gathered between December 1988 and January 1990. Two potential explanations for this observation are: (1) a change in geological effects on blast vibrations caused by the westward migration of the mine (i.e., of the blast sites); and (2) changes in blast design. When the USBM investigators compared recorded amplitudes for the

103	Zimmerman (Complainant)
103A	Shelton (Companion)
107	Harris (Complainant)
107A	Deutch (Companion)
107B	Hawles (Companion)
108	McCutchan (Complainant)
108A	Zinn (Companion)
108B	Arnold (Companion)
111	Brinker (Complainant)
113	Boettcher (Complainant)
113A	Ogg (Companion)
113B	Lavallo (Companion)
114	Kinney (Complainant)
114A	Wolff (Companion)
115	Christensen (Complainant)
115A	Board (Companion)
115B	Klausmeier (Companion)
118	Hoover
201	Effinger (Complainant)
202	Richey (Complainant)
202A	Stevens (Companion)
202B	Heil (Companion)
209	Gore (Complainant)
301	Fink (Complainant)
301A	Condict (Companion)
302	Greenfield (Complainant)
302A	Palmer (Companion)
316	Gorbett (Complainant)
401	Poston (Complainant)
403	McCutchan, W. (Complainant)
411	Carter (Complainant)
421	Osborn (Complainant)
421A	Rozanski (Companion)
KK1	Stahl
KK2	Oasis

Table 4. Study area, complainant and companion (non-complainant) structures.

same or similar blast designs between the McCutchanville and Baseline Road directions, they concluded that factors other than blast design caused the higher amplitudes along Baseline Road. However, this does not explain the amplitude differences between the USBM and IDNR data in each plot. The IDNR data are connected with blast designs and locations that did not occur during the USBM 1989-90 monitoring period. The actual cause(s) for the relatively low USBM amplitudes cannot be isolated with certainty.

The USGS investigators found the attenuation rate of blast vibrations in bedrock to be comparable to another region monitored under a previous study. They concluded that attenuation in the study area was not unusual. Topographic enhancement of vibrations was determined to be negligible. Soils in the study area amplified vibrations from bedrock by a factor of 2-4. The dominant frequency bandwidths of the amplification corresponded to the natural frequency of the soils.

The results of USBM and USGS indicated that the natural frequency range of the soils closely matched that of the houses. This contributed to the discernability of the blast vibrations experienced by the residents. When the two frequencies match, structure response increases and lower ground vibrations can result in damage. The problem of whether the mine blasting ever resulted in sufficient amplitude to generate damage is covered in Section 3.5.

The USGS investigators determined that ground-response variations within most of the six site pairs investigated were negligible. The sites of two complainant homes exhibited greater response over their companion non-complainant sites by factors of 1.5 and 1.6. This indicates that there are some locations in the study area where site conditions can contribute to different structure responses to blasting or other sources of ground vibration.

3.3 Have there been significant ground vibrations occurring at frequencies below 1 Hz?

The sensitivity of the USBM seismographs used during the 1989-90 investigation ranged from 1 to 5,000 Hz. OSM requested the WES Structures Laboratory to monitor vibrations below this range and determine whether the vibrations could be responsible for damages. The summary below is derived from Part VII of this report.

3.3.1 WES Monitoring and Analysis

The WES investigators monitored free-field ground vibrations near house 103 in Daylight using seismic accelerometers with flat frequency responses down to 0.5 Hz. The measurements took place in Spring 1992. Structure response to the vibrations were measured with seismic accelerometers with a useful frequency range of 0.3 to 100 Hz.

Vibrations below 4 Hz produced no measurable response in the house. Above 4 Hz the structure began to show some amplification of ground vibration and the largest amplifications occurred at frequencies of 7 to 15 Hz. There were isolated cases where amplification occurred in response to vibrations above 15 Hz. The WES investigators concluded that the house responded as a rigid body to vibrations below 4 Hz and moved with the ground without developing significant internal stresses.

3.3.2 Findings

Based on the above analysis and findings from other parts of the study (see Sections 3.2 and 3.5), vibration levels with any potential of causing damages in the study area did not fall within very low frequency ranges. Ground vibrations below 4 Hz were neither within the natural frequencies of the ground nor the buildings. There was no evidence of low frequency vibrations causing internal stresses in the WES monitoring and analysis program.

3.4 Have airblasts caused damages in the McCutchanville-Daylight area?

Summaries of two USBM investigations are presented below. The first investigation entailed the USBM ground vibration, airblast, and structure-response monitoring from November 1, 1989 to January 3, 1990 (Part IV). Parts of the final report relevant to this issue are covered in this section first. One observation made in this USBM report concerned a lack of historical airblast data. This resulted in some uncertainty regarding conclusions made on the effects of airblasts on structures in the study area. Consequently, the second investigation (Part V) took place in which additional airblast and structure-response monitoring was conducted at buildings most exposed to airblast effects. USBM also monitored airblasts at other locations to assess propagation into the study area. This work is also summarized below:

3.4.1 USBM 1989-90 Investigation

Airblast Monitoring

USBM placed seismographs with airblast channels in three houses in McCutchanville (107, 209, 303), two in Daylight (105, 215), and one on Base Line Road (334). In McCutchanville, two houses were located on east-facing slopes (towards the mine) for maximum airblast-induced structure responses.

The airblasts recorded in the McCutchanville and Daylight directions were highly variable, the vast majority of measurements falling between 90 and 120 dB. The highest airblast recorded by USBM, using a 5-Hz system, was 121 dB at house 334. The 121 dB airblast was very noticeable although still well below the 129 dB damage threshold established by USBM in RI 8485 (citation in Part IV). In the historical data, peak values were 125 dB in McCutchanville and 118 dB in Daylight.

Because airblasts travel more slowly than ground vibrations, airblasts arrive at a given location following ground vibrations by a time proportional to the distance from the blast. Increased duration of structure response due to the combined effects of airblast and ground vibration tends to increase human perception. Therefore, even low-level and inaudible airblasts could have been partly responsible for complaints.

Airblasts at houses 105 and 209 were also recorded using wide-band sonic-boom systems. The airblasts recorded during the USBM monitoring were characteristic of levels recorded at large distances. Most of the signal energy was near or below 1 Hz.

Structure Response to Airblasts

Airblast-induced structure responses were recorded for a few blasts in the two instrumented homes. Because of their relatively low dominant frequencies (less than 1 Hz, which is consistent with long distances and locations behind a highwall face), the airblasts produced responses generally lower than those seen in the historical data. Peak structure response amplitudes of 0.004 and 0.037 ips were recorded at homes 105 and 209, respectively. The low

structural height of house 105 probably contributed to its lower response. USBM concluded that these magnitudes should have been noticeable by persons inside a home, but should not have induced damage.

A proper assessment of past airblast impacts was not possible. This was because airblast measurements either did not exist for most events labeled "severe" by complainants or were obtained too far away to be of any use. The USBM investigators concluded that airblasts remained a possible contributing factor to threshold damages. However, USBM cited a lack of widespread glass breakage in the study area, making it unlikely that 140 dB had ever been exceeded. This value also represents the threshold for plaster cracking. There is no chance that airblasts below the glass breaking threshold would cause foundation cracks.

3.4.2 USBM 1991-92 Investigation

The USBM investigators conducted additional airblast and structure-response monitoring to address the following issues:

(1) **What is the dynamic response of large structures to impacting airblast?**

(2) **Does an abnormal response occur at one relatively nearby and new structure, a large church, and is it responsible for cracks in structural masonry?**

(3) **What are the responses from airblast at larger distances, and how does the airblast amplitude change with weather influences?**

USBM chose sites for geographical diversity and to test responses of structures which are larger than the homes studied in the first investigation (Figure 16). The St. John's Church (building 119) in Daylight is a large span, recently built structure about two miles from the highwall. Because of the large eastern-end activity room, its response to airblast was expected to be above average. The Blue Grass Church (building 224), situated northwest of the mine, also had a relatively large area of exposure because of a steep roof. A large home in McCutchanville (house 118) on an exposed easterly slope was selected to assess the effects of long-range airblast propagation. Seismographs were placed at two other sites to provide data for propagation plots. One is next to compliance station #16 and is close to the highwall. The other is situated at house 202 on an exposed hillside beyond the Blue Grass Church.

The field monitoring program included the measurement of ground vibrations at the building sites. The vibration data enabled the researchers to discriminate structure response to airblasts from response to ground vibrations.

Airblast Monitoring

Eight seismographs were employed during the field monitoring. Each instrument was a 4-channel, self-triggered seismograph with frequency ranges

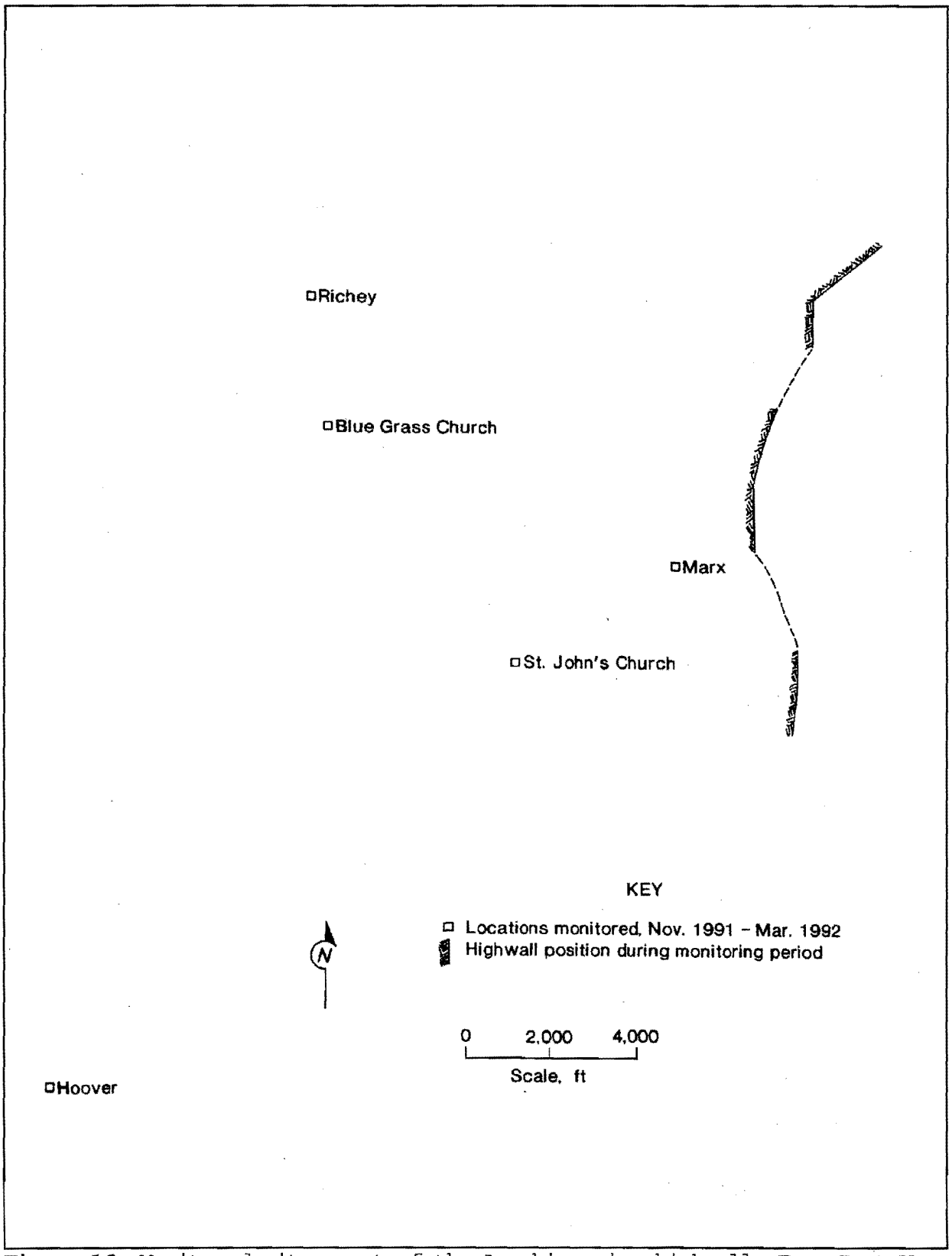

Figure 16. Monitored sites west of the Ayrshire mine highwall. From Part V, Figure 2.

of 2-200 Hz for both vibration and airblast, a 54 dB dynamic range, dual triggers with 0.01 ips and 1 dB selectivity, a 1/2-second pretrigger, and sufficient solid-state memory for 300, 9-second events.

The USBM investigators installed two seismographs at each of the three buildings for structure response. One seismograph was placed at ground level to monitor ground vibrations and airblast. The second was used to measure structure response with a three-component vibration sensor mounted high in an exterior corner or near the peak of the roof line. An airblast microphone was also mounted high on the structure. Regardless of which channel triggered the seismograph, all four channels were activated. The objective of the monitoring was to quantify racking (whole-structure distortional response) and compare the results to historical data on dynamic response.

A second objective was to examine airblast propagation and compare the acquired data to wind direction and speed. The two additional monitoring stations used for this analysis were roughly in line with the building sites, defining two arrays trending northwest and southwest from the approximate center of the highwall.

The monitoring program took place from November 13, 1991 to March 3, 1992. Trigger levels were first set at 0.1 ips for vibration and response; and 125 dB for airblast. The relatively high airblast trigger was needed to minimize false triggers. After review of initial results on January 9, some triggers were adjusted. The ground-vibration triggers at the three building site units and the structure-response triggers at two building sites were set to 0.02 ips. The structure-response trigger at building 119 was increased to 0.15 ips to filter events generated during basketball games in the activity room (on the other side of the wall).

Results and Conclusions

Airblasts, ground vibrations, and structure responses were very low during the monitoring period. The results obtained from each seismograph station are as follows:

- **St. John's Church (119):** There were 10 blast-induced structure responses exceeding 0.15 ips. Only five of these were accompanied by triggered ground vibration data. No airblasts exceeded the trigger thresholds of 125 dB. Basketball and other human activity produced hundreds of vibrational responses on the monitored east wall. The larger of these responses were estimated to be equivalent to those which would result from airblasts in the range of 130 dB.

- **Hoover (118):** No airblasts at this structure exceeded 125 dB and no airblast-induced structure responses exceeded 0.1 ips.

- **Blue Grass Church (224):** No structure responses exceeded the threshold triggering levels and all ground vibration amplitudes were at or below 0.04 ips. There were no airblasts or airblast-induced responses above the trigger thresholds.

- **Marx (station 16)**: This site was closest to the mine highwall and resulted in 58 vibration and airblast recordings. This was the only location where significant airblasts were measured, the highest being 120 dB.

- **Ritchey (202)**: The accuracy of all airblast recordings were suspect due to high wind turbulence on this hilltop location. Any true airblast would likely have been recorded outside the nine-second window for a vibration-triggered event. None of the maximum airblasts exceeded the 125 dB trigger level.

The USBM investigators found it impossible to make any meaningful interpretations on airblast propagation and weather effects. The airblast data was too sparse and low in amplitude. The few airblasts that produced measurable responses resulted in slightly greater structure responses than the average for single-family homes monitored in previous studies. This had been expected for taller structures with large surface areas exposed to the airblast wave front. However, the values are not above the range of amplitudes measured earlier.

3.4.3 Findings

The sum of the USBM work in this study did not result in any evidence that airblast produced by surface mine blasts ever caused damages in the study area. Airblasts monitored during this study never exceeded 125 dB. However, the lack of meaningful data identified in the first investigation has not been solved. There is still some uncertainty pertaining to the effects of airblasts on structures. The nature of airblast does not allow accurate prediction of levels during non-study periods. No evidence exists that airblast caused structure response sufficient to induce damage during the investigations.

3.5 What is the source and nature of the "lateral force" observed to be a causative factor of certain damages such as horizontal cracks?

The interagency site review in February 20 and 21, 1991, identified particular kinds of damage that could not be readily ascribed to soil or foundation problems. These included long horizontal cracks near or above ground level and other failures responding from some sort of lateral force. This lateral force, if present, could have acted on structures through ground vibrations, airblasts, or natural causes.

This section contains excerpts and summaries of all work in this study pertaining to the following question: Is there evidence that a dynamic force, from blasting or some other source, directly caused damages in the study area? The USBM Part IV investigation is reviewed first, followed by summaries of more recent work pertinent to this issue. The other work, which took place in 1991-92, includes: (1) the USGS analysis of building natural frequency and amplification of ground vibrations (Part VI); (2) the USGS evaluation of earthquake damage potential in the study area (Part VI); (3) the USGS comparative inspections of complainant and non-complainant houses (Part VI); (4) the WES structure-response monitoring and modeling program (Part VII); and (5) the WES analysis of horizontal basement wall cracks near the ground line (Part VII).

3.5.1 The USBM 1989-90 Investigation

The part of the early USBM investigation that is relevant to this issue is an assessment of vibration and airblast effects on structures based on structure-response monitoring; and analysis of cracks in construction materials. The methods and results are summarized as follows:

Structure Response to Ground Vibrations

House 105 in Daylight and house 209 in McCutchanville were instrumented by USBM with seven-channel tape systems to monitor above-ground structure response induced from the blast vibrations. Sensors for measuring the corner vibration were placed in the main living areas of the homes, directly above the sensors installed to monitor ground vibration. Data gathering at house 209 was supplemented by a third transducer placed several feet away from the corner on an inside window-frame located on the east-facing wall for "midwall response". Structure responses to specific ground vibration events were identified by referring to the approximate time frames of the blasts.

Related ground vibration and structure response time-histories in both houses were very similar, except for an amplitude increase in the structure. The maximum response amplitude recorded at structure 209 was 0.096 ips. In addition, some high frequency "bumps" were observed on the time history, which probably were modifications induced by specific characteristics of the structure (such as the materials and methods used in construction). The maximum response amplitude recorded at the midwall was 0.112 ips. The midwall response to the ground vibrations in the east-west direction was almost identical in shape and duration to the east-wall corner motion.

Ground-to-structure amplifications averaged nearly 2.0 at the corners and midwall amplifications ranged up to 3.0.

House 105 had a typical amplification factor of 1.3 and a maximum of 1.6. The maximum response amplitude at the corner was 0.110 ips. This house was subjected to a relatively wide range of ground vibration frequencies (3-20 Hz), as were all the homes in Daylight.

USBM found that all of the measured response values in houses 105 and 209 were within the range of those measured in homes during previous studies. Homes 105 and 209 could not be considered abnormal in terms of their responses to blast-induced vibrations. The established ground-vibration threshold for cracking plaster, the most sensitive building material, is 0.5 ips (2.0 ips structure response). The maximum structural amplitudes actually recorded were well below this value.

Structure Response to Airblasts

The USBM procedure for monitoring the response of two houses to airblasts is summarized in Section 3.3. The investigators concluded that the recorded peak structure-response amplitudes of 0.004 and 0.037 ips should have been noticeable by persons inside a home, but are well below any thresholds of damage.

Structure Responses to Human Activity

While the instruments were in place in the McCutchanville house 209, USBM personnel also recorded structure responses to aircraft operations and human activity. Aircraft-induced rattling was noticeable and produced midwall vibrations of lower amplitude than the strongest blasts measured during the monitoring period. The responses to human activity were comparable to the strongest blasts at the corners and were far greater than the strongest blasts at the midwalls. These results agree with previous studies.

Crack Monitoring during Blasting Operations

USBM inspected 45 cracked areas 38 times in the 6 monitored homes after every blast. The 1,710 inspections were made between November 1, 1989 and January 3, 1990. The selection of inspection areas concentrated on those areas with the highest estimated risk of damage, such as above doorways, and those with high probability of visible change. All inspected areas were inside the homes and most of them involved cracks in wallboard. A few masonry cracks were also monitored but the rough surface textures made assessments of crack tip locations difficult. This was less of a problem for crack widths.

Of the six homes monitored, four had minor changes in crack widths. One home experienced an extension of a crack which was not one of those preselected for monitoring. USBM found that the cracks generally opened and closed without responding to blasting activity.

House 105 had a crack which became slightly wider (by 0.1 millimeter or 0.004 inch) after a blast but returned to its original width the next morning. During three successive inspections, this crack steadily widened until it reversed back to its original width. House 107 had a ceiling crack, not preselected for inspection, which extended through a mark identifying the tip of another crack. The highest vibration level during the time period in which the crack extension was noticed was 0.031 ips. The highest vibration level recorded at this home during the monitoring period was 0.06 ips, which produced no observed changes.

House 209 had a crack which opened and closed just at the resolution of measurement, $\pm$0.05 mm. At least one change occurred during a period of time when no blasts were taking place. Another crack in this home all but disappeared after a very cold spell of -19°F. At the same time, a concrete driveway outside the walk-in basement lifted enough to prevent the opening of a door. The door could be opened a few weeks later when the temperature had increased by 60°.

House 215 had two cracks which cycled $\pm$0.10 mm. This house, like the others, had cracks which widened and closed both during blasting and, in three of eight cases, during non-blasting times. These events also appeared to be a reaction to temperature extremes.

The USBM investigators concluded that there was no clear correlation between blasting and the observed crack changes. Based on the above information and previous studies by the USBM, they considered weather-related influences the most likely cause of the cyclic changes in crack width.

3.5.2 USGS Assessment of Building Response

Building Natural Frequency

USGS investigated the natural frequency of 21 houses and structural amplification of ground vibrations in 2 houses. The procedure for obtaining these parameters consisted of installing portable horizontal seismometers on the top and at the midpoint of load-bearing walls of each structure. The natural frequency of the structure was determined by recording the induced vibration into the building by body movement of a person in close synchronization with the structure's approximate natural frequency.

The natural frequencies of the short-axis of the one- and two-story houses ranged form 5.6 to 10.5 Hz, which is similar to the USGS results from other investigations. The natural frequencies of non-complainant houses were within 2-3 Hz of the complainant houses. The general correspondence between the soil and building natural frequencies is covered in Section 3.2. USGS noted particularly close matches (within 0.2 to 0.5 Hz) with 2 houses (115 and 301) (Table 5).

Building Amplification of Vibrations

To assess structural amplification, vibrations induced by mine blasting were recorded on the ground and simultaneously responses were measured in the

SITE CATEGORY - SITE CODE	OWNER	SITE FREQUENCY Hz	BUILDING NATURAL FREQUENCY, Hz	
			Short Axis	Long Axis
1 - FRI	Harris	5.1	7.1	7.2
1 - FIN	Fink	5.8	5.6	6.5
1 - MCC	McCutchen	5.1	9.7	No Data
2 - ZIN	Zinn	6.9	8.3	14.6
1 - OSB	Osborn	8.6	10.5	14.5
1 - BOE	Boettcher	6.4	7.2	8.6
1 RIC	Richey	7.3	6.6	No Data
1 - EFF	Effinger	5.4	8.7	9.9
1 - GRE	Greenfield	5.4	9.3	13.6
1 - ZIM	Zimmerman	5.9	9.9	No Data
1 - CHR	Christensen	7.2	6.8	No Data

Table 5. Comparison of natural frequencies for sites and buildings. From Part VI, Table 10.

attics of two-story houses. Measurements were made above load-bearing walls. Recordings from the attic sites had PPVs in the horizontal plane about 4-4.5 times the free-field ground vibrations and greater durations by approximately 4 seconds. The USGS investigators observed two notable resonances in house 107 (Figure 17). The vibration amplification was nearly equal in both horizontal axes. This was not surprising since the natural frequencies were about the same. Also, an amplification factor of approximately 3 occurred in the vertical axis. USGS stated that most structures have very low amplification in the vertical axis; and suggested that this house might have been more sensitive to vibrations than normal.

3.5.3 USGS Building Damage Inspections

USGS conducted walk-through, structural engineering inspections of 52 houses in the vicinity of Evansville. The purpose of this field exercise was to identify possible causes of damage based on visual observation. This summary emphasizes observations pertinent to the possible effects of the vibrational forces of mine blasts and earthquakes. Observations of USGS pertaining to other potential causes of damage are covered in Section 3.9.

Inspections were performed on three categories of houses located in the study area and a remote area west of Evansville and approximately 10 miles from the mine. The categories are:

- **Category 1:** Homes in the study area from which formal blasting complaints had been made by the owners and/or occupants;
- **Category 2:** Homes in the study area from which formal blasting complaints had not been made by the owners and/or occupants;
- **Category 3:** Homes in the remote area where blasts were not felt by the occupants.

Inspections of Categories 1 and 2 Houses

Thirteen Category 1 and twenty Category 2 houses were inspected. Category 2 houses were selected as comparisons to Category 1. When possible, each house in Category 1 was matched with a nearby Category 2 house with similar type structure, foundation construction, and site conditions. Ideal matches in all these parameters were not always possible.

Most residents of the complainant homes contend that the severe damage had not appeared until the onset of cast blasting in 1988. All of the Category 1 and 2 residents felt the blasting and some described it as severe. People from two homes described certain cracks as resulting from specific blasting events.

The Category 1 structures were damaged to widely different degrees. In general, the damages in Category 1 houses were more than expected for well-constructed houses subjected to normal seasonal variations with no foundation

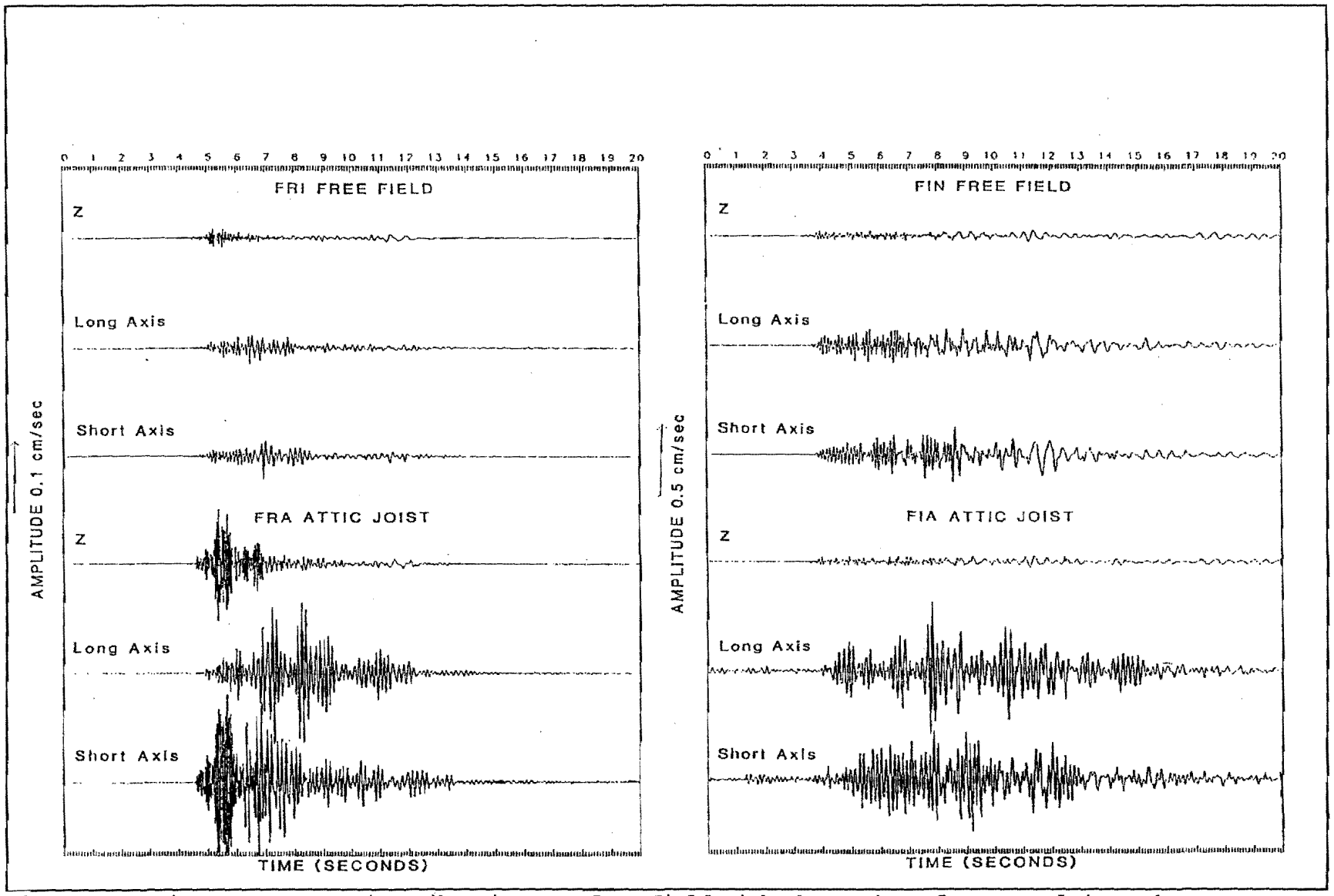

Figure 17. Seismograms comparing vibrations at free-field with the attics of two complainant homes. From Part VI, Figure 29.

problems. Some of the more severely damaged of these structures had Category 2 companions with little or no damage. A few of the houses in Category 2 had damage as severe as some of the structures in Category 1.

Inspections of Category 3 Houses

Nineteen Category 3 houses were inspected. These houses were used as a control group. The remote area was selected to match as closely as possible the site conditions of the study area, but to be far enough away from the mine as to preclude damage from blast-induced vibrations. Damage in the remote area was assumed to have resulted from causes other than blasting. Damages in the Category 3 houses were common. The damage was not as severe as in some of the structures in Category 1 and not much different from that in Category 2. One structure in Category 3 had severe damage attributed by the owner to faulty construction. Two home owners identified specific cracks as a consequence of earthquakes.

Causes of Damage

Because the houses are different and are on different foundations, there may be no single explanation for all the damages observed. Presumably, similar houses subjected to similar vibrations exceeding threshold damage levels would have similar damages, barring differences in foundation conditions or other factors. Significant differences in damage level between Category 1 houses and their Category 2 companions, however, do commonly occur. USGS concluded that the severe damage is not related to blasting without other contributing factors. The reasons include indications of soil-related problems (see Section 3.9) and the observation that the blast vibrations levels monitored during this study have been well below historical data associated with severe damage. The houses with relatively simple shapes and crawl spaces had less damage than those with the more complex shapes and basements.

The causes of threshold damage such as hairline cracks observed during the inspections often could not be isolated. Many structures may have this type of cracking from normal conditions. The vibration amplitudes estimated by OSM as a worst-case scenario may have been sufficient, considering the scatter of historical data for this level of damage to occur (Figure 18). Consideration of the USGS findings on site response and building natural frequencies (see Section 3.2 and above) strengthens this point of view. However, the effect of earthquakes (summarized in Section 3.5.4) should not be ruled out either.

USGS pointed out that more than one factor may contribute to damage. For example, load caused by a combination of vibration-induced stresses imposed on top of existing settlement stresses might be sufficient to cause cracking, when neither condition alone would be enough to do so. It is the total stress level, regardless of the cause(s), that results in damage. Usually, the magnitude of the loads must be known in order to separate the stresses due to various effects. The issue is complicated by the fact that there were cracks present in virtually all of the structures inspected.

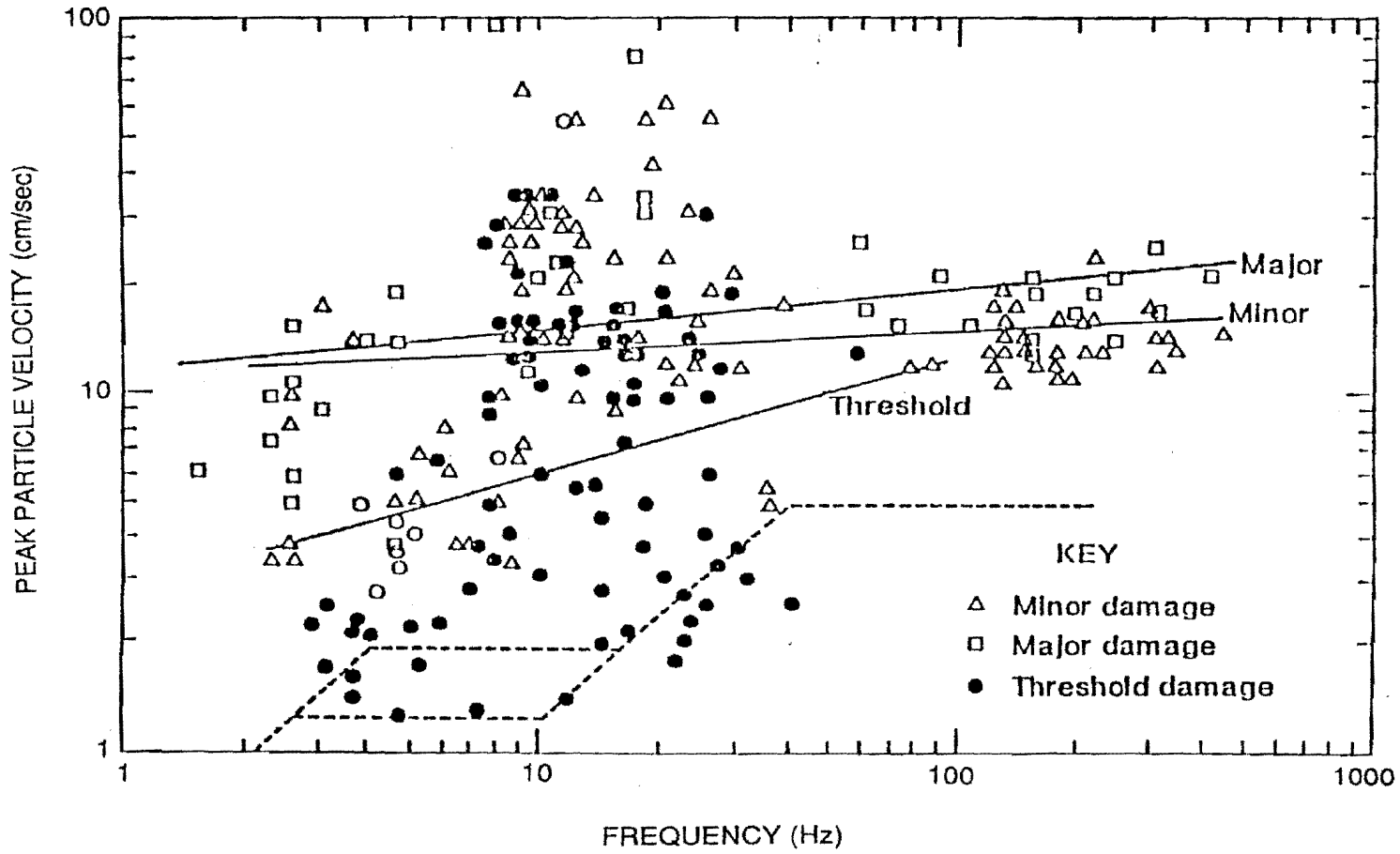

Figure 18. Envelope of safe peak-particle velocities from Bureau of Mines RI8507 (Siskind and others, 1980). From Part VI, Figure 30.

3.5.4 USGS Assessment of Earthquake Damage Potential

Historically, Evansville has been shaken by earthquake ground vibration and building damage has occurred. Accordingly, possible historical earthquake building damage must be considered in reviewing the known building damage in the study area. The objectives of this part of the USGS investigation were to: (1) evaluate the historical earthquake record at Evansville; (2) estimate historical earthquake shaking at Evansville based on the historical record; and (3) estimate the future earthquake shaking potential at Evansville assuming possible large shocks in the Mississippi Valley and on the basis of a probabilistic model of earthquake occurrence in the central United States.

The two most relevant historical earthquakes for this study that have affected Evansville are the 1968 and 1987 earthquakes which caused Modified Mercalli Intensity (MMI) of VI at Evansville. In particular, the 1968 shock was reported to have caused damage to numerous buildings. An alternate interpretation of the 1968 damage at Evansville might be that damage approaching the intensity VII level occurred. The 1987 earthquake resulted in the cracking of "chimneys, sidewalks, and streets." USGS hypothesized that the cracking of streets and sidewalks may be indicative of liquefaction or differential compaction.

While there are no records of damage in the study area, it is entirely possible that damage occurred. In fact, some damage was reported by home owners in the remote area during the USGS structural engineering inspections. Several homeowners in the study area reported feeling earthquakes.

USGS determined the historical distribution of intensity of shaking at Evansville based on the record of past earthquake occurrences. This distribution of shaking was obtained by using the locations of historical earthquakes and either attenuating the ground vibration intensity from the epicenter to Evansville using isoseismal maps or using actual reports of damage in Evansville. A 1811-1812 series of shocks in the New Madrid region of Missouri were predicted by this analysis to result in intensities of VII and VIII in Evansville. These intensities were projected since their actual occurrence at Evansville are unknown. This does not mean, however, that intensities of this degree or greater will not be experienced in the future. The important historical ground shaking at Evansville in terms of the present study are the 1968 and 1987 earthquakes (MMI of VI at Evansville) and, to a lesser extent, the earthquake of 1990 (MMI of V at Evansville).

The 1987 earthquake caused intensity VI damage in Evansville and triggered instruments at a number of coal mine monitoring stations, including four in the Daylight area. The peak amplitudes from these instruments ranged from 0.13 to 0.44 ips for the horizontal components and 0.04 to 0.09 ips for the vertical components. The subsurface material was not identified for the recording stations, so any effect of site response is not known. There were no instrument recordings in Evansville where the damage was documented. However, USGS reasoned that the PPVs would have been smaller than in Daylight, assuming similar site conditions, since Evansville is more distant

from the epicenter. Therefore, damage could possibly occur in the study area from the range of amplitudes recorded in Daylight.

3.5.5 WES Structure Response Modeling

The Structures Laboratory of WES developed linear-elastic, multi-degree-of-freedom, FE models of one- and two-story structures in the study area (Figure 19). The models were subjected to simulated ground vibrations based on the time-history characteristics of field recorded vibrations. The vibrations were scaled in amplitude to represent a worst-case scenario based on historical data (see Section 3.1). Stress distributions resulting from the cyclic vibrations of the structures were documented. The WES investigators estimated the maximum stress occurring during the structure response and compared it to the strengths of the building materials. If the stress exceeded the material strength, material failure was assumed to occur.

Field Tests

The development of a FE model necessitated data collection at sampled homes in the study area. Forced vibration tests were conducted to determine dynamic response characteristics such as natural frequencies, mode shapes, and damping. Also, structure response was monitored along with free-field ground vibration and airblast during blast events. Other loading conditions used in the tests included wind, flying aircraft, and road traffic. The field tests were used to develop, refine, and validate the FE model.

The criteria for selecting the houses for field testing were based on accessibility, the complexity of the structure, and the degree of damage. In order to limit the number of variables and, thus, the chances for error in the model, a simple, rectangular plan was desirable. It was also important that the buildings did not have severe damages. Large cracks or displacements can significantly alter the dynamic characteristics of a structural system.

The WES investigators collected data during two time periods. Monitoring and a limited amount of forced vibration testing on a two-story house in McCutchanville (house 316); and monitoring and modal tests at a one-story house in Daylight (house 103) took place from December 1 to 12, 1991. Additional modal tests and the use of remote instrumentation for long-term monitoring at the one-story house occurred from March 12 to April 15, 1992. A full modal test was not possible on the two-story house due to limitations on the right of entry.

Modal testing is the process of measuring the frequency response functions (FRF) from the input-output relationship of a physical system (such as the two houses sampled for this investigation). The FRF is computed by dividing the Fourier transform of the output by the Fourier transform of the input. Modal analysis extracts the dynamic characteristics of the system from the FRFs, including the parameters defining the modes of vibration. The vibrational modes represent the system's movement at corresponding natural frequencies (the frequencies at which the system maximally amplifies input

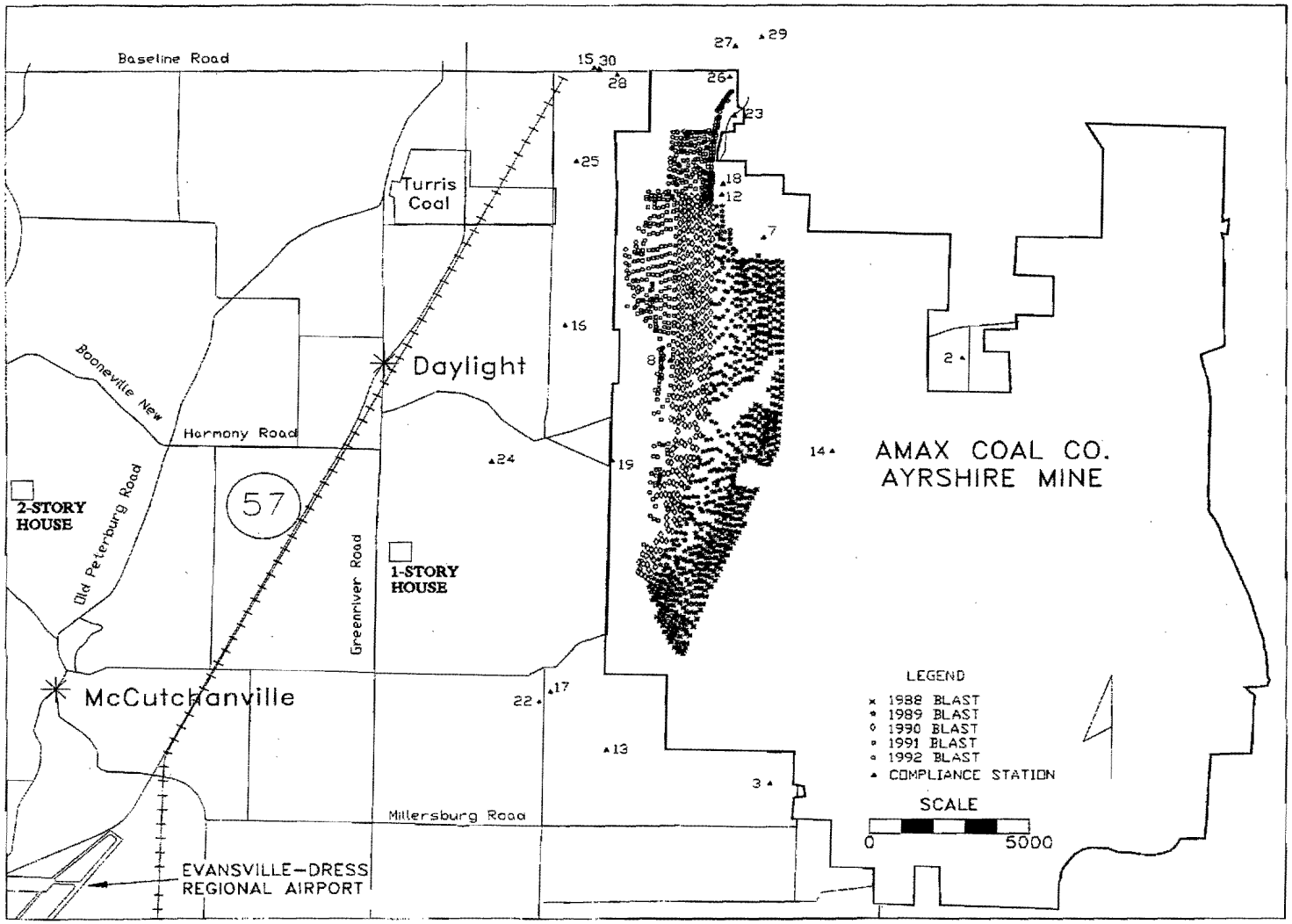

Figure 19. Study area, mine blast locations (1988-1992), and compliance monitoring stations. From Part VII, Figure 1.2.

vibrations). The first mode is associated with the lowest or "fundamental" natural frequency.

The procedure for modal testing involves the use of forced vibrations. The forced vibration inputs used in this investigation were an electrodynamic inertial mass exciter or "shaker" and an instrumented hammer. This instrumentation enabled the investigators to subject the structure to a broad frequency band of excitations and, during the modal test, to identify several vibrational modes. While the excitation was recorded during forced vibration tests, it was necessary to simultaneously record the response of the house at several strategic locations. Seismic accelerometers with a useful frequency range covering 0.3 to 100 Hz were used for the output recordings.

Structure response data from blasts and other "ambient" sources were used to check the validity of the modal test results. Recordings from mine blasts were also essential for determining structure response to free-field ground vibrations and airblasts. The accelerometers were primarily oriented to monitor the horizontal response vibrations during the ambient and forced vibrations. For a few tests in the one-story house, vertical response was measured at two locations.

Peak amplitudes of measured structure response at the one-story house were very low and ranged from 0.005 to 0.05 ips. The PPV for the only blast recorded on the two-story house was 0.01 ips. Peak airblast pressures at the one-story house were less than 1×10^{-3} pounds per square inch (psi). Pressure measurements from wind were in the same order of magnitude. Structural amplification factors were calculated by averaging the ratios of the vibration responses measured at various locations in the house to the free-field ground vibrations. The amplification factors ranged from 2 to 6. From the forced-vibration data the fundamental natural frequency of the one-story house was estimated to equal 7.5 Hz.

Analysis of Previous Field Tests

Data on the relationship between peak ground vibrations and material strains had been documented in 1984 by USBM. The WES investigators compared this data to critical tensile strains (CTS) for the same materials. The materials evaluated include: wallboard; wallboard tape joint; plaster; and brick and block mortar joints. The CTS levels for the first three materials were obtained from USBM tests documented in 1980 and 1984. A range of CTS values for mortar joints were computed by WES, using a design standard for (1) allowable flexural tension stresses and (2) the modulus of elasticity for 1,500- and 2,000-psi masonry units. This is supplemented with tested failure strain levels reported in the literature for 4-in. brick and 8-in. hollow block.

The comparisons for wallboard, wallboard tape joint, and plaster indicate that maximum strain responses are less than all CTS levels for ground vibrations below 2.0 ips (Figure 20). Cracking in these materials is not expected for potential worst-case ground vibrations of 0.17 and 0.39 ips. The maximum strains in mortar joints easily exceed both documented failure strain levels (for the brick and block) and the WES-computed CTS levels under

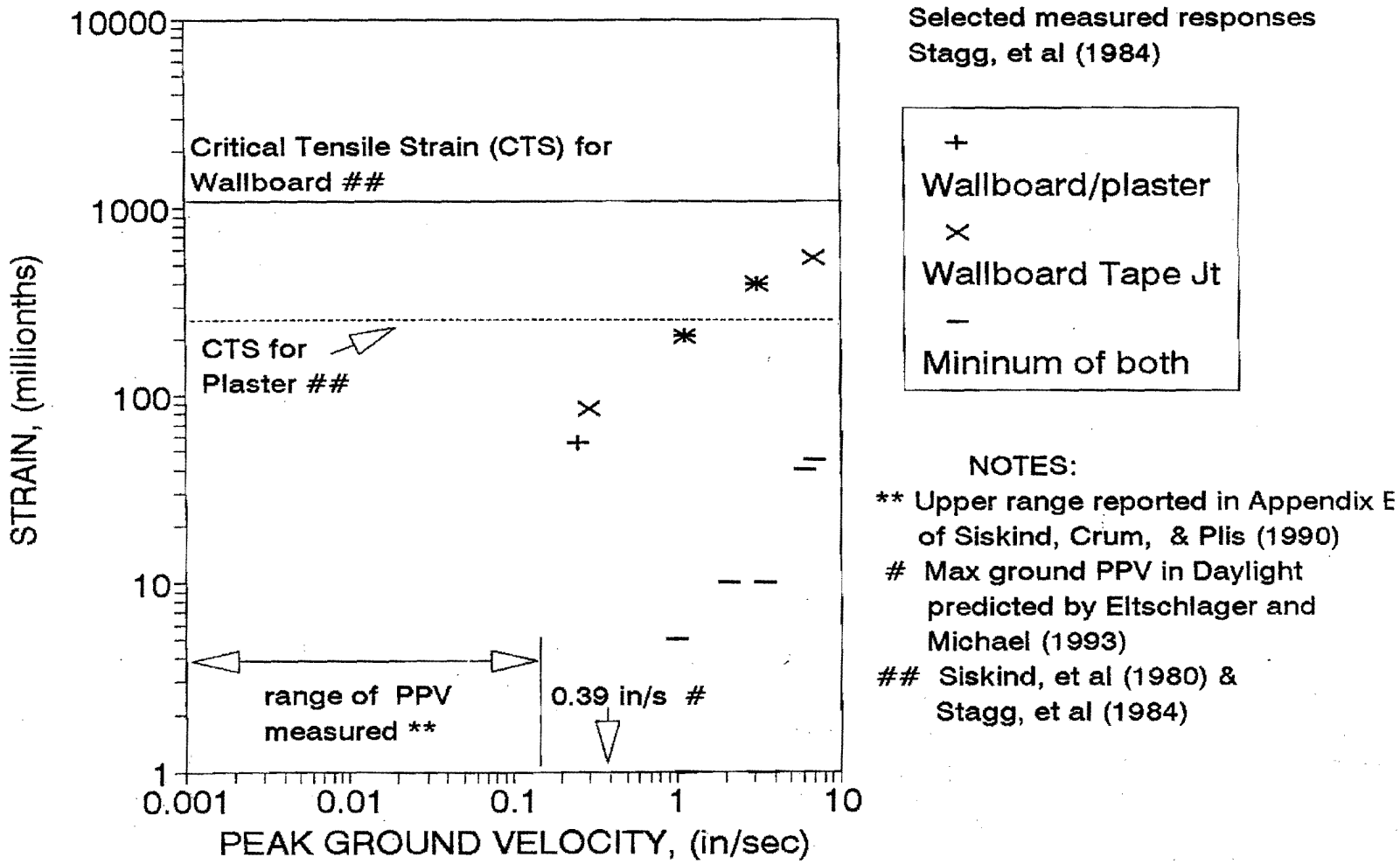

Figure 20. Strain versus peak ground velocity for wallboard, plaster, and wallboard tape joint. From Part VII, Figure 2.21.

the worst-case ground vibrations (Figure 21). However, reservations concerning the comparability of the three sets of data were expressed by the USBM investigators and discussed with WES subsequent to this analysis. For instance, some of the USBM strain data were obtained from measurements across preexisting cracks in the mortar. Cracks are expected to increase displacement within a material in response to vibrations. Other problems identified pertain to: (1) different methods of measuring strains between the failure strain data and the USBM strain data; and (2) the acceptability of the low CTS values computed and utilized by WES. Because of the above concerns, conclusions related to material failures from measured or estimated ground vibrations for the study area are also dependent on the following FE analysis.

Finite-Element Analysis

In the FE modeling method, equations of vibration describing the response of a structure are solved numerically. The structure is reduced into a simple assemblage of nodes where degrees of freedom are specified. Each degree of freedom has an associated mass, damping, and stiffness. The nodes are connected with elements, called "finite elements." Physical problems to be modeled by finite elements are defined by specifying the: (1) geometric shapes; (2) material properties; (3) boundary conditions; and (4) applied loads. The mass, damping, and stiffness assigned to each element are dependent on the material properties and structural dimensions of the structure under study. The nodes in a three-dimensional structural model can have up to six degrees of freedom. The degrees of freedom represent displacements in the coordinate x, y, and z directions and rotations about each of these coordinate axes.

The model of the one-story house was calibrated by comparing the first mode shape and frequency of the FE results with the corresponding mode shape and frequency determined from the modal test. The two-story house was then constructed using the elements developed for the one-story.

The main structural elements of the one- and two-story house were made up of composite parts. A typical wall section consisted of a two- by four-in. stud with plywood attached to the exterior face and gypsum board attached to the interior face. The studs were placed on 16-inch centers. This composite element was approximated with a uniform shell element. Thickness of the shell element is computed to give the same moment of inertia and flexural rigidity as the composite element. The mass of the brick veneer of the one-story house was added to the horizontal degrees of freedom for the nodes in the exterior walls. The WES investigators assumed that the vertical inertial vibration of the brick veneer acted independently of the wall, i.e., the mass of the brick transmitted the vertical inertial force directly to its base support.

The FE model reproduced overall structural motions such as torsion (twisting in place), side sway in the strong and weak axes, and higher order vibration modes. A ground vibration recorded at the one-story house was selected as

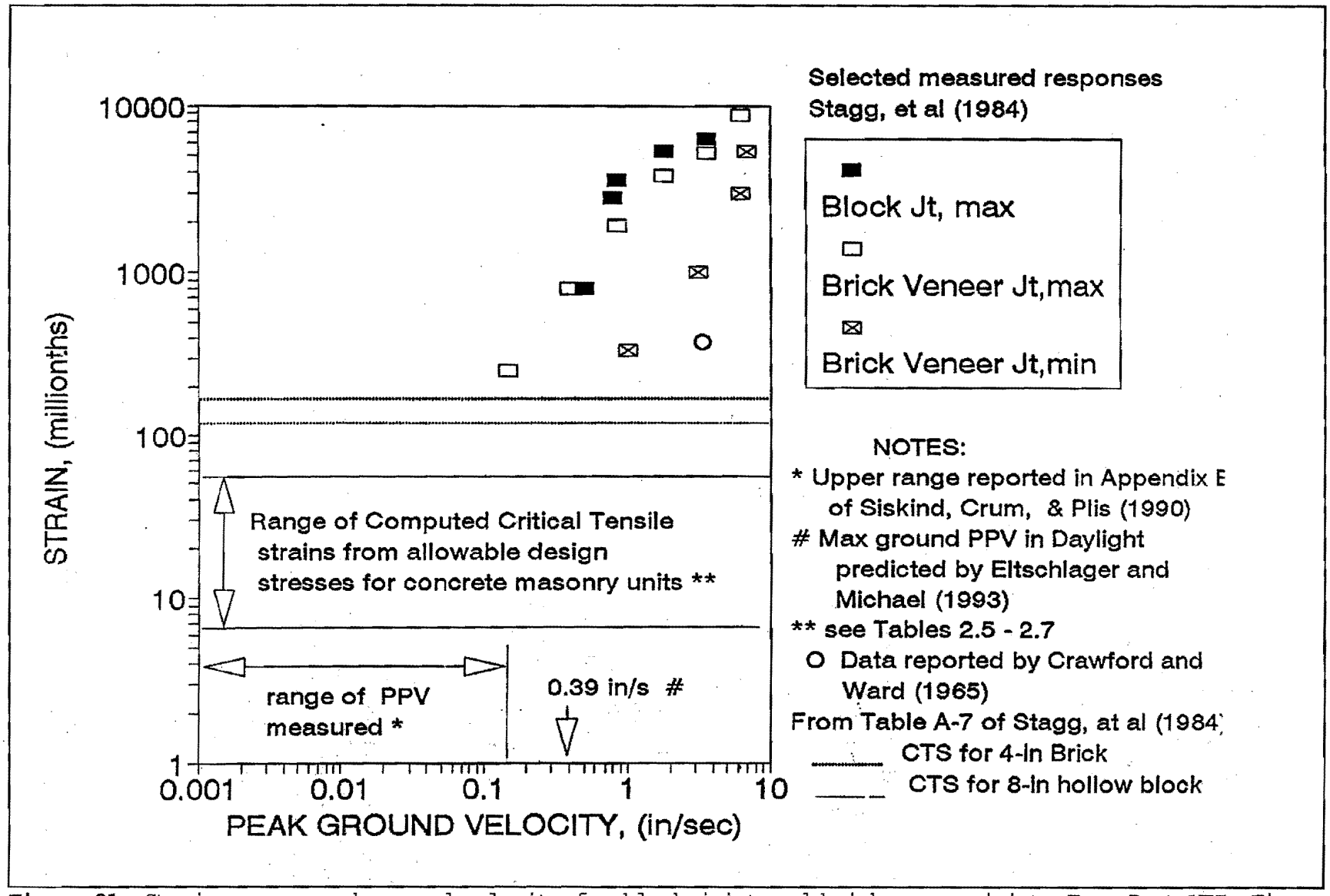

Figure 21. Strain versus peak ground velocity for block joint and brick veneer joint. From Part VII, Figure 2.22.

the model input. The vibration time history was scaled to produce a peak amplitude of 0.39 ips.[1]

One-story house: The model resulted in a fundamental natural frequency for the one-story house of 10.2 Hz, which compared favorably with the field measured value of 7.5 Hz. The increase in the computed natural frequency over the measured frequency results from an overestimation of boundary stiffness between the building foundation and the ground.

The dynamic amplification factor (DAF), a ratio of output structural vibration to the input base motion, was computed to be 4.88. This corresponded well with the DAF range of 2.0 to 6.0 obtained from the field tests. The FE model indicated a maximum tensile stress level of 55 psi from blast-related ground vibrations. This is well below USBM tensile strength test results of 170 to 250 psi for wallboard. Assuming linear-elastic behavior, threshold cracking in the walls of the modeled structure should occur at ground vibration velocities equal to or exceeding approximately 1.0 ips (Figure 22). Based upon this model, no damage is expected to the simulated one-story structure from measured or estimated study-area vibrations.

Two-story house: Since a modal test was not performed on the two-story house, the same structural elements developed for the one-story house were used to construct the two-story FE model. The selected ground vibration time history for input base vibration was the same as that applied to the one-story house model.

The first mode frequency was computed to be 23.7 Hz. Generally the fundamental natural frequency for houses is a side-sway vibration about the long axis with a frequency of 7-12 Hz. A garage extension to the two-story model added considerable stiffness to the structure, resulting in the house responding in a more complex first mode with a higher frequency.

The FE model of the two-story residential structure indicated a maximum stress level of about 45 psi. This is well below USBM tensile strength test results of 170 to 250 psi for wallboard. Based upon this model, no damage to the simulated two-story structure is expected.

Brick veneer: The dynamic response of the brick veneer was modeled separately from the rest of the building. The field test of the one-story structure demonstrated that the brick veneer can respond independently of the structure since it is only loosely coupled to the interior wall. The modeled wall was based on 4 rows of thick shell elements and was 62 feet long and 8 feet high. All edges except the base were unsupported. To account for out-of-plane response limitations from the main building, the model was tested only for in-plane vibrations. Damping values of zero, two, and five percent were included in the analysis.

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[1]The worst-case scenario for Daylight is identified as 0.38 ips in Section 3.1. The value presented here was provided to WES prior to the completion of the Part III investigation.

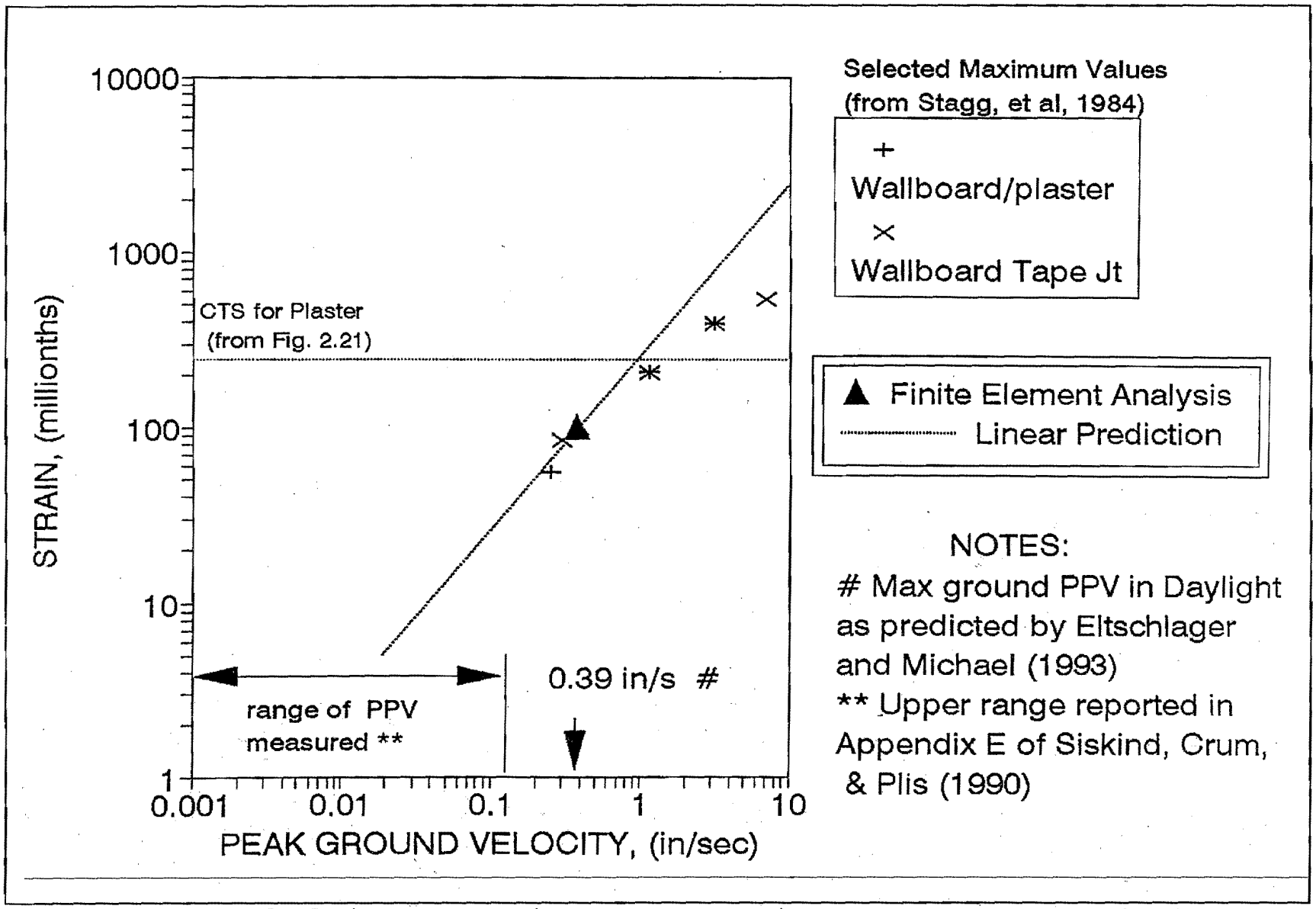

Figure 22. Peak ground velocity versus strain. From Part VII, Figure 4.22.

The model output demonstrated a concentration of stress near the base of the wall. Maximum tensile stresses of 30, 10.2, and 6.6 psi resulted from a ground vibration of 0.39 ips, for 0, 2, and 5 percent damping, respectively. The predicted response of the undamped wall falls within the tensile strength range of 12.8 to 60 psi for brick and block mortar joints (based on the CTS levels discussed above).

The response for two and five percent damping fall below the lower bound of the tensile strength range. Assuming (1) linear-elastic behavior and (2) the minimum CTS value of mortar, cracking can occur from ground vibration velocities that equal or exceed 0.13, 0.4, and 0.7 ips for 0, 2, and 5 percent damping, respectively (Figure 23).

3.5.6 WES Evaluation of Horizontal Basement Wall Cracks

In a March 11, 1991 report on the February 1991 interagency site review, the investigator from the Geotechnical Laboratory of WES presented a table summarizing his findings. From 15 homes inspected during the site review, 9 homes were listed as having damages indicative of horizontal differential structural movement. These damages included long horizontal cracks at or above ground level; and systems of cracks or displacements suggesting horizontal vibration in a preferred N-S direction. He cited these types of damage as "...clearly associated with horizontal loading or horizontal movement...", for which "...this author can find no source for such movement or loading other than the blasting." His subsequent 1993 final report of investigation includes an addendum of observations and remarks concerning this earlier finding.

The addendum proposes additional potential causes for the kinds of damages depicted above. These are described as follows:

1) Earthquakes: The 1987 earthquake producing PPVs of 0.20-0.44 ips and the 1990 event with a MMI of V are cited.

2) Thermal Expansion and Contraction: During a October 15, 1991 visit to the study and remote areas, the investigator observed the construction of two concrete block basements in progress. The block was unfilled and unreinforced. Short dowels had been grouted into the top course of blocks at intervals of about 10 feet which were to be connected to the sill of the wood frame. Since these dowels extended only into the top course, he reasoned that any horizontal expansion or contraction of the superstructure due to temperature changes would be transmitted to the top course of block. If large enough, this could be a cause of the continuous horizontal cracks observed in the mortar just below the top course of block in many of the structures.

3.5.7 Discussion

USBM Structure Response and Crack Monitoring

The USBM monitoring of structure response to blast-induced vibrations and crack changes did not produce evidence of surface mine blasts causing damages

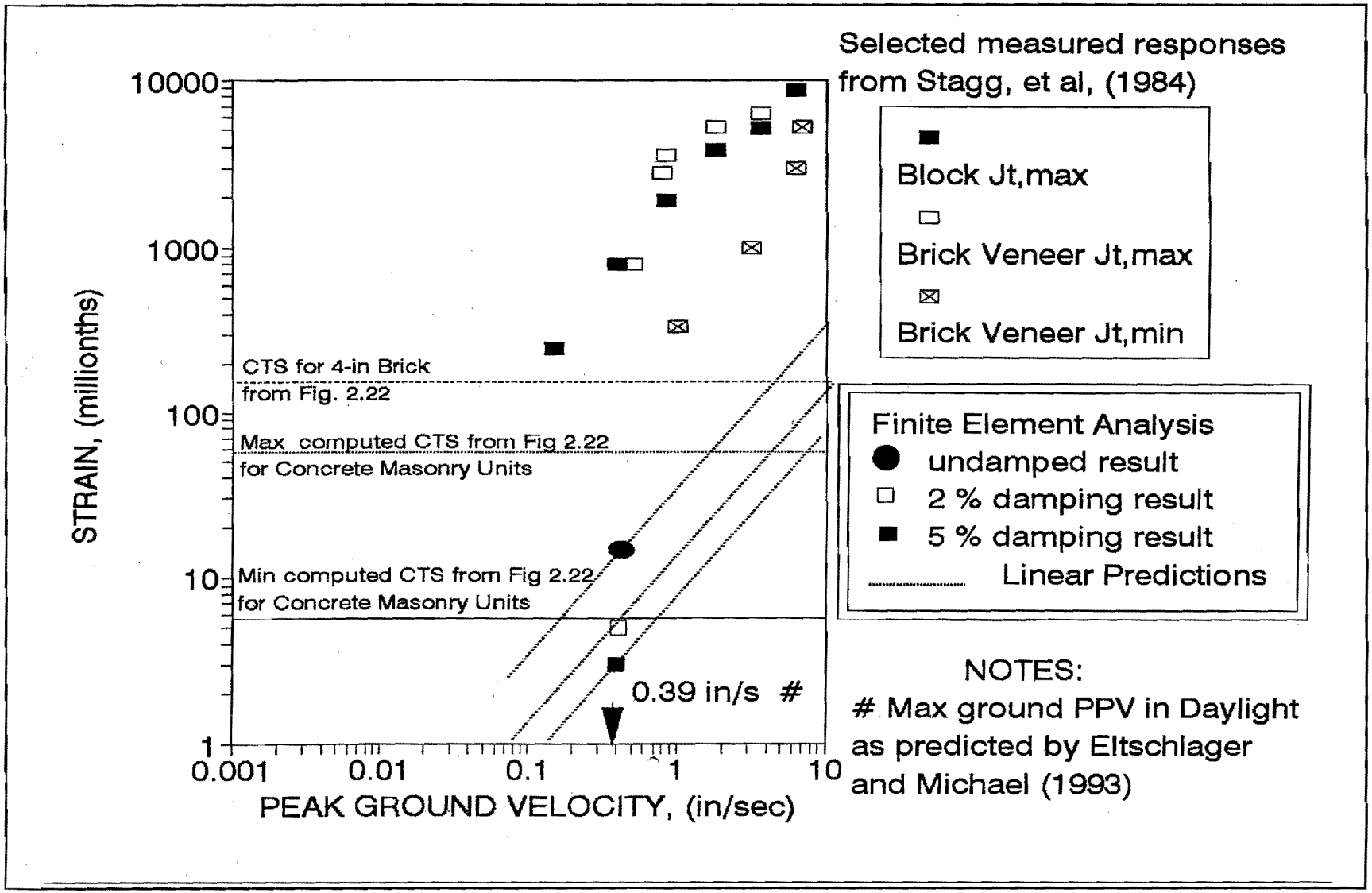

Figure 23. Selected maximum responses of block joints and selected maximum and minimum responses for brick veneer joints measured by Stagg (Stagg, et al., 1984) and fitted linear predictions. From Part VII, Figure 4.23.

to houses in the study area. Structure-response levels were well below the established damage thresholds for ground vibrations. Only one permanent crack change, a crack extension at house 107, was noted after a blast. Since this blast did not produce the strongest monitored structure response, one could argue that a cause other than blasting should have been necessary for this change to occur. However, it cannot be concluded that blasting did not have any influence on the extension of this crack.

The USBM investigation was dependent on concurrent blasting. To conclude blasting probably never caused any damages requires the assumption that the ground vibrations and airblasts during the USBM monitoring represented the strongest blast-related vibratory events which occurred. AMAX records clearly show that Ayrshire blast vibrations were monitored at one site in Daylight in the range of 0.15 - 0.20 ips. The maximum vibration monitored in Daylight by USBM was 0.1 ips.

WES FE Model Analysis

In an attempt to evaluate impacts from the theoretical worst-case ground vibrations, the WES Structures Laboratory applied the WES and OSM (Section 3.1) analyses of blasting data to the structural FE models. Subjecting the simulated one- and two-story buildings to the theoretical worst-case vibration resulted in maximum tensile stresses lower than documented tensile strengths of plaster and wallboard. If we accept the validity of the model, the evidence suggests that the projected worst-case ground vibrations should not have damaged plaster or wallboard in structures in the study area. This conclusion is limited to buildings not under significant stresses from other sources. The model assumes a simple, statically stable foundation and superstructure.

In consideration of the brick veneer model analysis, cracking in brick veneer mortar in the study area in response to blasting is a theoretical possibility only when damping is assumed to be zero percent. The analysis is conservative because of several factors:

(1) The sides of the modeled wall are not constrained;

(2) The WES investigators expect some margin of safety in the standards used to compute the range of mortar CTS;

(3) The analysis takes into consideration the effects of the lowest computed CTS for mortar and the worst-case ground vibration; and

(4) This is a stress vs. strength, or strain vs CTS, analysis. Cracking that may occur near the point where stress begins to exceed strength (or where strain begins to exceed CTS) may not be visible to the naked eye.

(5) The model could not be verified by comparison with field tests. The motions measured in the brick veneer of the one-story house were perpendicular to the plane of the wall (not in-plane).

This conservatism is balanced to some degree by the total absence of out-of-plane vibration in the model and the unknown effects of mortar strength deterioration in the field with time. As would be expected, the theoretical possibility of cracking from blasting would be greater in Daylight and other locations relatively close to the mine. However, it is our opinion that the results of brick veneer model do not constitute evidence that blasting was the primary cause of damage to brick veneer around buildings in the study area. This is due to (1) the absence of empirical data that would support a cause-effect relationship and (2) that the realistic damping values of two and five percent result in strains below the CTS.

USGS Site and Structure-Response Monitoring

Building natural frequencies measured by USGS fell within the range of those of previous studies. Other USGS analyses demonstrated the variation in which different houses responded to ground vibration events. Where both parameters were measured (2 houses), building and site natural frequencies were generally in close agreement, but the degree of correspondence ranged between a difference of 0.2 and 5.4 Hz. Two complainant homes (structures 107 and 301) measured for response relative to ground vibration had similar maximum horizontal amplification factors, but one (107) had similar horizontal amplifications across both axes and an unusually large vertical response. This response behavior may be related to the similar horizontal dimensions of the structure and the effect of a second story addition, respectively. These same two homes have measurably greater site-response differences than the non-complainant companion houses. It is well established that structure response to local surface mine blasts have been discernable to residents of the study area. The variance in structure response from one building to the next could have contributed to a variation in the citizens' experience of the blasts. This could also have caused differences in the probability of damage.

USGS Inspections

From its structural inspection of the buildings, USGS cites structure 115 as having damages most closely resembling vibration effects. The structure is located very near to some of the blast locations and there is a close ground to structure natural frequency match. USGS further makes the statement that the structure's interior wall cracking "...was more extensive than would normally be expected in a house subjected to normal use." This finding is based on the author's professional judgement. The author also acknowledges several items of circumstantial evidence for vibration effects with respect to structure 107. These are: (1) the homeowner's meticulous and systematic highlighting of cracks and crack extensions; and identification of allegedly associated blast dates and times; (2) the crack extension after a blast documented during the USBM Part IV investigation; (3) the amplification effects mentioned above; and (4) the greater site response relative to the companion structure. It is important to note that most of the damages occur in the basement, where amplification of ground vibration is minimal.

Other Forces and Prestress

The results of the WES FE model analyses do not support the proposition that damages to plaster, wallboard, or mortar joints in any study-area residential building are caused by mine blasting. USGS and WES identified that the potential existence of static stresses within a structure concurrent with a blasting event is another factor that can influence damage probability. The USGS house inspections revealed significant variations in damage level among homes in close proximity to each other. These differences indicate that there are forces other than blasting affecting buildings (further discussion on other damage causing processes, including those influencing structures 107 and 115, is presented in Section 3.9). Also, natural strains generated by daily environmental changes, such temperature and humidity changes, have been documented by Siskind et al. as corresponding to ground vibrations of 1.2 to 3.0 ips (1984, citation in Part III). Stresses from a structure's response to a vibration event (or an airblast) may either add or subtract to the magnitude of existing stresses in different parts of the building. Theoretically, where they are additive, the resulting total stress may exceed material strength and generate cracking. The actual likelihood of such a stress combination actually occurring is unknown, in part because the actual loading history of the buildings are unknown. In any event, the magnitude of the difference between the low levels of vibration-induced stresses and actual material strengths for wallboard and plaster walls would require structural problems that are significant, if not severe. This is true even for the Daylight worst-case scenario. However, the amount of prestress needed in brick veneer for this theoretical additive effect to result in cracking may not be as great.

Earthquakes

Earthquakes potentially capable of causing damages have taken place in the study area. The problems of structure response and stress addition are also applicable to natural tremors.

The 1987 earthquake recordings in Daylight and the documented damages in Evansville further from the epicenter suggest that damages in the study area could occur during horizontal PPV measurements of 0.13-0.44 ips. The possible association between a peak amplitude measurement in the range measured and potential damage is a significant observation. At this point, however, we do not know the precise location of the damages in Evansville relative to zones of hypothetical maximum MMIs (see Figures 5-7, Part VI). It is possible, though uncertain, that damages occurred in Evansville and not in Daylight because of significantly different geological conditions. Another unknown is the site response characteristics of instrument location.

3.5.8 Findings

The February 1991 field review lead to the hypothesis of a lateral force causing damages. Initially, this lateral force was attributed to blast-induced vibrations. Subsequent work identified other potential sources including earthquake-induced vibrations, thermal expansion and contraction of building materials, and soils-related phenomena.

There is no evidence that mine blasting caused foundation-related problems or any other problems that impair structural integrity in the study area.

Damages to wallboard and plaster walls and other interior parts of structures with normal foundation or superstructure conditions should not have resulted from mine blasting. There may be a comparatively greater probability for blasting to have reached levels that caused threshold damage to brick veneer mortar joints. However, there is still insufficient evidence to suggest that blast-induced threshold damage has actually occurred. Theoretically, blast-related vibrations could have contributed to threshold distress for structures already stressed from other non-vibratory processes--or with unusual response characteristics. The range of measured and predicted blast-induced ground vibrations and results of the WES FE model analysis suggest that this combining effect, if it has actually taken place, should not have been a common occurrence in wallboard and plaster. It may have taken place more frequently in brick veneer. However, any link between mine blasting and damages are neither confirmed nor negated by empirical evidence observed by any of the investigators on the interagency team.

3.6 Can observed damage be ascribed to fatigue induced by the repetitive exposure to ground vibrations and/or airblasts?

The subject of the USBM 1989-90 investigation was limited to immediate effects of blasting on structures. The cumulative effects of ground vibrations and airblasts on building materials had been studied by USBM near the Ayrshire Mine in 1984. This research involved a thorough analysis of the response of one house to long-term cyclic loading. The house was specifically built for the project and was typical of many single homes in the area.

In consideration of the diversified population of structures and site conditions in the study area, the WES Structures Laboratory further evaluated the potential of fatigue on construction materials (Part VII). This was carried out mostly under subcontract with Dr. Sam Kiger of the Department of Civil Engineering at West Virginia University.

3.6.1 West Virginia University Assessment

Kiger cited an observation in the report on the USBM 1989-90 investigation (Part IV) that 5 to 10 ips ground vibrations is the minimum amplitude required to crack concrete walls, driveways, and foundations and to cause major superstructure cracks. He compared this to the maximum PPVs that could have occurred, as estimated in the WES Geotechnical Laboratory March 1991 field inspection report. The maximum predicted values of 0.5 and 0.2 ips for Daylight and McCutchanville, respectively, are at least an order of magnitude lower than the minimum to cause major damage. He judged the difference too great for fatigue to occur. The effect of compression on basement walls should also have been a factor preventing fatigue damage.

The probability of repeated blast vibrations causing minor or threshold damages is also discounted for the following reasons:

1) Subjecting wallboard to 0.5 ips PPV mechanical ground vibrations in the USBM 1984 fatigue study resulted in joint cracking after 56,000 cycles. This was equivalent to two blasts per day, 5 cycles per blast at 0.5 ips PPV for 28 years. According to a May 10, 1991, memorandum from the Director of IDNR, the PPV recorded in the study area by IDNR in 1987 was 0.03 ips and was still 0.03 in 1991. The average peak amplitudes seem to have been at least an order of magnitude less than the 0.5 ips.

2) Another study by Dr. Charles Dowding of Northwestern University demonstrated that 88 percent of gypsum panels tested did not crack before 100,000 cycles at 50 percent of their static strength. Tests on plasterboard had similar results. The USBM fatigue study found bending strengths of wallboard to range between 900 and 4700 microin/in. The maximum strain from 0.5 ips PPV was shown to be less than 100 microin/in, which is at least an order of magnitude less the strain needed to cause fatigue damage.

3.6.2 WES Assessment

The WES Structures Laboratory attempted to make a direct quantitative comparison between the mine blast ground vibration environment in the study area and required conditions (documented in the literature) for the occurrence of material fatigue. The investigators estimated a total of 156,000 vibration cycles over a 10-year period from 1983. This was based on: a predominant house-response frequency of 10 Hz; 50 cycles (5 seconds of structure response per blast); and 6 blasts per week. However, a typical blast response abruptly decays, with its largest acceleration occurring only during the first two seconds. The investigators estimated 62,400 cycles of peak response in the study area. This was applied to a documented relationship for gypsum panels between (1) number of cycles needed for material failure and (2) the magnitude of strain generated as a percent of static strength (Figure 24). It was determined that the peak acceleration of records collected by WES must result in strains of almost 70 percent of the static failure strain, i.e. a strain level of 182 millionths, for fatigue to occur. This strain level corresponds to a stress of 104 psi, which is almost twice as great as the maximum stress of 55 psi (based on an estimated worst-case vibration of 0.39 ips for Daylight) determined by the FE model analysis of the one-story house.

The WES fatigue analysis also includes a review of some relevant findings in the 1984 USBM fatigue study:

1) It would require at least five years to produce the necessary number of cycles to cause cracking in new wallboard at continuous sinusoidal shaking a 10 Hz of 0.5 ips peak response.

2) The results of cyclic (cycled at 2 Hz) load tests on 1/2-in.-thick wallboard show that 475,000 cycles were required to crack the wallboard at 0.5 ips peak response.

3) The occurrence of a 20 percent prestrain had little effect on the fatigue level of failure for gypsum wallboard.

3.6.3 Discussion

WES did not conduct a fatigue analysis for brick veneer mortar joints. The relationship between percent of static strength needed for fatigue in this material and the number of vibration cycles is not documented. However, we have made a conservative estimate of percent static strength using (1) the lowest CTS of concrete masonry computed by WES (6.2 microin/in), (2) the strongest ground vibration amplitude recorded in Daylight (0.2 ips), (3) 2 percent damping and (4) the linear-elastic relationship developed by WES (Figure 23).

The vibration amplitude 0.2 ips results in a strain of 2.3 microin/in, which is 37 percent of the CTS. This value is low compared to the 70 percent requirement for wallboard/plaster fatigue from 62,400 vibration cycles; and does not constitute evidence for the occurrence of fatigue (without the influence of prestress) in brick veneer mortar in the study area.

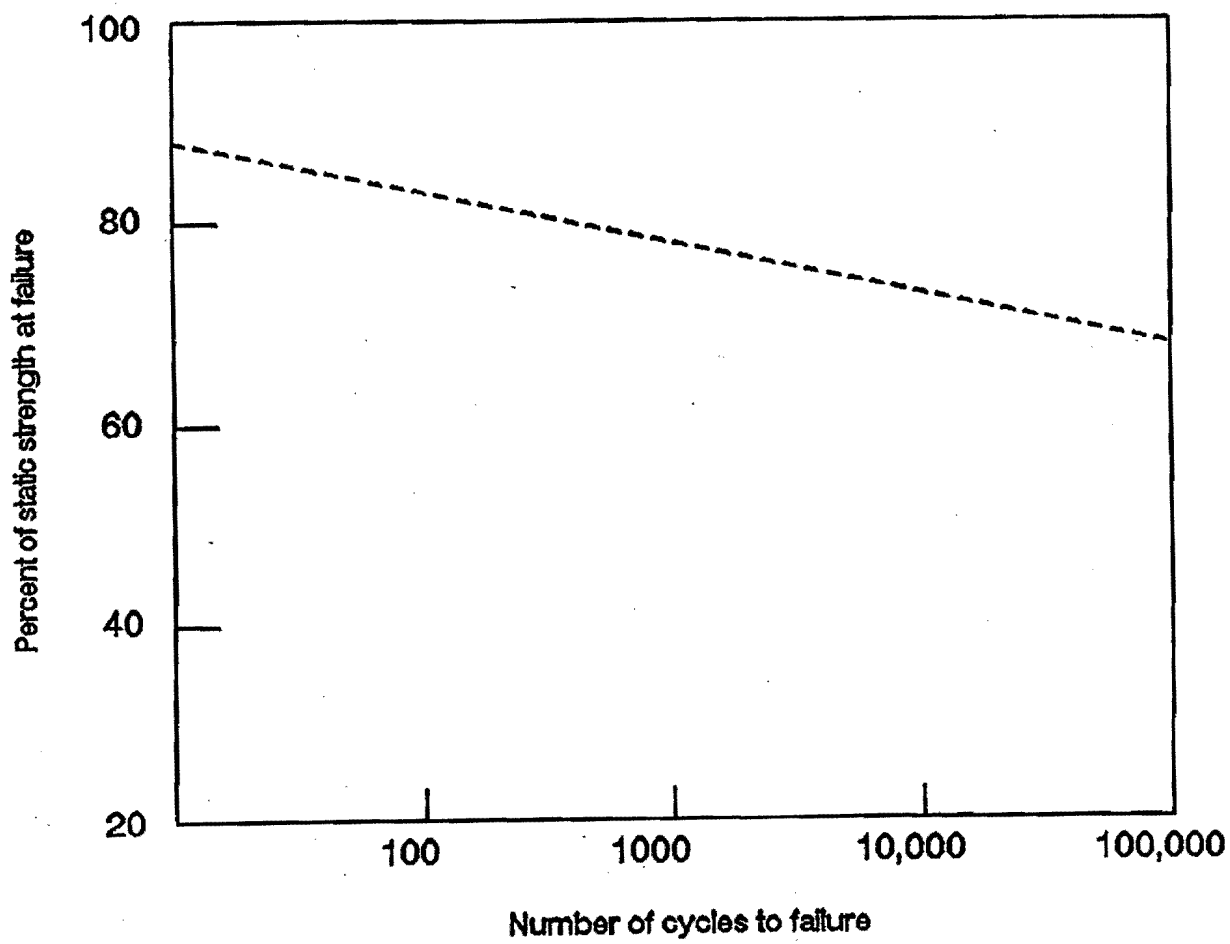

Figure 24. Fatigue behavior of gypsum panels (after Leigh, 1974). From Part VII, Figure 5.51.

3.6.4 Findings

The available evidence suggests that fatigue failure of building materials from the cumulative effects of mine blasting was not responsible for damages in the study area. Furthermore, for prestrain to significantly modify the requirements of fatigue failure, its level must at least exceed 20 percent of static failure strain. This agrees with statements made on the effects of prestrain on wallboard and plaster in Section 3.5.

There is no evidence for the occurrence of fatigue in brick veneer mortar in the study area.

3.7 Have synergistic effects occurred between ground vibrations, soil movement and structure distress?

The Geotechnical Laboratory of WES examined the possibility that repetitive, low level vibrations from blasting have affected structures by dynamically loading the foundation soils (Part VII). This investigation had the following two objectives:

(1) **To determine if undisturbed, unsaturated soil samples from the study area could collapse under many cycles of low amplitude vibration;**

(2) **To determine if soil samples form the study area would experience an increase in pore pressure or loss of shear strength as a result of cyclic loading.**

The purpose of the first objective was to evaluate the potential for foundation soils to consolidate or collapse from blasting. The second objective was conducted to assess the potential for slopes around structures to destabilize under the influence of blasting.

In May 1992, 26 undisturbed soil samples and 27 jar samples were collected using a fixed piston sampler in the study area and the remote area (see Sections 3.5.3 and 3.8) for classification and cyclic testing. The soil samples were classified in accordance with the Unified Soil Classification system. A preliminary, back-pressure saturated, consolidated, undrained triaxial compression test was first conducted to determine whether the soil tended to expand or contract in shear and to gain an understanding of the time required for consolidation. The rest of the tests performed to achieve the above objectives are summarized below. References to laboratory testing standards are provided in Part VIII.

3.7.1 Cyclic Torsional Tests

Dynamic low-level cyclic torsional shear testing was conducted to evaluate the potential for collapse of the structure of unsaturated soils when under vibratory loads. Fourteen tests were conducted on 2.8-inch diameter by 5.6-inch high specimens in a Drnevich longitudinal-torsional free-fixed resonant column apparatus. Testing procedures consisted of a consolidation phase followed by a dynamic torsional shear phase for each specimen. The consolidation phase consisted of the application of an isotropic stress equivalent to the estimated in situ overburden stress for that specimen. After the specimen had equilibrated under the applied stress, the dynamic shear phase (with open drainage) was begun.

The shear phase consisted of the application of a cyclic torsional shear stress to the specimen to cause a desired amplitude of shear strain at the frequency of interest. Two amplitudes of shear strain (0.01 and 0.04 percent) were applied to each specimen in the drained condition. The cyclic frequency was 20 Hz, the lowest at which control of the apparatus could be maintained. It was on the upper end of the range of frequencies measured in the field rather than in the middle as would have been most desirable. Seventy-two thousand cycles at 0.01 percent shear strain were applied; then

the specimen's vertical deformation was monitored for two hours. An additional 72,000 cycles at 0.04 percent shear strain were applied and the specimen was monitored for another two hours. The axial deformation of each specimen was monitored throughout the dynamic shear phase. Each specimen was tested at its in situ water content.

A one-dimensional shear wave propagation model was used for selecting the shear strain levels used in these tests. This model was mathematically tractable and known to give results within the correct order of magnitude for three-dimensional explosion generated wave propagation strain estimates. It provides a relationship between horizontal PPV, shear wave velocity, and maximum shear strain. The maximum measured PPV near a complainant's home by USBM during limited monitoring periods was about 0.10 ips. Resonant column tests performed on two specimens resulted in shear wave velocities of 372-481 ft/s. Shear velocities measured in the field by USGS ranged from 380-780 ft/s.[2] Based on these data and the assumed model, a peak shear strain of 0.002 percent was estimated to have occurred in the field. To overcome any error in estimated strain associated with (1) the assumed model or (2) the possibility that a somewhat larger particle velocity occurred in blasts where there was no monitoring equipment at complainant residences, strain levels 5 and 20 times the predicted value were used.

The mine had been in operation since 1973 with approximately six significant shots per week, all with about ten significant cycles of vibrations (indicated in ground vibration records for some of the shots). This represents about 60,000 cycles of vibration. Again, to be conservative, 144,000 cycles were applied.

Under the sustained oscillating shear strain environment created in the torsional shear tests, the six inch high specimens changed in height by an amount ranging from 5 to 15 ten-thousandths of an inch. This was a vertical strain of 0.025 percent or less. After the shear phase was completed, most specimens rebounded to the original height measured prior to the test.

If 0.025 percent strain occurred uniformly over a 100-ft deep soil column, only 0.3 inch of surface displacement would result. Differential displacement between two surface points would be less. Conventional residential structures would not be damaged by these conditions. The investigators concluded that the torsional shear tests offered no evidence for a collapse mechanism or creep mechanism triggered by sustained low level vibration (below 0.5 ips).

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[2] The USGS final report (Part VI) and Section 3.2.2 of Part I state that there are no computed shear wave velocities below 150 m/s (492 ft/s). USGS provided WES the range of velocities mentioned above based on preliminary field results prior to completion of the USGS investigation.

3.7.2 Monotonic and Cyclic Triaxial Compression Tests

This group of tests was conducted to determine whether there was any loss of shear strength of near surface soils due to cyclic loading and if there was any potential for pore pressure generation by cyclic loading. Triaxial compression tests were conducted on four specimens at their natural water contents (Figure 25). Two of the specimens were consolidated and sheared monotonically (with drainage open). Following the consolidation phase on the other two specimens, 20 cycles (at 1 Hz) of dynamic deviator stress were applied (with drainage) using a stress controlled loading mode. When the cyclic loading was completed, each specimen was subjected to strain controlled monotonic loading with drainage open until failure (5 percent axial strain) occurred.

During dynamic loading, the extension and compression loads applied to the specimens were sufficiently large to ensure that a reversal of the major principal stresses occurred. Cyclic reversal of principal stresses has been shown to generate pore pressure in some soils. The zero to peak amplitudes of the cyclic deviator stress were approximately 2 psi, which is 4-7 percent of the deviator stresses at failure under static load. The resulting zero to peak cyclic axial strains were from 0.001 to 0.02 percent; and were, respectively, within the range of and significantly above the estimated 0.002 percent peak shear strain for blast vibrations in the study area.

The results of the consolidated drained triaxial compression tests on specimens tested at their natural water contents (unsaturated) indicated that the application of 20 cycles of dynamic axial loading did not affect the consolidated drained strength of the specimens significantly. Deviator stress data from the two tests were compared at 5 and 15 percent axial strain, as follows:

- The value of 5 percent strain corresponded to the axial strain at which the back pressure saturated triaxial compression test specimen failed. For the specimens from a depth of 6.1 feet, the deviator stress was 31 psi for the cyclicly loaded specimen and 30 psi for specimen subjected to only monotonic shear. For a depth of 13.5 feet, the deviator stress of both specimens was 50 psi.

- For the axial strain of 15 percent and specimens from a depth of 6.1 feet, the deviator stress was about 34 psi for the cyclicly loaded specimen and 28 psi for the specimen subjected only to monotonic shear. For a depth of 13.5 feet, the deviator stress was approximately 49 psi for the cyclicly loaded specimen and 55 psi for the specimen subjected only to monotonic shear.

Back pressure saturated, consolidated, undrained triaxial compression tests were conducted on two specimens. One of them was the preliminary test discussed above, in which the specimen was sheared monotonically. The other test included an undrained cyclic loading phase following consolidation (in which the specimen was isotropically consolidated to 5 psi) and before the undrained monotonic shear phase. The specimen was subjected to 20 cycles of

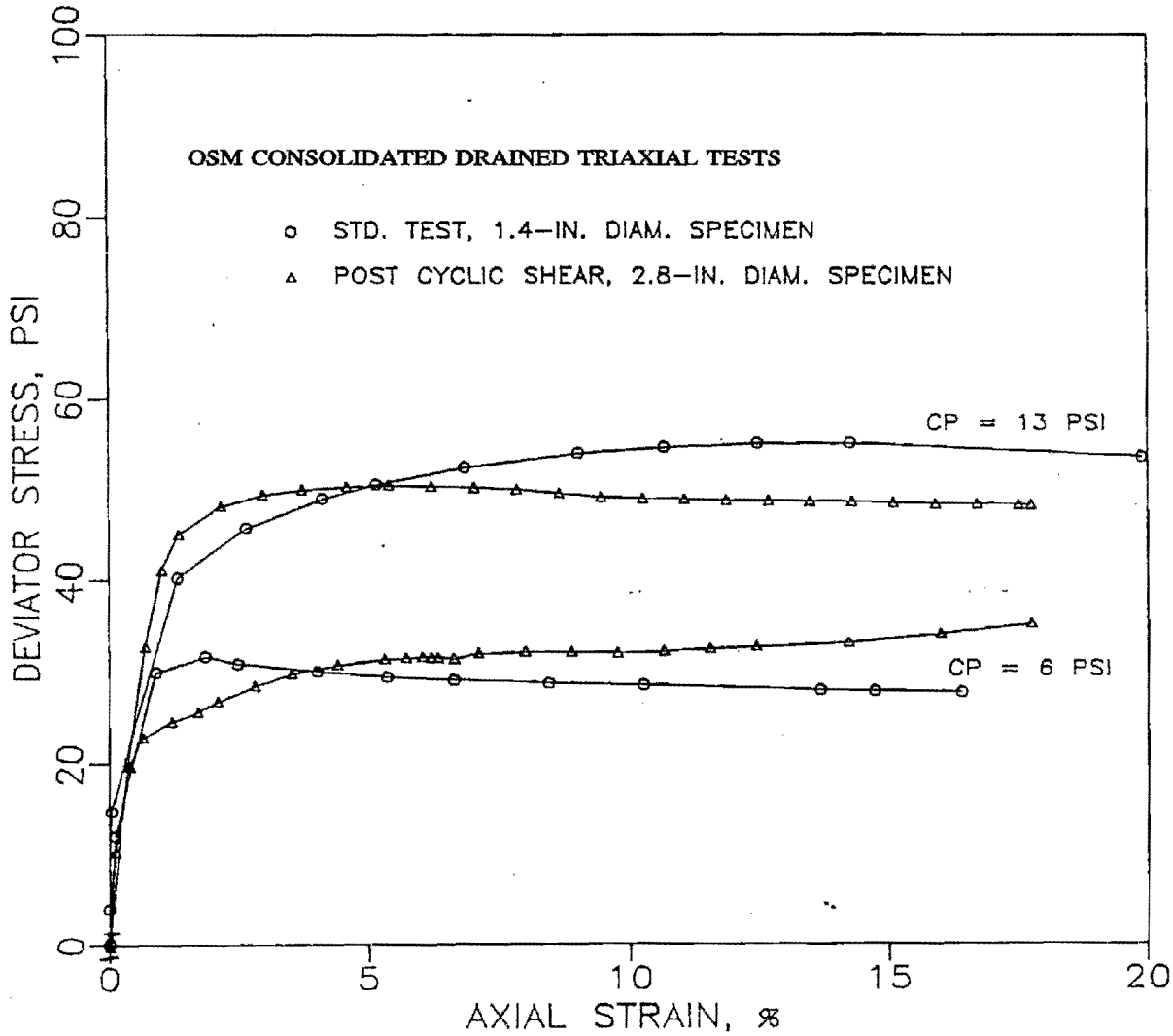


Figure 25. Deviator stress versus axial strain for triaxial tests with and without small amplitude cyclic loading. From Part VIII, Figure 6.

a 1 Hz cyclic deviator stress of about 2 psi above and below the consolidation stress.

The results of the back-pressure saturated, consolidated, undrained triaxial compression test with pore pressure measurements indicated that both artificially saturated specimens tried to dilate during the final monotonic shear phases. Failure occurred at an axial strain of 5 to 6 percent. The effective angle of internal friction was 30 percent, which is fairly large for a clay soil. Dilation inferred that the specimen was highly overconsolidated.

The companion back-pressure saturated compression test did not result in any residual pore pressure mobilization at the ends of the specimen after cyclic loading. There was a small pore pressure oscillation of approximately 0.4 psi during the cycling of the deviator stress. As previously mentioned, this specimen tried to dilate during the subsequent monotonic loading. The maximum deviator stress during shear was 8.4 psi versus 12.8 psi in the companion uncycled specimen. However, the initial and post consolidation void ratios of the cyclically loaded specimen were substantially higher than those of its companion. The void ratio difference fully accounts for the strength difference.

The WES investigators determined that the differences in post-cyclic loading strength in both saturated-undrained and unsaturated drained tests were not large enough to cause foundation instability and are explainable by the small variations in initial water content and void ratios between companion specimens.

3.7.3 Findings

The evidence from the cyclic torsional tests; and the monotonic and cyclic triaxial tests on specimens collected from the study area indicates that blast-induced ground vibrations did not consolidate or destabilize foundation soils. Structure distress in the study area was not caused by synergistic effects between vibrations and soil movement.

3.8 Are there comparable damages in a remote area (unaffected by mine blasting) with similar geology, soils, and topography?

One of the USGS tasks in this study was to make comparative structural inspections of complainant and non-complainant homes in the study area. The purpose of this exercise was to identify causes of damage and to determine if there were trends indicative of blasting effects. Examination of structures in a remote area was also incorporated into the inspection program. This was an attempt to identify types of damage common to the study and remote area.

In addition to USGS inspection results, the summary below includes the remote-area selection procedure; and an OSM comparative analysis of the geology of the remote area and the study area.

3.8.1 Remote Area Inspection

The remote area selection took place in October 1991. The selection criteria were developed by OSM and later approved by the principal investigators of the other agencies. The first criterion required the area to be distant enough from the mine to be unaffected by blast-induced ground vibrations and airblasts. The remaining selection standards required the remote area to be similar to the study area with respect to the following additional primary factors: soil types, topography, and types of construction. Secondary factors were geology, building density (number of buildings occupying a unit area), presence of septic systems, and vegetation.

OSM identified five candidate areas based on topographic and geologic maps: Kasson, St. Phillips, Wadesville, Red Bank, and Heusler. These communities are located on the Evansville North and Kasson, IND. USGS quadrangles. With the assistance of a soil scientist from the Indiana Soil Conservation Service the areas were visited and evaluated according to the criteria listed above. Kasson and St. Phillips appeared to be the best candidates based upon the similarity of primary selection criteria. One seismograph was positioned in each of these communities and confirmed the absence of blast-induced vibrations. Following consultation with USGS and the Geotechnical Lab of WES, and upon receiving their concurrence, Kasson and St. Phillips were chosen to constitute the remote area (Figure 26) and site selections within the area commenced.

OSM attempted to obtain a sample of houses reflective of the sample used in the study area. The majority of the remote area homes are single story and have brick or stone veneers. Other aspects of the structures, such as whether or not they had basements, could not be assessed until OSM personnel entered the residences to seek right of entry or, more often, until the USGS structural inspection was carried out.

The damage inspected by USGS in 19 remote-area houses was not as severe as in some of the complainant structures in the study area, but not much different from that in the non-complainant homes. Typical damages included cracks around door and window openings and ceiling cracks. One structure had severe damage attributed by the owner to faulty construction. Two home owners identified specific cracks as a consequence of earthquakes.

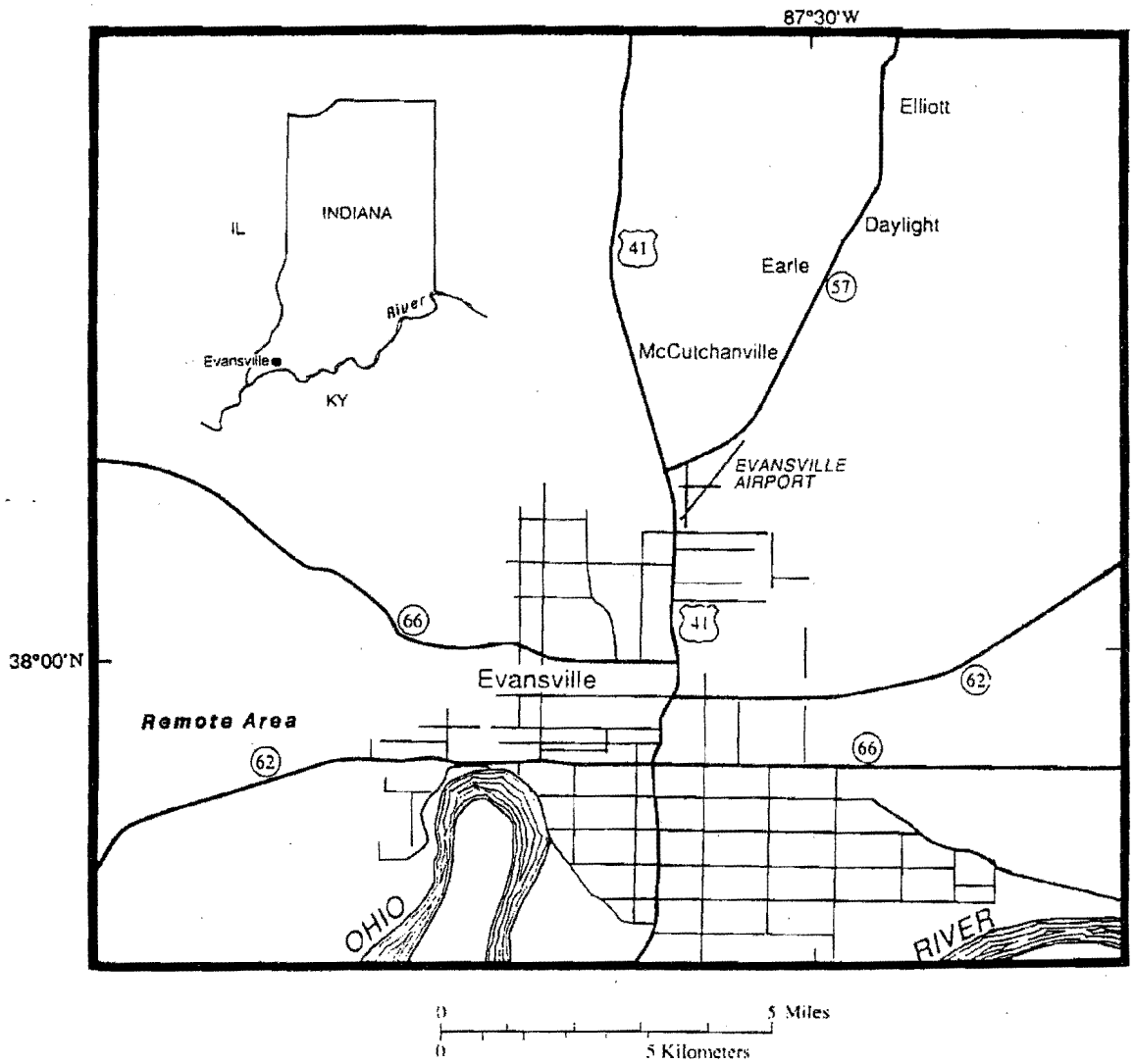


Figure 26. Location of the McCutchanville-Daylight and remote study areas. From Part VI, Figure 1.

3.8.2 Comparative Geology of the Remote and Study Areas

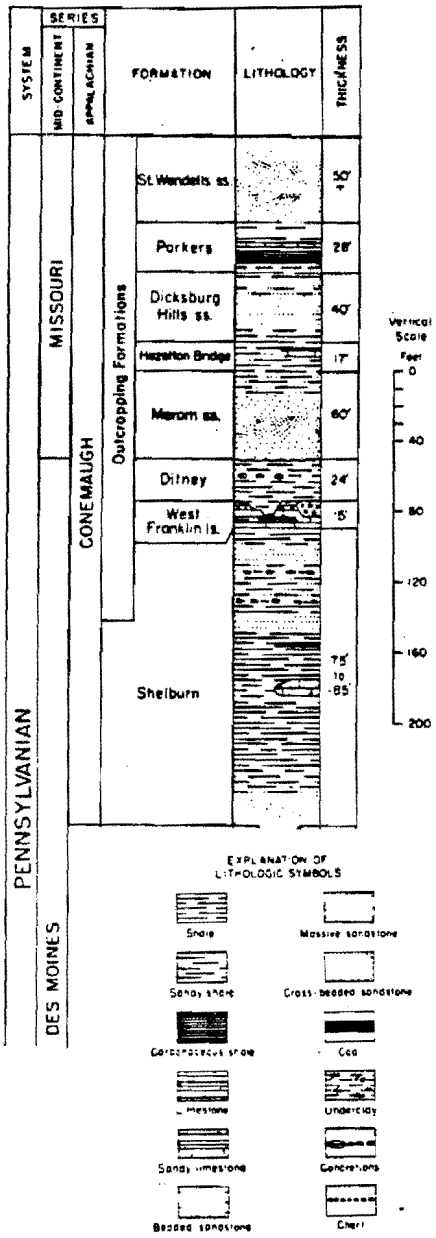
A surface geology reconnaissance of the remote area revealed a difference in strata lithology with the study area (Part IX). The uppermost strata in the remote area is the Merom sandstone which is younger in age and higher in topographic elevation than the uppermost shale of the Shelburn Formation of the McCutchanville ridge in the study area (Figure 27). The apparent thickness of the Merom sandstone was determined by locating the furthest upstream and downstream exposures along a stream bed on Cochise Road (off Schaefer Road). From the elevation difference between the uppermost and lowermost exposures, a bed thickness of 60 feet was determined. Below this sandstone, gullies and streams contained coarse sand stained by iron oxide with limonitic nodules and concretions. Below the iron oxidized sandy zone is the West Franklin limestone followed by the rest of the Shelburn formation. The stratigraphic column for Vanderburg County consists of the Merom sandstone (65 feet), Ditney sandy shale with concretions (24 feet), and West Franklin limestone (15 feet).

Merom sandstone is a thick competent ridge former in the Kasson area and is eroded from the study area. Only weathered evidence of the Ditney unit was found in the remote area. The Ditney unit was not found in the study area. The West Franklin limestone exposures below the Merom sandstone in the remote area are thicker and more competent than those in the study area. Valley-side and bottom exposures of the West Franklin in the control area show a dense grey fossiliferous limestone with varying degrees of solution.

The Merom sandstone might influence the presence and level of damages in the remote area in one way. If the sandstone is significantly more permeable than the uppermost shale in the study area, there should be less ground water around the foundations. This could have resulted in less severe damage in the remote area relative to the study area. However, the permeability of the sandstone is not known.

3.8.3 Findings

Damages to inspected buildings in the remote area were similar to those in non-complainant structures in the study area. However, the damages observed in the remote area were not as severe as those of some of the complainant structures. A probable explanation for this disparity is related to the differences in bedrock geology between the two areas.



Upper most bedrock unit:

Control Area
Merom sandstone

Study Area
West Franklin limestone member
sandy shale or limestone

Figure 27. Generalized stratigraphic column of Vanderburgh County. From Part IX, Figure 2.

3.9 To what extent do alternative mechanisms (expansive soils, hydrology, inadequate foundations, slope/soil movement, piping, etc.) contribute to the observed damages?

This summary covers: (1) USBM house inspections and level loop surveys during the 1989-90 investigation (Part IV); (2) USGS structural engineering inspection of houses (Part VI); (3) WES preliminary and addendum comments on field observations of selected complainant structures and common construction practices (Part VIII); (5) WES analysis of the potential for soil consolidation and bearing failure beneath foundations (Part VIII); (6) WES static load analyses of cracking in basement floor slabs and in basement walls (Part VII); and (7) OSM assessment of the influence of geology on damages (Part IX).

3.9.1 Geo-environmental Setting

Bedrock

The study area is underlain by Pennsylvanian bedrock units of the McLeansboro and Carbondale Groups. The bedrock consists mostly of sandy shale and a thin bed of limestone. Stratigraphic correlation was established by a rock-cored hole located at the McCutchanville Fire Station. The uppermost bedrock of the McCutchanville ridge is sandy shale. The underlying thin bedded West Franklin limestone member of the Shelburn Formation can be toprock along a narrow outcrop zone in the ridge covered by recent deposits. Down-section of the West Franklin limestone member shale prevails. The strike of the strata is north-northeast and the dip is west-northwest at a rate of 25 to 30 ft/mi.

The McCutchanville ridge is considered preserved by the West Franklin limestone. Approximately four feet of the limestone was recovered at the Fire Station at an elevation between 476 and 466 feet. A void was encountered between 472 and 470 feet. This void may represent a solution zone. Additional evidence for solution activity in the study area includes karst-related features along the McCutchanville ridge. Blocks of West Franklin limestone which are displayed as decoration in the front yards of homes along the ridge show rounded edges and features of solution and running water. Also common along the ridge are open-ended/cup-like depressions in the surface, indicating the presence of subsurface drainage.

Soils

Modern soils in the study area are underlain by two loessial sequences that contain several buried soil horizons. The measured depth of the loess ranges from three to nine feet. Colluvium has been found to occur between the loess and bedrock in some locations and varies in thickness from one to three feet.

Loessial deposits can occur as: (1) primary deposits that have a loose highly porous structure and are susceptible to hydrocompaction (compaction due to water application); and (2) secondary deposits resulting from hydro-compaction. The moisture content and dry density of the study area soils indicate that they have experienced natural consolidation; and should be

capable of supporting ordinary structures without settlement from further hydro-compaction.

The capacity for transmitting water or permeability of the soils is sufficiently slow to result in USDA residential development limitations. The soils are subject to seasonal high water tables, ponding, poor drainage, or slope movement. The Hosmer and Zanesville soil series have a slow to very slow water absorption zone ranging from 25 to 36 inches below the surface called a fragipan. This low-permeable surface acts as an aquitard, limiting water infiltration. Seepage of water that has penetrated below the level of the fragipan is impeded by the shale or controlled by solution cavities in limestone.

Occurrences of swelling soils in the study area are discussed in Section 3.9.5.

3.9.2 USBM Level Loop Surveys

The USBM investigators performed two level loop surveys around the seven homes monitored for blast vibrations (see Section 3.2.1). All but one of the buildings were measured from a foundation or brick course. The exception, 107, had to be surveyed in reference to the roof eave. The surveys were performed three months apart and the results were analyzed for evidence of displacements associated with settlement.

The elevation data was handled in two ways: (1) maximum elevation changes between the two surveys were determined; and (2) the ratio of angular distortion was calculated from elevation differences between measuring points. The maximum angular distortion ratio corresponded to the greatest deflection between any two measuring points. The total angular distortion ratio represented the distortion of the total structure.

Among the seven structures, the maximum elevation change between surveys ranged between +0.03 and -0.03 feet. Maximum angular distortion ranged between 1:430 (1 part distortion per 430) and 1:80; and total angular distortion ranged between 1:1730 and 1:174. The significance of these ratios depends on the assumption that the houses were originally level. Five structures had ratios exceeding 1:300, which is cited in the literature as a threshold for cracking of panel and load-bearing walls. Four of these buildings were located on hills, and in all four the lower survey points were on the down-slope end of the house. Structure 107 had the maximum distortion of 1:80, surpassing the referenced threshold for structural damage (1:150). This result is subject to greater potential error than the rest due to the use of the roof eaves in the survey.

3.9.3 WES Analysis of Soil Settlement Potential

The WES Geotechnical Laboratory conducted an engineering analysis for typical structures in the study area to determine probable extent of foundation settlement from estimated static wall loads. This analysis consists of two parts: (1) comparing foundation soil bearing capacity to the estimated loads

and (2) estimating differential settlement from foundation soil consolidation.

The Structures Laboratory provided the vertical wall loading base case from calculated structure dead loads for one- and two-story residential buildings. The base case assumed a square foundation excavation containing a 50 x 50 feet basement with walls supported on a strip footing 20 inches wide and 4 feet below the original ground. This resulted in bearing pressures of 1,260 lb/ft^2 and 3,150 lb/ft^2 for the one- and two-story structures, respectively.

Soil index property data were available from numerous borings in the study area provided by IGS. Shear strength values were obtained from 63 standard penetration (SPT) tests and 15 unconfined compression (UC) tests. SADL conducted 14 consolidation tests, 11 of which were on non-swelling CL soils and 3 on swelling CH or CL-CH soils. All test results indicated that the soils were preconsolidated; that is, they had been subjected to a vertical stress larger than the current overburden stress for some geologic time period. The tests also indicated low initial void ratios, which are consistent with preconsolidation.

Bearing Failure Potential

The bearing capacity of a 20-inch wide strip footing was calculated under the very conservative assumption that it was a surface strip footing (instead of being 4 feet deep) (Figure 28). A factor of safety of 3.0 was chosen. For a two-story building, the allowable bearing capacity exceeds the estimated stresses when using shear strength values exceeded by approximately 85 percent of the test data. For a one-story building, the capacity is greater than the estimated stresses when using strengths exceeded by 95 percent of the data. The investigator concluded that bearing capacity failure was not a reasonable scenario for the footing size and load in the base case and the soils encountered in the subsurface investigations.

Consolidation Potential

Estimates of foundation settlement from soil consolidation were based on: (1) analysis of soil data to determine pre-consolidation pressure, compression index, and rebound compression index; (2) calculation of vertical stress increments as a function of depth under the middle of a footing and at a basement corner due to excavation of the basement and the addition of the wall loading; (3) calculation of immediate settlement under the footing load; and (4) calculation of long term settlement.

Vertical stress versus depth: The Boussinesq Solution was used to calculate vertical stress increments in a linear-elastic medium under a corner and the mid point of a one-story-structure strip footing around the perimeter of a 50-foot square excavation 4 feet deep due to the combination of the excavation (a decrease in stress) and the loaded strip footing (an increase in stress). The results showed a net increase in stress in the top 7 feet below the footing. This meant that only shallow depth soil properties had any influence on settlements. It was further noted by the investigators that the maximum stresses were less than the measured pre-consolidation pressures.

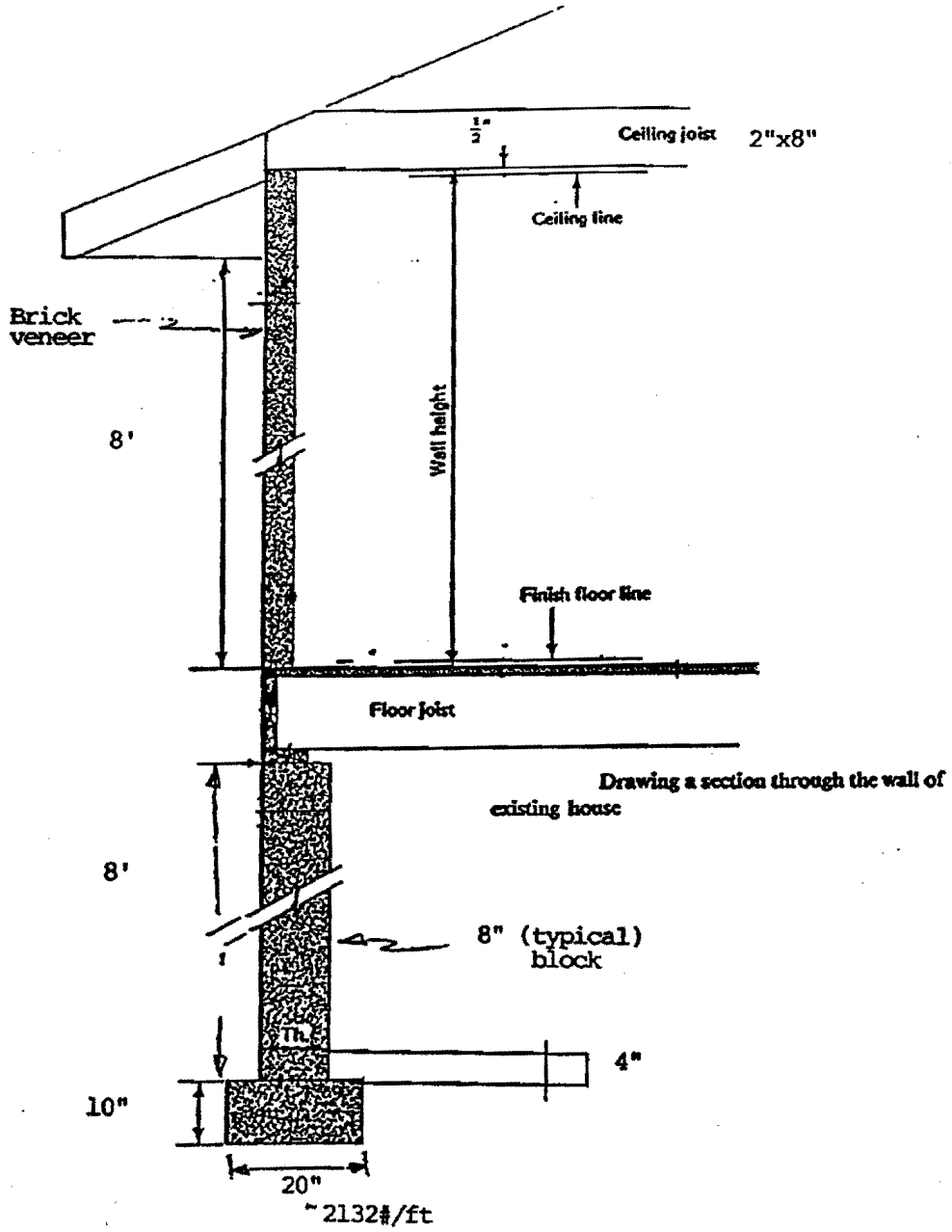


Figure 28. Idealized cross section of the one-story house for estimating footing loads. From Part VII, Figure 3.1.

Immediate settlement: A theoretical solution based on the theory of elasticity for the elastic settlement under a uniform, infinitely long strip loading on a linear-elastic half space was used to calculate immediate settlement. The calculated settlement was 0.03 inch for a one-story structure and 0.045 inch for a two-story structure.

Time dependent settlement: The time dependent settlement calculations were based on the rebound compression index (since maximum stress increment is less than preconsolidation pressure), a void ratio of 0.6, and the stress vs. depth relationship. For the one-story building, maximum time dependent settlements of 0.18 and 0.11 inch were calculated for the centerline and corner, respectively. Centerline and corner settlements of 0.23 and 0.13 inch, respectively, were computed for the two-story building.

Differential Settlement and Field Observations: The calculated total settlements was 0.14 inch at the corner and 0.21 inch at the centerline for the one-story structure. The resulting differential settlement was 0.07 inch Corner and centerline settlements for the two-story building were 0.275 inch and 0.175 inch, respectively, resulting in a differential settlement of 0.1 inch. Maximum differential settlements reported from the USBM 1989-90 investigation for seven houses in the study area were greater than one inch. Differential settlement larger than those calculated could only be explained by elastic and consolidation settlement only if (1) the soil profile was nonuniform (one side of the building was founded on or close to rock and the other side was founded on at least 20 feet of soil) and the compression rebound index was substantially larger than measured or (2) the pre-consolidation pressures reported are wrong. There was no evidence to suggest either of these possibilities and there was strong internal consistency among the soils data. The investigator concluded that most of the differential settlement occurring in those buildings was from some other cause.

3.9.4 WES Analysis of Cracks in Floor Slabs and Basement Walls

Floor Slabs

The WES Structures Laboratory conducted a static analysis into the potential for cracking in basement floor slabs from foundation settlements observed in the study area. Some structures in the study area were observed by WES during the February 1991 interim field evaluation to have both footing settlements and basement floor cracks. The question arose as to whether the cracks in the concrete slabs were caused by the settlements or by some other mechanism. In order to verify a potential relationship between these two damages, the deflection necessary for stress to exceed the cracking strength of a two dimensional slice of slab was estimated.

The tensile strength of concrete can range between 230 and 400 psi. The WES calculations treated a slice of the floor slab as a beam. As the footings settle, vertical soil pressures develop under the slab (Figure 29). Based on the tensile strengths above, deflection from settlement necessary for cracking ranges between 0.7 and 1.2 inches. The results indicate that settlements of approximately one inch that were observed at some houses are sufficient to cause cracking in the floors.

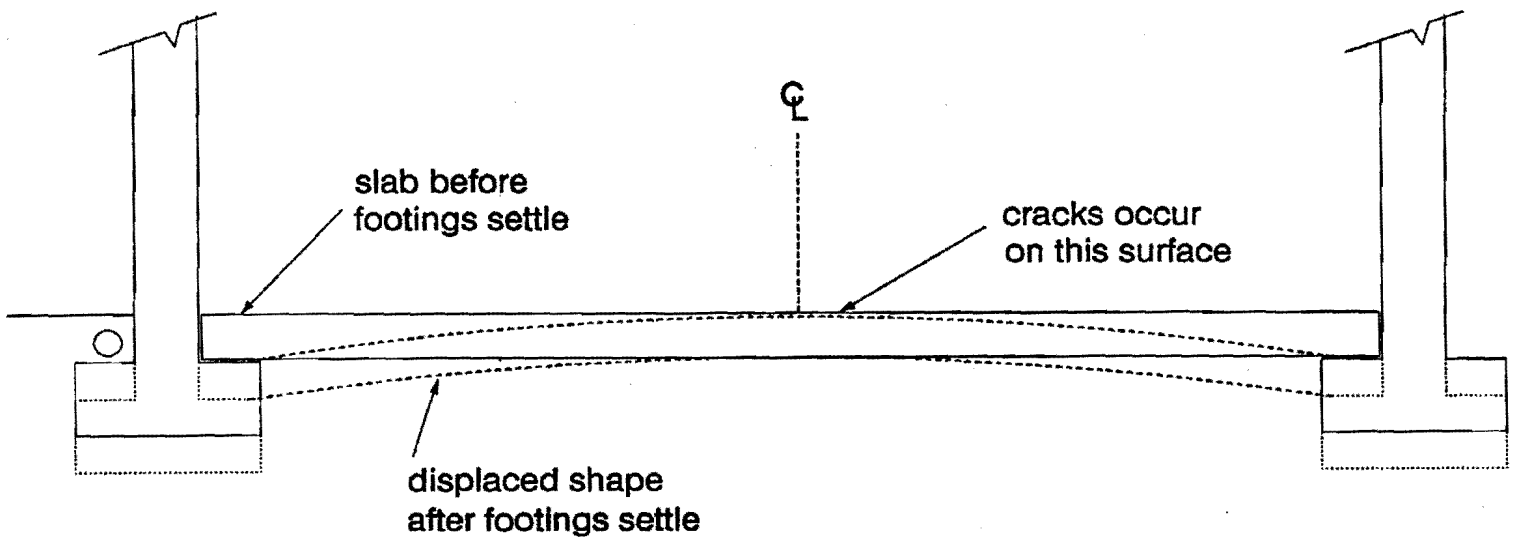


Figure 29. Idealized displaced shape of concrete slab after footings settle. Tension occurs on top surface. Amplitude of displaced shape is exaggerated to illustrate. From Part VII, Figure 3.3.

Basement Walls

Cracks in the mortar of basement block walls were also commonly observed during the interim field evaluation. The WES investigators performed a static load analysis to confirm the potential relationship between soil pressure and these material failures. The bounding values of lateral pressure estimated by the WES Geotechnical Laboratory included the value for confined swell pressures in expansive soils.

The Structures Laboratory applied the lateral pressure against estimated tensile strengths of the blocks and mortar. The values of maximum tensile stresses on the interior basement block wall vary from 19.8 psi to 220 psi. Based on the tensile strength of mortar (the "weak link" between blocks) ranging from about 14 to 82 psi, the investigators determined that cracking could occur in the mortar joints from static loads alone.

3.9.5 Assessment of Damage Causing Processes

In addition to inspecting six homes for crack changes during the 1989-90 investigation (see Section 3.5.1), USBM researchers inspected existing damages in these houses and three other complainant structures; and assessed the potential causes of the damages. Fifteen buildings (13 residences, 2 churches) were inspected by representatives of WES, USGS, and USBM during the interim field evaluation. USGS inspected 33 buildings in the study area during several field visits between November 1991 and December 1992. The thirteen residences mentioned above were complainant structures and the rest of the homes were used as non-complainant comparisons (see Section 3.5.3). OSM assessed the influence of geological conditions on structures in Part IX, which includes case studies of four structures. This included the compilation of available data, field observations, and statistical analysis. Available data included that obtained during the course of the study and information in the literature. The sources are enumerated in Part IX.

A summary of observations and interpretations pertinent to damage-causing factors other than blasting are presented below.

Swelling Soils

Contributions from OSM and the WES Geotechnical Lab to the discussion of the potential influence of swelling soils on structures are summarized as follows:

IGS and SADL tested selected soil samples obtained from drilling for clay mineralogy. IGS found the expansible clay mineral, smectite, in 38 samples from 12 drill holes. SADL testing program found smectite in one sample from 7 drill holes. Soil samples from 25 structural sites were tested for Liquid Limit and Plasticity Index by IGS, SADL, and AmTech Engineering, Inc. during this study and by Engineers International, Inc. during an earlier OSM

subsidence investigation. These two properties determine swell potential.[3] Loessial samples from 4 sites had medium swell potential. Colluvial or weathered-shale samples had either medium to high (9 sites) or high (12 sites) swell potential. The amount of heave that could result from soils of medium and above swell potential range from 0.3 to 1.2 inches.

A few sets of flooded consolidation tests conducted by SADL suggest swell pressures of 0.6 to 2.5 tn./sq ft at the 5- to 10-foot depth range. This would be enough to deform and severely crack unfilled, unreinforced concrete basement walls of the types observed in February 1991.

Appendix H in Part IV of this report includes two articles that hypothesize a mechanism for swelling soils causing damages to basement walls. A sequence of events is described, beginning with an extended period of low precipitation. Soils with high swell potential that surround a structure become very dry, shrink, and pull away from the foundation walls, leaving gaps for soil particles and other debris to enter. Additional material at various depths against the foundation augments lateral pressures as the soil swells in response to increased precipitation. After one or more shrink-swell cycles, lateral pressures may be enough to engender cracks in basement walls.

The occurrence of this process in the study area would depend in part on the presence of swelling soil in contact with basement walls and within a soil zone where significant drying could occur. Soil tests indicate that moderate swell potential in loess and at shallow depths does occur around some structures in the study area. Where occurrent, shrink-swell might have bowed in and cracked concrete-block basement walls. Swelling soils appear to be predominantly positioned below the loess, in weathered shale or colluvium. Here they can possibly generate lateral and vertical pressures on basement-wall footers. However, deeper soils are less affected by evaporation and the weathered shale, as an aquitard, can limit ground-water flow away from a site.

Significant shrinkage and swelling of soils would also require sufficient fluctuations in precipitation. Between 1949 and 1992, annual precipitation ranged between 27.88 (1963) and 60.13 (1950) in./yr. (Table 6). Cycles of decreasing-to-peak annual precipitation have periods ranging from 2 to 6 years and variances from 1.27 to 19.73 in./yr. The cycles with variances equal to or greater than 15 in./yr. terminate in the years 1953, 1957, 1977, 1982, and 1990. The 1990 precipitation cycle is the only one with a period of six years: and has a variance of 18.01 inches. This cycle includes the beginning of cast blasting and the time (1987-88) during which complainants allege the strongest vibrations and most serious damages occurred. Precipitation in both 1987 and 1988 was relatively low, with 34.51 and 38.43 in./yr., respectively; and significant increases--and the greatest potential for swelling on a yearly basis--did not take place until 1989 (47.34 in./yr.)

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[3]Plots of soil sample data for Plasticity Index vs. Liquid Limit and swell potential classification are presented on pp. 70 and 71 in Part VIII.

A. Annual Precipitation 1949 through 1992
Evansville, IN Regional Airport: Weather Service Office (WSO)

Year	Inches	Year	Inches	Year	Inches
1949	**53.54**	1964	38.37	**1979**	**52.21**
1950	**60.13**	1965	35.03	1980	35.76
1951	**51.37**	1966	36.53	1981	43.35
1952	33.93	1967	43.19	* **1982**	**52.68**
1953	**53.66**	1968	43.21	1983	48.48
1954	33.83	1969	49.23	1984	49.75
1955	40.52	1970	45.93	1985	45.89
1956	33.93	1971	40.25	* 1986	37.68
1957	**53.66**	1972	42.27	* 1987	34.51
1958	42.22	1973	46.18	* 1988	38.43
1959	45.12	1974	43.27	1989	47.34
1960	34.48	**1975**	**51.01**	* **1990**	**52.52**
1961	47.38	1976	32.09	1991	32.68
1962	40.91	**1977**	**50.08**	1992	35.81
1963	27.88	1978	42.96		

B. FREQUENCY AND DURATION OF WEEKS WITH 0.20 IN. AND LESS OF PRECIPITATION PER YEAR. (# = FREQUENCY EACH DURATION OCCURRED)

YEAR	ANNUAL PRCP (ins.)	ONE WEEK	TWO WEEKS	THREE WEEKS	FOUR WEEKS	FIVE WEEKS	TOTAL - WEEKS (DAYS)	PERCENT OF YEAR
1982	52.68	10	2	--	--	--	14 (98)	26.8
1983	48.48	6	3	2	--	--	18 (126)	34.5
1984	49.75	6	--	2	--	1	17 (119)	32.6
1985	45.89	9	2	1	--	--	16 (112)	30.7
1986	37.68	11	4	1	--	--	22 (154)	42.2
1987	34.51	7	4	2	--	--	21 (147)	40.2
1988	38.43	9	4	1	--	--	20 (140)	38.3
1989	47.34	4	1	--	1	1	15 (105)	28.7
1990	52.52	5	3	--	--	--	11 (77)	21.1

Table 6. A. Annual precipitation in inches for 1949 through 1992, and B. Frequency and duration summary of weeks with 0.20 in. and less of precipitation 1982 through 1990. From Part IX, Table 2.

and 90 (52.52 in./yr.). However, on a weekly basis the low-precipitation years of 1986-88 are roughly characterized by relatively long intervals of low precipitation between shorter periods of heavy precipitation (Figure 30). It is possible that shrink-swell cycles, where occurrent, are most active during this weather pattern.

Direct data on the occurrence of shrink-swell cycles in the study area are not available. The amount of precipitation variance and the duration of a dry period required for this mechanism to work in the study area soils are unknown. If shrink-swell cycles have occurred, it is reasonable to suspect that structures have been affected several times under the documented climatic conditions. For example, many of the complainant homes were present during significant (15 in/yr variance or greater) pre-1990 precipitation cycles. As of the 1989 OSM survey three significant cycles had occurred during the previous 15 years. Approximately 70 percent of the structures had existed for at least this length of time. About 40 percent of the complainants had occupied their dwellings this long.

Erosion

Evidence for erosion around structural foundations was identified by USGS, WES, and OSM at a total of 7 of the 15 inspected complainant structures. The evidence included sediment in the outlets of underdrains; deposits in basements and crawl spaces; sink holes; and solution features in limestone.

Underdrains: In the March 1991 Interim Field Evaluation report (Appendix A, Part VIII), the WES Geotechnical Lab investigator noted that the predominantly CL soils found in the study area are moderately erodible and recommended that underdrains around the outside of strip footings should be built as filters. Three homeowners described the installation of their footing drains. The WES investigator concluded that the drain filtration systems were inadequate. Also, CL soils partially or totally filled underdrain outlet pipes at three residences. The investigator stated that this material could have been eroded from beneath footings, resulting in differential settlement of foundations.

Material transport into basements and crawl spaces: Sediment that had been washed into basements or crawl spaces was observed at several complainant structures. The material is derived from below or immediately adjacent to the structures, which is evidenced by a shallow subsurface void discovered by a home owner below a garage floor slab, a collapsed garage floor slab, and an 18-inch diameter hole against an outside wall. At one residence, a large volume of soil material was washed from outside the structure, beneath the footing, and into a crawl space. The gap between the footing and the ground was approximately 1 foot high, 1.5 feet long and open into the crawl space. Accumulations of soil material were also observed to be associated with cracks in basement floors and walls in several other homes.

Natural piping: Indications of piping (subsurface water transport) of unconsolidated material other than through drain pipes and structural walls or footings include: swales; piping outlets in valley sides of tributary re-entrants along the McCutchanville ridge; and solution features in the West

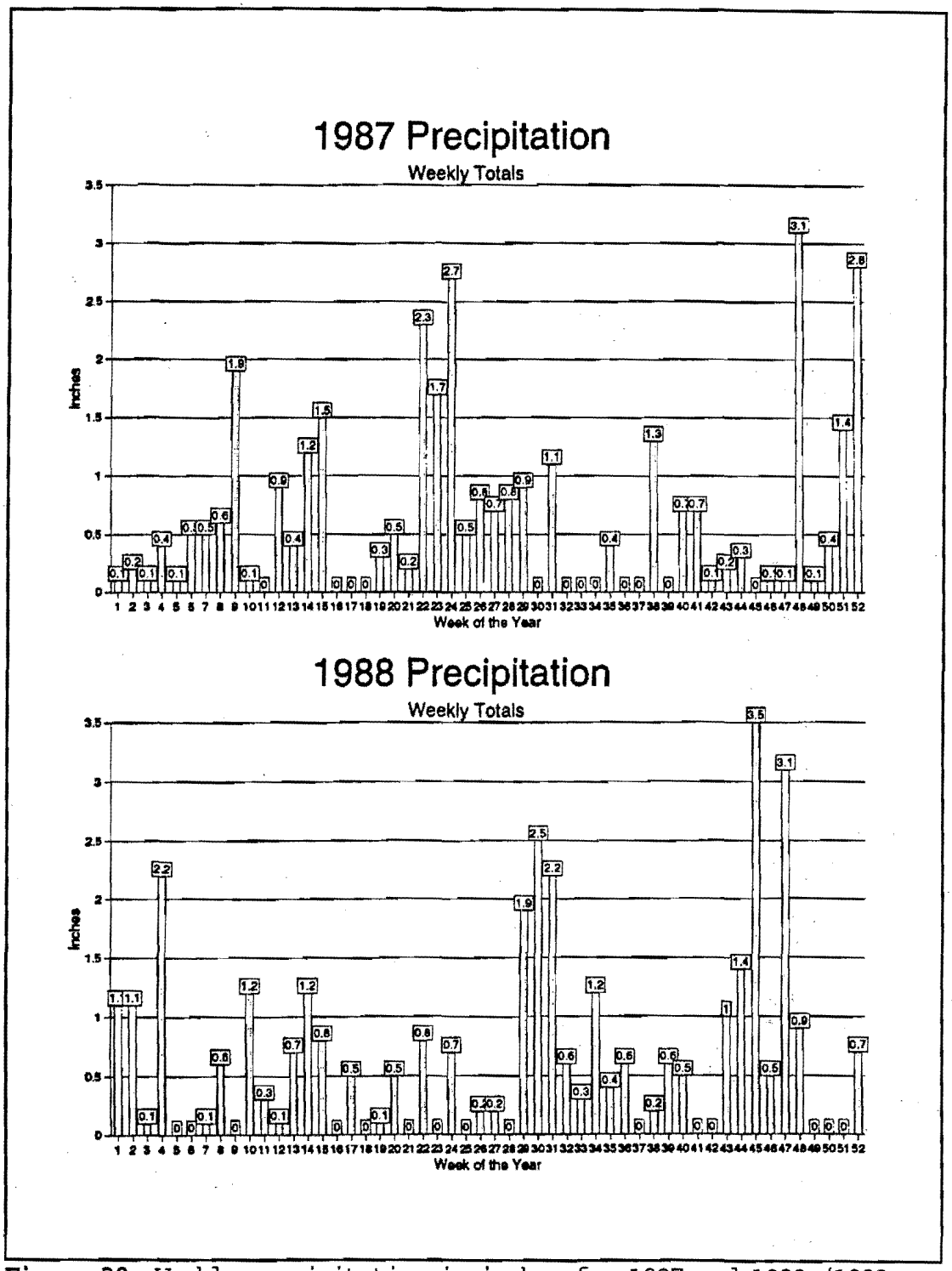

Figure 30. Weekly precipitation in inches for 1987 and 1988 (1992, USDOC, NOAA, NCDC). From Part IX, Figure 3.

Franklin limestone. Surface runoff encountering linear objects (fence posts, buried pipes, tree roots, etc.) can slow, infiltrate, and drain in subsurface streamlets that may emerge from valley slopes, flow as a surface stream, and again disappear beneath the valley bottom. Stream emergence may occur from outcrops of colluvium, loess-bedrock contacts, limestone, or clayey paleosols.

Evidence for piping into limestone was found at a structure OSM investigated for mine subsidence damage in 1985 (house 202). One angle core hole was drilled. The upper-most layer of bedrock was the West Franklin limestone which occurred at 11 feet below the surface. Drilling water was lost at one foot below top of bedrock. The water loss is interpreted as indicating flow through open fractures (reported in the drill log) in the rock. Loose limestone blocks and in situ limestone exposures on the property exhibit solution channels and other solution features. During the USGS inspection of the companion home (202A), the occupant described the appearance of holes in the ground. One required several cubic yards of dirt to fill it.

Some evidence was also found at another structure (house 301) that was 250 to 300 feet from a valley exposure of the limestone. The house is estimated to sit 12 to 15 feet above the West Franklin stratigraphic position. A gap beneath a footing of the structure was identified during the USGS inspection. Soils were washed through the gap into the crawl space from outside. A piping hole was found in the floor of the crawl space, indicating that the material was also being transported somewhere below the foundation. Three holes were augered in part to verify the presence of limestone below the structure. Indications of limestone were limited to the chattering of auger rods at one hole at the estimated West Franklin depth and the recovery of limestone associated "terra rossa" soil at another.

Slope Movement

Part IX references the findings of the USBM level loop survey discussed in Part IV and summarized in Section 3.9.2. The apparent foundation settlements on the down-hill side of structures suggest the influence of slow-moving, en-masse displacement of soil down slope. Evidence for slope movement includes steeply dipping, slicken-sided joints in expansible clay found in auger cuttings at structure 209. The WES report on the interim field evaluation mentions tree growth patterns and parallel cracks in a driveway as potentially related to slope movement. At least some of the damages at four complainant homes were tied to down-slope movement during the USGS inspections and the interim field evaluation. These include concentrations of cracks on the down-slope side of a structure and the horizontal spreading of a crack in a basement floor slab. During the interim field evaluation, the front-porch posts of one home were reported to have lost contact with the floor in March 1989. This was interpreted in the WES report as consistent with the building trying to rotate downward in the down-slope direction.

Drainage

There were many observations of damages associated with actual or potential concentrations of water (wet ground; ground below damaged or clogged roof

gutters; ground around downspouts, underground drains, septic tanks, etc.). Not all of the damages could be connected with the mechanisms of erosion or creep.

Shrink-swell can be considered a potential mechanism for some locations, keeping in mind the uncertainties discussed in the "Swelling Soils" section above. Among six structures with reported damages that are apparently unrelated to erosion or slope movement (104, 107, 108, 114, 224, and 421), soil samples from three (107, 108, and 421) tested for Atterberg Limits at 2.5 foot intervals. Testing at two of these three sites reached the depth of weathered shale. Swell potential was identified in the weathered shale, which occurred at depths of 5-9 and 9-12 feet.

Whether or not shrink-swell has taken place, concentrations of water below and adjacent to foundations could possibly have caused differential structural settlement and/or distress in basement walls by increasing soil plasticity (i.e., decreasing shear strength) and affecting lateral loads. Examples of settlement where ground is poorly drained--and where a decrease in soil strength might have occurred--include a sagging garage floor slab in house 104, a concentration of outside wall cracks in building 224, and stair step cracks in a concrete block garage at house 421. Another indication of the influence of water pertains to the reported collapse of a basement wall from water pressure during the construction of house 421. Part IX discusses the presence of two sump pumps in house 107 and the possible effect of <u>decreasing</u> water pressures around the foundation.

Frost Action

The addendum of Part VIII mentions frost action as a potential mechanism of damage. This applies to foundations and other construction units above the frost line. USBM documented frost-related problems at house 209 in the 1989-90 investigation.

Construction Practices

Comments of the Geotechnical Laboratory of WES on potential causes other than blasting of long horizontal basement wall cracks are reviewed in Section 3.5.6. These causes include the observed construction practice of using short dowels to connect the superstructure with the top course of unreinforced basement wall block. Thermal expansion and contraction in the superstructure should be transferred to the top course of block, resulting in stresses between the top and lower blocks.

Other observations made by USBM, USGS, WES, and OSM of construction practices that could make damages more likely are: (1) reduction in thickness of concrete block at or near ground level from the basement wall to the base of brick facades; (2) structural additions and structures founded on different types or depths of footings; (3) addition of full basements after superstructure construction; (4) unconnected step footings; (5) lack of reinforcement in foundations, basement walls, or concrete floors; (6) lack of midwalls or pilasters to support basement walls and (7) lack of control of groundwater flow into excavations. The first three can effect differential

response of a structure to environmental stresses. The other practices may result in insufficient resistance of wall, floor, and foundation members to movements and pressures exerted from surrounding materials.

Distress in structural members supporting building superstructures was noted by USGS at two homes. In the crawl space of house 301, a long steel beam supported floor joists. In addition to the end supports for the beam, there were two intermediate concrete block supports. The steel beam was supported on the top of the concrete blocks by a single brick at each end. These bricks were crushed. The beam lined up with vertical cracks in the brick veneer outside the house. In house 115, the floor of the master bedroom seemed to slope towards a waterbed in the middle of the room. The joists supporting this floor could be examined in the basement. The joists bulged at their sides above their end supports and thus appeared to be partially crushed.

3.9.6 Conditions Influencing Presence and Degree of Damage

As documented in Section 2.4 and Parts IV, VI, and IX, the degree of damage in homes in the study area ranges between very little to none and severe. The influence of construction and site conditions on structure response to cyclic forces is evaluated in Part VI and summarized in Sections 3.2.2 and 3.5.2. The response of buildings to other, non-cyclic influences also depends on construction and site conditions. Aspects of construction that can result in damage are summarized in the previous section. Size, shape, foundation type, and other building characteristics vary in the study area. The construction practice applied to a particular structure should have at least partially governed its damage level.

Part IX contains a general assessment and statistical treatment of environmental conditions that potentially affect damage level. The factors summarized below emphasize the role of water around and within a building and its foundation type.

Relative Up-slope Water Catchment Area

Drainage area up-dip of structures determines the quantity of water that will collect, drain down slope, down-dip, and encounter structures. The volume of up-dip water affecting a structure depends on several factors such as: the size of the catchment area; the amount of precipitation received and the rate of evapotransporation; and rates of ground-water retention and discharge. Relative up-dip drainage can be qualitatively assessed by: studying a topographic map and making field observations; estimating the reach of the catchment area base on the regional dip direction and the location of drainage divides; and observing the size of the catchment area, the presence of streams, and the amount of contour crenulation.

Geologic Structure

Precipitation and waste water reaches bedrock along western slopes of the McCutchanville ridge by infiltrating the loessial profile and colluvial zone.

Water then drains along weathered (clay) and competent shale. All ground water movement follows the buried landscape and bedrock dip-slope.

These conditions suggest structures on the western slope will have more water around basement walls and foundations. Most of the complainant structures are on the regional dip-slope. The bedrock is shale with colluvium mantling the down-slope side of the ridge axis. The amount of accumulative water increases down slope. Structures with basements intercept water that contributes to basement wall and foundation wetness.

Precipitation and waste water should infiltrate the eastern slopes of the ridge until it encounters permeable bedrock and flows along the regional dip (to the northwest) or along the buried landscape to the valleys. Ground water movement may fill depressions and form springs, ponds, or streams; or emerge at locations where man has interrupted ground water flow from ridge to valley floor.

Foundation Type

Structures with basements tend to have greater damages than structures that have crawl spaces or structures built on slabs. Basement foundations that extend below the fragipan level are commonly in contact with shale bedrock. Since both materials behave as aquitards, houses with basements are more likely to be influenced by the presence of water. If shrink-swell occurs in some parts of the study area, the presence of swelling soils around a foundation could also influence damage level.

Septic Systems

The study area is a rural residential area primarily dependent on private "septic tank" sewage systems. Less than 20 percent of the 60-square-mile study area is serviced by public sewage facilities. Eighty-five percent (90 of 104 structures) of the homes in the 1989 OSM damage survey were on septic sewage systems. Eleven percent (12 of 104 structures) were on public facilities. Waste water disposal systems for the two remaining structures are not known. By reference to the USDA county soil map information, all of the complainant sites known to utilize septic systems have severe limitations for construction of septic tank absorption fields.

On the assumption that water used is water discharged, it is estimated that a family of four discharges 312 gallons per day (gpd). The yearly discharge from a family of four would be 113,958 gallons per year (gpy). For a 200 foot square absorption field, 114,000 gpy is equivalent to 4.5 in./yr. of precipitation. This increases the amount of available shallow-subsurface ground water that can affect the stability of foundations and basement walls.

Artificial Water Bodies

The residential area along the McCutchanville ridge contains more than 50 artificial water bodies. Their construction may have raised local water table levels and increased moisture levels around nearby structural foundations.

3.9.7 Findings

Several non-vibratory processes are damaging buildings in the study area. The following processes have been identified: soil erosion into basements and drainage outlets; lateral, downslope spreading of soils; inadequate drainage around foundations; natural piping of soil materials; and distress in joists or beam supports; shrink-swell; failure of basement walls from lateral earth pressures; and structural weaknesses related to construction practices. The evidence suggests that non-vibratory processes are the primary mechanisms of damage in the study area. This finding results from many visual observations in the field indicative of foundation and other structurally related problems. The evidence for non-vibratory mechanisms far outweighs theoretical worst-case scenarios that predict, with no empirical data confirmation, only a slight possibility of blast-induced cracking in brick veneer mortar joints.

The level of damage caused by non-vibratory processes depends on the construction practices applied to a building; and geologic and cultural conditions affecting the soils and hydrology around the foundation.

4.0 SUMMARY AND CONCLUSIONS

4.1 Summary

Blasting energy not used to break rock above coal seams is released into the environment in the form of ground vibrations and airblast. Both side effects cause a structure to vibrate. Consequently, various levels of strain develop throughout the structure.

Building materials within a structure can be damaged when critical strains are exceeded. Strain-inducing loads include natural loads (soil pressures, temperature changes, humidity changes, earthquakes, etc.) and man-made loads (mine blasting, household activity, vehicular traffic, etc.). Man-made loads can be controlled whereas natural loads cannot. Any type of load or any combination of loads can result in stresses in construction materials that exceed their critical strain. The capability of the structure to resist the damaging effects of these loads is dependent on the building materials, construction methodology, workmanship, and age.

Blast vibrations induce either restrained or free structure response. Restrained response occurs in the below ground-level portions of the structure. The below-ground structure response is nearly identical to the measured ground vibrations. Free response occurs to the above-ground-level portion of the structure. Free response is best explained using the analogy of a flag pole. When the base of the pole is shaken, the top shakes at a greater amplitude and over a longer period of time. It is this ability to respond freely (i.e. without confinement) that results in structure amplification of ground vibrations. Airblast also induces free response. The amount of stress and strain that takes place in the structure is directly related to the structure's amplification of a particular vibratory load. Structure amplification is a function of the frequencies as well as the size of the load.

The amplitude of the structure response to ground vibrations and resultant strains within building components are also dependent on the efficiency of energy transfer from the foundation to the framework and wall components. The efficiency of energy transfer increases significantly when the natural frequency of the ground vibrations matches the natural frequency of the structure.

A wide variety of structure types exists in the study area. The observed damages at complainant homes ranged from threshold to major. The effects of both natural and man-made stresses were evaluated in the study area to identify the most probable causes of structural damages.

Man-Made Stresses

The focal point of this study was blast-induced ground vibrations and airblasts. Other man-made stresses include normal household activity (door slamming, walking) and vehicular traffic (cars, trucks, trains, airplanes).

Blast-induced ground vibrations in the study area were not found to be unusual relative to the results of other studies. The attenuation rate of blast vibrations in bedrock was comparable to another region monitored under a previous USGS study. Propagation of ground vibration amplitudes monitored for this study were similar to that reported by USBM for the same mine (RI 9226). Relatively efficient seismic propagation occurred in a northwesterly direction towards Baseline Road.

Topographic enhancement of vibrations was determined to be negligible. The effect of site response was more significant, i.e. compared to bedrock vibrations, soil vibrations were higher in amplitude by a factor of 2 to 4. When site-response at complainant and companion non-complainant structures were compared, most of the differences were negligible. The exceptions were two complainant sites which exhibited greater response over their companion sites by factors of 1.5 and 1.6. This indicates that there are some locations within the study area where site conditions can contribute to differences in structure response to blasting or other sources of ground vibration.

The natural frequencies of the soils closely matched those of the structures. This contributed to the residents' discernability of the blast vibrations. When the two natural frequencies match, structure amplification of the ground vibrations and the potential for damage also increase. However, this study did not result in any empirical data that indicated there was sufficient structure response for damage to occur. Peak structure response from blast vibrations in McCutchanville was 0.096 ips at a corner and 0.112 ips on a midwall. Peak structure response in Daylight was 0.11 ips at a corner (no measurements were made for the midwall).

Normal household activities recorded in a structure in McCutchanville generated structure response comparable to that caused by blast vibrations during the same monitoring period. Jumping on the floor generated structure response of 0.055 ips at a corner and 0.36 ips on a midwall. Vehicular traffic, in particular aircraft landings and takeoffs, were recorded in McCutchanville. Peak structure response was 0.009 ips at a corner and 0.034 ips on a midwall.

Ground vibrations with very low frequencies (below 4 Hz) were outside the natural frequency range of both the ground and structures. There was no evidence of very low frequency vibrations causing significant internal stresses in any structures.

The available evidence suggests that the use of different blast designs did affect peak amplitudes of ground vibrations in the study area. There may have been blast-induced vibrations in the study area that exceeded the PPV of ground vibration recordings during IDNR monitoring and during both monitoring phases of the OSM study, particularly during the summer of 1988. The maximum amplitudes that could have occurred are predicted in this study to be 0.38 ips in Daylight and 0.17 ips McCutchanville. These predictions are derived from ground vibrations monitored at many locations in the study area and, therefore, should account for variations in site response. From all of the

ground vibration data available to OSM, the maximum recorded PPV values were 0.2 ips in Daylight and 0.06 ips in McCutchanville.

The linear-elastic FE analysis of one- and two-story structures estimated the structural strains and stresses in building materials from ground vibrations. Subjecting the simulated buildings to the Daylight worst-case ground vibration resulted in maximum tensile strains well below documented critical tensile strains of plaster and wallboard. The results of the analysis indicate that damages to wallboard and plaster walls in structures with normal foundation or superstructural conditions should not have resulted from amplitudes below 1.0 ips. An FE analysis was also applied to a brick veneer wall. Assuming a realistic damping range of two to five percent, a ground vibration PPV equal to at least 0.4 ips is required for threshold cracking in the mortar to become a theoretical possibility. These cracks would not necessarily be visible and would not compromise the integrity of the wall. Neither the 1.0 ips for plaster or the 0.4 ips for masonry compare favorably with published research on observed damages caused by ground vibrations. This may be attributable to unaccounted prestresses in the wallboard and the difficulty in identifying masonry cracks.

The available evidence suggests that fatigue failure of building materials from the cumulative effects of mine blasting was not responsible for damages in the study area. There is no evidence for the occurrence of fatigue in brick veneer mortar in the study area.

Evidence from vibratory and triaxial soil tests indicates that blast-induced ground vibrations have neither consolidated nor destabilized foundation soils enough to generate structure distress. Damages in the study area have not been caused by a synergistic effect between vibrations and soil movement. Furthermore, below-ground structure response to ground vibrations is restrained by the surrounding ground and exhibits no amplification. Construction materials below ground level subjected to the predicted ground vibrations would not have experienced significant stress, barring any influence from the responding superstructure.

There is no evidence that airblast caused enough structure response to induce damage. However, meaningful data is lacking. Atmospheric conditions strongly affect airblast propagation and prohibit prediction of overpressure levels during non-study periods.

The evidence obtained in this study indicates that man-made stresses, including mine blasting, did not cause the observed damages to structures within the study area.

Natural Stresses

Natural stresses result from soil conditions, moisture availability, temperature fluctuations, humidity fluctuations, and earthquakes. As with man-made stresses, damage caused by natural processes is dependent on the construction methodology. Geologic and cultural conditions also affect the soils and hydrology around a structure.

Earthquakes periodically affect the study area. Coal company seismographs in the Daylight area recorded the ground vibrations of a June 10, 1987, earthquake with amplitudes ranging from 0.13 to 0.44 ips. Earthquake-induced vibrations were recorded at one of the complainant structures in McCutchanville at 0.06 ips on September 26, 1990.

Natural stresses generated by soil and water conditions are the primary causes of damages to structures in the study area. Field observations and analyses pertinent to these mechanisms are summarized as follows:

- Foundation settlements of approximately one inch observed in some houses were enough to cause cracking in basement floor slabs.

- Evidence of soil erosion from around foundations and beneath concrete slabs includes the presence of sediment in underdrains; deposits in basements and crawl spaces; sink holes; and solution features in limestone.

- Evidence of down-slope movement of soils includes foundation settlements and crack patterns on the down-hill side of structures; slicken-sided joints in auger cuttings; and tree growth patterns.

- Drainage control around foundations was often found to be inadequate. Concentrations of moisture decreases the strength of foundation soils and accentuates the effects of soil erosion and lateral soil pressures on basement walls.

- Shrink-swell of shallow soils may have occurred around some structures in the study area. With enough cycles of wet and dry weather and enough swell pressure, this process may have caused distress in basement walls and shallow footings.

- Estimations of (1) lateral earth pressure against an unreinforced masonry block wall and (2) the tensile strength of mortar joints indicate that cracking in at least some basement walls could have resulted from loads exerted by adjacent soils.

- Thermal and humidity fluctuations may have also resulted in sufficient structural movement for cracking to occur.

Construction practices influence the ability of a structure to resist loads. Some construction practices in the study area made damages more likely. Examples include: (1) the use of short dowels for connecting the superstructure to unreinforced masonry wall block; (2) structural additions; and (3) lack of reinforcement in foundations, basement walls, or concrete floors. Distress in joists and beam supports resulted from the use of improper materials or from excessive loading from the superstructure.

The degree to which these processes affect a specific structure depends on a number of factors related to structure type and environmental conditions. For instance, among structures inspected by USGS the houses with relatively simple shapes and crawl spaces had less damage than those with the more

complex shapes and basements. Most of the complainant structures are located along the base of higher topography or along the western (dip) slope of the McCutchanville ridge; and therefore are more prone to drainage problems. Most of structures with basements intercept up-slope ground water and experience basement water intrusion and related damages. Other potential factors pertain to a structure's location relative to upslope water catchment area, septic systems, and artificial water bodies.

Combined Natural and Man-Made Stresses

A combination of forces can damage structures when one force by itself is insufficient. Where foundations are affected by non-vibratory-related forces or natural processes, the superstructure may be significantly prestressed. Mine blasts, earthquakes, or household activity could have resulted in sufficient stress addition for threshold damage to occur, especially in the free response part of the structure. The difference between the magnitude of the vibration-induced stresses and actual material strengths for wallboard and plaster walls would require structural problems that are significant, if not severe. This is true even for the Daylight worst-case scenario. However, the amount of prestress needed in brick veneer for this additive effect to result in cracking may not be as great.

4.2 Conclusions

Several processes unrelated to mine blasting are the primary causes of damage to structures in the study area. These primary processes are related to foundation problems associated with soil movement and water; and construction practices.

Major or minor damages to structures in the study area are not the result of mine blasting. Evidence is also lacking for threshold damages to wallboard and plaster. Cracking in brick veneer mortar joints from mine blasting--which may or may not reach threshold levels of damage--is a small theoretical possibility, based on the results of a conservative model analysis. There is insufficient evidence to conclude that there is a direct link between blasting and mortar cracks in the study area.

Cracking in some structural materials may have resulted from vibratory forces acting on buildings prestressed by other forces. The occurrence of this additive effect on wallboard or plaster should require structural problems that are significant, if not severe. Less prestress may be needed in brick veneer for this additive effect to result in cracking.

Fatigue of building materials from the cumulative effects of mine blasting was not responsible for damages in wallboard, plaster, and brick veneer in the study area.

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