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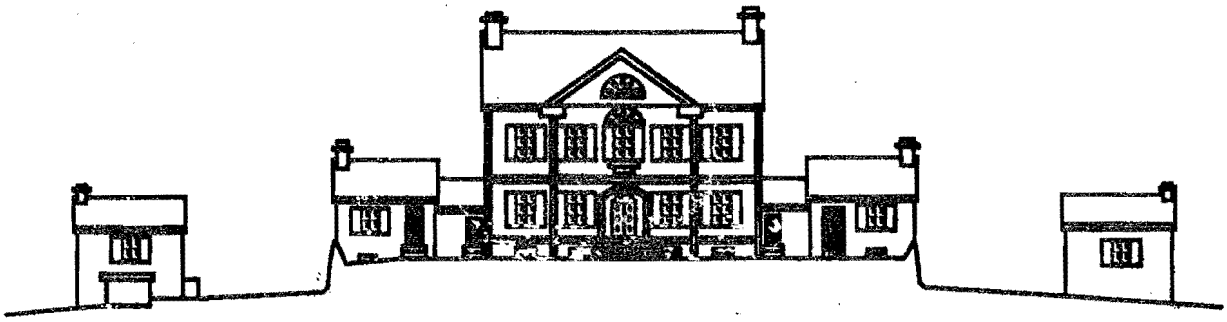

EVALUATION OF POTENTIAL BLASTING-INDUCED DAMAGE TO THE MEASON HOUSE

EXECUTIVE SUMMARY

This report addresses the potential for damage to the Meason House from proposed blasting on property behind the house. Very little in the world is black or white, and the potential for blasting damage to the Meason House is no exception. Blasting and its resulting ground vibrations and airblast constitutes a complex subject involving geology, structural dynamics, and structural strength, none of which are certain, particularly when dealing with a house which is nearing 200 years old.

In an attempt to consider both the salient issues involved in the potential for blasting damage and quantify to some degree the uncertainty inherent in the technical issues, a probabilistic approach was adopted. Drawing upon research data for ground vibrations and airblast, probabilistic models of peak ground velocities and peak airblast overpressures were developed. These models were combined with a structural dynamics analysis of the Meason House and available data on the strength of internal and structural elements such as the plaster and mortar between the limestone blocks to obtain probabilities of failure resulting from a single blast.

The plaster in the Meason House is original as is the mortar between the limestone blocks. The plaster is cracked and at places it is sagging. Particularly for the sagging portions, there are existing stresses which reduce the strength and make it more damage prone than new plaster or plaster which is in good condition. Based on a structural model of plaster ceiling panels and approximations to observed sagging, existing stresses were evaluated. These stresses were compared with failure stresses to estimate remaining strength.

A structural dynamics model was used to determine the natural frequencies of the Meason House and estimate the response to various levels of ground vibration excitation. The model and analysis followed conventional structural dynamics procedures used in earthquake engineering analyses of structures. The computed response was used to estimate the strains in the mortar between the limestone blocks. Two concerns were addressed. The first dealt with either new cracking or the extension of existing cracking in the mortar. The second considered potential sliding of the limestone blocks relative to one another treating the mortar as a frictional material with no cohesive strength.

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Airblast considerations focussed principally on potential window damage as the most sensitive element to airblast overpressures. Comparisons of criteria to other airblast damage such as cracking plaster were made to demonstrate that window damage is the most sensitive element.

Potential damage from subsidence induced by the failure of pillars located to the northeast of the Meason House caused by blasting vibrations was also considered. Existing pillar stresses were estimated based on simplified pillar analyses used in coal mine ground control. It was found that the pillars are in a marginal condition with overburden stresses and strengths about the same. Published values of acceleration thresholds were adjusted to account for the estimated existing stress conditions in the pillars.

Subsidence profile calculations were used to assess the potential effects of subsidence of the Meason House. The same calculational method was used to predict the subsidence profile which resulted in damage to the Cellurale house, located several hundred feet in front of the Meason House. The analyses predicted the damage which occurred which provided a reasonable verification of the validity of the analytical model. The subsidence profile predicted around the Meason House was based on the pillars in question remaining active in strata support and showed no damage as long as the pillars remained in place. Failure of the pillars would result in some outward spread of the subsidence profile which could reach the Meason House.

Regression analyses of peak ground velocity versus scaled distance data were used to develop probability distributions of peak ground velocity at the Meason House. In the case of subsidence, published scaled distance relations for accelerations were used to develop the probability distribution of accelerations at the pillars. The published relation was developed for underground mine blasting and is considered more representative because the proposed blasting will occur in the roof strata of the underground mine. No downward transmission of energy will be required to cause vibrations at the pillar locations.

Probabilities of damage from a single blast were determined by the intersection of peak ground velocities or accelerations required for failure with the appropriate probability distribution. In addition to evaluating the single blast damage probability based on the minimum blasting distance of 1250 feet to the Meason House, damage probabilities for a distance of 1600 feet were evaluated for use in an analysis of the probability of damage from multiple blasts. In the case of subsidence, the minimum distance to the pillars of 720 feet was used along with an average distance of 1,000 feet for the multiple blast probability evaluation. For airblast, all evaluations were made at the minimum distance such that the resulting multiple blast probabilities are

overestimated. However, the low levels of the airblast damage probabilities found indicated that no significant error was introduced through the use of the minimum blast distance.

The effects of multiple blasts on damage probabilities were evaluated based on two phenomena: (1) fatigue, and (2) random trial probabilities. Fatigue was found to not increase the probability of damage significantly because the number of cycles anticipated over the permit life were well below fatigue endurance limits which were available in the literature. The primary multiple blast effect resulted from considering each blast as a random trial of a process having a given probability of damage. The binomial distribution was used to calculate probabilities as a function of the number of blasts. It was assumed that approximately 100 blasts would be conducted during a period of two years when vibration levels at the Meason House would be the controlling factor.

The following table summarizes the probabilities of damage from airblast, ground vibrations and subsidence determined for a single blast and for 100 blasts. The probabilities are stated as decimal fractions which can range from 0.0 to 1.0. A probability of 0.0 means there is no chance of the damage occurring. A probability of 1.0 means it is certain that damage will occur. Very small probabilities are written is scientific notation. For example, the probability of 1×10^{-7} is equivalent to the decimal fraction 0.0000001 and means there is 1 chance in 10 million of the damage occurring.

SUMMARY DAMAGE PROBABILITIES

CAUSE	DAMAGE	PROBABILITY 1 BLAST	PROBABILITY 100 BLASTS
Airblast	Windows	1×10^{-7}	1×10^{-5}
Ground Vibrations	Mortar Cracking	0.15	0.999
Ground Vibrations	Block Sliding	0.006	0.01
Ground Vibrations	Plaster Damage	0.08	0.975
Subsidence	All	0.004	0.02

Proposed additional DER criteria consisting of limiting peak ground velocities to 0.5 inch per second at the Meason House and using a wave superposition method to determine optimum delay

intervals for eliminating low frequency content in the blast waves. The 0.5 inch per second criteria was found to be too high based on a potential damage level of 0.2 inch per second for extending mortar cracks. The wave superposition method was considered a worthwhile endeavor, but difficulties are anticipated because the natural frequencies of the Meason House, which vary between 10 Hz and 30 Hz depending on modes, are above the range of typical structures where the wave superposition method has been demonstrated to provide mitigation against blasting damage.

In conclusion, it is worthy to note that whenever numbers are presented in a technical report, there is a temptation to consider them to be engraved in concrete as if they were definitive fact instead of the result of numerous assumptions required for their derivation. While the assumptions made to arrive at these probabilities rest on sound engineering principles and various intermediate results of analyses compare favorably with empirical data where available, they remain assumptions. It is hoped that the presentations of the derivation of the probabilities in the above table shed some light on the issues involved as well as the sensitivity of resulting probabilities to assumptions made in the analyses.

The analytical models used to estimate the effects of existing conditions on damage probabilities are consistent with the engineering and scientific principles involved, and generally represent a more rigorous approach than is typically applied to conventional residential structures. The historic nature of the Meason House as well as its different structural characteristics and existing conditions from conventional residential structures was thought to warrant a more detailed evaluation than a simple check of peak ground velocities against regulatory criteria.

Respectfully submitted,

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1.0 INTRODUCTION

This report describes an evaluation of the potential for damage to the Isaac Meason House (Meason House) located in Dunbar Township, Fayette County, Pennsylvania, from blasting associated with surface mining operations to be conducted on land behind the house. The historic nature of the Meason House provided the impetus for this investigation as to potential damage since consequences of damage extend beyond simple economics associated with repair or replacement.

1.1 APPROACH

The historic value of the Meason House is not the subject of this report and has been left to others to consider in deciding on the cost-benefit tradeoffs between economic enterprise and historic preservation. However, the historic and architectural nature of the house do create special circumstances in the evaluation of potential damage. These circumstances dictate that such an evaluation should go beyond simple criteria designed for the protection of structures adjacent to mine blasting operations, criteria which are geared to providing optimal protection for average structures for which economic damage can be recompensed.

At the core of the assessment of damage potential lies uncertainty. Blasting associated with surface mining activities involves the response of the earth to a man-made event. Both involve uncertainty. The response of the earth is determined by the geologic structure of the area combined with any modifications made to that geologic structure by man's activities, such as underground mining. Blasting is also the subject of uncertainty relative to blast design parameters such as burden, spacing, delay intervals, and stemming, all of which are factors in determining the response of the earth and all of which are subject to uncertainty in operations. Measurements made of the earth's response at the same location with the same geology to the same blast parameters have shown variability such that blasting operations and the prediction of their potential impact on surrounding structures is, at best, an empirical science subject to reasonable uncertainty. In addition, there is uncertainty in the interaction between the earth's response to blasting, the structure's ensuing response, and whether the structure's response is sufficiently great to cause damage.

As a result of the uncertainty, the potential damage to any structure becomes probabilistic in nature. It is not black or white. The best that can be achieved is an estimate of the probability of

damage in a given situation. Consequently, a probabilistic approach was selected for the evaluation of the damage potential to the Meason House.

One of the difficulties with using a probabilistic approach is that an individual's reaction to a given level of probability of damage is highly subjective. On one side, the owners of the Meason House may view a given probability as unacceptable while others may consider the same probability acceptable. Recognizing the subjectivity involved in interpreting probability levels, the goal of this investigation was to identify the uncertain elements and estimate the probability of damage to the Meason House based on the body of knowledge existing relative to blast effects on structures. No attempt has been made to interpret the resulting probabilities as high or low. This interpretation has been left to others charged with evaluating the tradeoffs between damage to the Meason House and the mining activities.

1.2 CONDUCT OF THE INVESTIGATION

The investigation of potential damage to the Meason House was performed based on the following elements:

- A visit to the offices of the Department of Environmental Resources (DER) in Greensburg, Pennsylvania to review the blasting plan filed with the permit application which resulted in further information relative to DER plans associated with monitoring and controlling blasting operations.

- A visit to the Meason House to review the structural and internal condition, although no attempt was made to perform a pre-blast survey, and

- Engineering analyses based on published literature dealing with blasting and its effects on structures.

- Observation of a five-pound blast conducted on January 29, 1989, with vibration monitoring by the DER and other consultants.

1.3 PROBLEMS WITH THE INVESTIGATION

Generally the investigation was conducted without major problems and full cooperation was provided by the DER and the Kriss family who own the Meason House.

One problem did arise which affects the presentation in this report. During conversations with the DER, it became apparent that the blasting plan as provided with the permit application, a copy of which was provided for this investigation, does not fully describe the methods which will be used for production blasting.

The blasting plan provided with the permit simply calls for a charge weight per delay of 125 pounds with no definition of the delay interval. Discussion with Mr. Fred Ulishni of the DER indicated that in addition to the limitation regarding charge weight that a program of monitoring at least one test hole, and the determination of delay times to reduce peaks and low frequency content in resulting ground vibrations would be implemented. Subsequent sections will address the program in greater detail. The problem created for this presentation arises from the fact that nothing has been documented regarding the planned program. The DER plans to finalize the program pending the outcome of the decision relative to blasting.

To address this problem the results of the investigation are presented based on the formal information provided with the blasting plan. In addition, the planned program as understood based on conversations with Mr. Ulishni is discussed along with its potential mitigating measures; however, no attempt was made to determine any reductions in probabilities of damage which might result because no details are available.

2.0 PRESENT CONDITION

The Meason House is located approximately four and one-half miles south of Connellsville, Pennsylvania. It was constructed by Isaac Meason in 1802. Access is gained by means of a private road from U.S. Route 119. The house and grounds consist of the main house with a wing on each side plus two additional outbuildings in line with the main axis of the house. Figure 1 shows the house and grounds.

The house is constructed of limestone and mortar and has a full basement which exposes part of the foundation walls plus three floors. Figure 2 shows a front elevation of the house and Figures 3 through 6 show floor plans of each of the floors.

The basement consists of the outer walls of the foundation plus two inner walls which run from front to back and are located beneath the interior stud walls of the first and upper floors which form the center hallway. Between the two inner foundation walls, lateral support is provided by a wooden beam of approximate dimensions 13" wide by 14" deep which spans between the two inner foundation walls at the approximate location of the interior stud walls on the first and second floors.

The foundation and outer walls of the house are constructed of an inner and outer course of limestone block with rubble between. The total wall thickness is approximately 18". Flooring for the first floor is supported on wood beams which span from front to back and are approximately 10-12" deep and 3-4" thick. The wood beams are keyed into the inner course of limestone block. While the structural support for the second and third floors was not readily visible, beams were observed behind the wood lathe where plaster has fallen so that it is presumed that the second and third floors are supported similarly to the first floor with beams keyed into the inner course of limestone block.

While the connections of the interior stud walls to the outer limestone walls were not observable, the interior walls spanning from front to back of the house which form the center hallways are reported as being butt connected to the outer walls according to the owner of the house. It is speculated by the owner that the second floor at one time consisted of only two large rooms on each side of the center hall with the dividing walls added at a later time. This speculation is apparently based on observation of 2"x 4" stud construction which was not characteristic of

construction when the house was built. While this explanation of the 2"x 4" construction may be correct, it is also possible that the lateral interior walls on the second floor were replaced at some time during the life of the house.

The roof of the main house, the wings, and the outbuilding is of gabled construction. The roof trusses are of wood and are connected at the apex using the mortise and tendon technique with a wooden pin. The roof was replaced in the 1950's and is made of oak with asphalt shingles according to the present owners.

2.1 STRUCTURAL CONDITION

While the purpose of the visit was primarily to observe the construction of the house, the structural condition was observed for purposes of helping with the assessment of damage potential. No attempt was made to observe or document all existing damage as would be required for a pre-blast survey.

Generally the house is in reasonably good repair as far as structural aspects are concerned. To a casual visitor, the combination of peeling paint and loosened or falling plaster could create an impression that the house is in poor condition. However, most of the readily observable damage is essentially superficial while the principal structural elements appear to be in reasonable condition.

2.1.1 Foundation

The foundation walls are constructed of limestone block and mortar. In places the mortar has deteriorated such that the foundation walls are uncemented. In areas of the south wing, the owners of the house have repointed the block. While the deterioration of the mortar obviously weakens the walls, it is not as severe as might be imagined due to the overlying weight on the walls and the interlocking asperites[1] of the relatively coarsely cut limestone block forming the foundation walls.

The 13"x 14" wooden support beam spanning between the two front-to-back foundation walls, which forms the primary lateral support for the upper floor interior walls, shows signs of sagging. Horizontal cracks were observed near the center of the span which appear to be delamination type cracks parallel to the wood grain and are characteristic of the shear stress

[1] Asperities are small, jagged contours along rock interfaces. In rock mechanics they are responsible for higher values of peak strength compared with residual strength because they create an interlocking which increases the shear resistance along joints. If joints such as the mortar between the limestone blocks are subjected to relatively low stress, the asperities add to the resistance. If a higher stress is applied, the asperities will fail, decreasing the strength.

distribution across the beam arising from flexural loading. As these cracks have appeared through time, the stiffness of the beam has decreased with a resulting increase in deflection (sag).

The sagging of the foundation lateral support beam is confirmed by observations of the first and second floor. With minor exception, space above doors shows a larger gap toward the outer walls than the inner. This would occur if the transverse interior stud walls have deflected downward somewhat due to the sagging of the main support beam. Cracks in the plaster which appear to be reasonably wide are located in the front to back interior stud walls near the connections to the outer walls. This is again suggestive that these walls have deflected downward at the center causing a tension at the butt joint with the outer walls.

2.1.2 Shear Wall Construction

The outer limestone walls of the house are the primary load carrying elements. Their construction is such that they provide shear resistance to horizontal loading in addition to supporting the vertical loading. The shear resistance of the walls can be expected to provide a considerably greater horizontal stiffness compared with modern frame construction. This is important in estimating the natural frequencies of the house which play a major role in determining how the house will respond to ground vibrations or airblast resulting from blasting. Subsequent sections will address the frequency aspects of the house relative to typical frequency content of blasting ground vibrations in greater detail.

Generally, the interior of the limestone walls was not observable because of the plaster covering. At a few places such as above the north window in the first floor room which is labelled as the Study on Figure 2, the interior wall was visible. At that location, which is above the window, a deteriorated condition was observed for the inner course of limestone. From the outside, the limestone and mortar do not show obvious signs of major deterioration.

While the condition noted above the window in the Study may be indicative of localized problems around and over the windows, it does not signify that the structural walls are in a state of serious disrepair. Generally, the walls appear to be competent and should perform fairly well as intended at the time the house was designed and constructed. However, the existing cracks in the mortar at various locations raises a potential for greater damage susceptibility in the form of continued crack propagation.

2.2 INTERNAL CONDITION

As noted previously the internal condition of the house appears worse than it is. Superficial damage exists in the form of cracks in plaster and peeling paint. While the cracks in the plaster increase the susceptibility to further damage because of stress concentration effects, they are not severe relative to the overall condition to the house. The following subsections briefly discuss the internal condition relative to observed damage and its potential effects on the damage susceptibility to blast vibration damage.

2.2.1 Plaster and Wood Lathe

The plaster on the ceilings of the house was constructed over oak wood lathe and is about one inch thick. In some locations the original plaster has fallen. On the third floor, the condition of the plaster was seriously deteriorated, possibly due to water damage through time, and was removed by the present owners. All of the plaster on the first and second floors is original.

In some areas the ceiling plaster can be observed to be sagging as the plaster has separated from the lathe. In other areas sagging cannot be observed directly, but the possibility of separation from the lathe exists for all of the original plaster. The sagging of plaster results in the plaster becoming self supporting as a plate over the area where it has separated from the lathe. This produces stresses in the plaster which decrease its strength and make it more susceptible to blast vibration damage compared with competent plaster on wood lathe. Subsequent sections consider the reduced strength in greater detail.

2.2.2 Plaster on External Walls

On the external walls of the house, the plaster is applied directly to the limestone block. Thin cracks are observable in this plaster, apparently the result of stress cracks in the limestone mortar propagating through the external walls. In the area above the window in the Study, the plaster has separated from the wall and is missing, exposing the inner course of limestone block.

Two forms of damage are conceivable for the plaster on the external walls:

- Crack initiation or propagation due to any crack propagation in the mortar of the limestone walls, and

- Separation of the plaster from the limestone allowing it to fall.

The susceptibility of the first type of damage to blasting vibrations is largely the same as for damage to the mortar between the limestone blocks as cracks are initiated or existing cracks propagate. The susceptibility to the second type of damage is more difficult to evaluate because no information is available relative to the strength of the bond between the plaster and the limestone block. Generally the bond between plaster and limestone block should be good because the plaster fills the small depressions in the block and between the blocks. However, decay over time coupled with stresses which have obviously existed in order to produce the observed cracks may have weakened the bond. Without additional information, it appears impossible to judge the susceptibility of the plaster on the external walls to separation damage beyond noting that it exists.

2.2.3 Internal Stud Walls

The plaster on the internal stud walls is applied over lathe fastened to the stud work. Generally they exhibit two types of damage:

- Hairline cracks, and

- Major cracks which have opened a measurable amount near the butt connections of the stud walls to the outer walls .

The hairline cracks are probably the result of deformations of the walls through time and are associated with the major cracks as part of the same phenomena. The major cracks near the connections to the butt walls appear to be the result of the sagging of the primary center support beam as discussed previously. The cracks are vertical and located either in or close to the connections of the stud walls with the outer walls. The sagging of the support beam would have allowed a downward deflection near the front-to-back center of the house causing tension on the butt joints with the external walls. The severity of the cracks may indicate that the butt connections have been weakened or failed to some degree allowing the stud walls to pull away from the outer walls, forming the cracks in the process.

The susceptibility to damage from blasting of the plaster on the internal stud walls is probably between that of the outer walls and the ceiling plaster. The existing cracks are a source of stress concentration which could result in continued propagation due to blasting damage. However, the vertical nature of the wall plaster is such that the potential for separation from the lathe over time does not appear as great as for the ceiling plaster. With the provision that the existing cracks are a source of stress concentrations which increases the susceptibility to continued propagation of

existing cracks, the plaster on the internal stud walls can probably be assumed about as susceptible to damage as the plaster in the structures used for the analysis of damage by the U. S. Bureau of Mines.[2]

The potential weakening or failure of the connections of the internal stud walls to the external walls apparently caused by the sagging support beam introduces a damage susceptibility in the form of damage to the internal stud walls directly. Generally, this would be classified as major damage in the U. S. Bureau of Mines study since it would involve the structure to some degree. However, stresses induced in the stud walls by the sagging support beam would decrease the strength of the internal walls such that correlating such damage with the classification of minor damage may be more appropriate.

Depends on Freq and How much Amplification

[2] Siskind, D. E., et. al., **Structure Response and Damage Produced by Ground Vibration from Surface Mine Blasting**, U. S. Bureau of Mines, RI 8507, 1980

3.0 TOPOGRAPHIC AND GEOLOGICAL SETTING

The data which forms the basis for estimating probabilities of damage to the Meason House from blasting is to some degree generic, drawn from many sites having many different topographic and geologic features. To evaluate whether there are any unique topographic or geologic features of the Meason House and the mine site where blasting is proposed, the topography and geology were reviewed.

3.1 TOPOGRAPHY

Figure 8 shows a topographic map of the area surrounding the Meason House. The map was produced by computer scanning the bond map filed with the permit application for the mining activities. The basic scanned map was then modified to highlight important aspects. This map then served as the basis for subsequent maps produced. Figure 9 shows a section drawn through the line noted as A-A on Figure 8. Figure 9 provides a better perspective on the surface topography than the contour map and illustrates the underlying geologic formations as well.

Surface topography has been identified as a potential source of amplification of stress waves propagating through the earth.[3] Of particular interest was possible amplification of ground motion caused by the knob on which the Meason House is located. As shown in the section, the slope of the hill forming the knob are shallow such that amplification of ground motion was considered to be negligible. Consequently, no unique circumstances of surface topography were observed which would invalidate the use of the general statistical data relative to peak ground velocities resulting from blasting.

3.2 GEOLOGY

Geologic formations underlying the Meason House act as the media for the transmission of blast waves between the mine site and the house. The presence of underlying stratigraphy having high compression and shear wave velocity contrasts creates the possibility of wave reflections which can increase peak ground velocities compared with statistical data. Consequently, the geology was reviewed to assess whether any such stratigraphic layers may exist.

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[3] D'Appolonia Consulting Engineers, **Seismic Input and Soil-Structure Interaction**, NUREG-CR/0693, United States Nuclear Regulatory Commission, 1979, Section 3.1.

3.2.1 Geologic Formations

The section shown on Figure 8 illustrates the two basic geologic formations underlying the Meason House:

- The Dunkard formation, and the underlying
- Monogahela formation.

The contact between the Dunkard formation outcrops some distance behind the Meason House, between the house and the mine site.

The Dunkard formation is immediately below the Meason House. It consists predominantly of sandy shales with coarse sandstone and thin limestone strata. Wave propagation velocities for these materials are essentially similar such that no strong reflective layer is expected.

Underlying the Dunkard formation is the Monogahela formation. It consists of alternating layers of sandstones, shale, limestone and coal. Its thickness is on the order of 350 feet. It is marked regionally by Waynesburg coal at the top (not present at site) and Pittsburgh coal at the bottom, which is 8-10 feet thick, and alternating shale, sandstone, and limestone layers. Between the top of the formation and the Pittsburgh coal are the Redstone and Sewickley coal layers, both considerably thinner than the Pittsburgh coal.

3.2.2 Pittsburgh Coal Seam

The Pittsburgh coal at the base of the Monogahela formation appears to be the primary objective of the strip mining activities behind the Meason House. Figure 9 shows the contours of the Pittsburgh coal underlying the Meason House superimposed on the topographic map.

The Pittsburgh coal has been the object of previous mining activities in the area using both underground and surface mines. Of particular interest to the Meason House is the Mt. Braddock Mine underlying the house. Figure 10 illustrates the mining activities conducted by U. S. Steel Corporation until 1978 plus additional mining that occurred during the period from 1983 through 1984.

The presence of underground mining surrounding the Meason House poses a complicating circumstance in evaluating the probability of damage from proposed blasting activities as will be discussed at greater length in subsequent sections.

3.2.3 Overburden

Of principal interest is the Pittsburgh sandstone which at places lies above shale layers overlying the Pittsburgh coal and at others is directly over the Pittsburgh coal. Erosion preceded deposition of sandstone and removed some shales so that the sandstone is directly overlying coal at some places. At other places the sandstone does not exist. Figure 11 shows a topographic map illustrating the location of borings which were made relative to the permit application. According to the boring logs the Pittsburgh sandstone forms a principal part of the overburden at some locations for the proposed mine and is in large part responsible for the proposed blasting.

One geologic feature of importance arises from the dip of the geologic formations as illustrated on Figure 8. Blasting in the overburden associated with the strip mining activities will be in the same geologic strata as the roof of the Mt. Braddock underground mine. This arises from the fact that the strata overlying the Pittsburgh seam at the surface mine location are the same strata overlying the Pittsburgh seam in the underground mine. The significance of this fact will be discussed subsequently relative to the possibility of damage to the Meason House arising from additional subsidence in the underground mine and the potential for such subsidence to be aggravated by blasting vibrations.

4.0 INTERACTION OF BLASTING WITH THE MEASON HOUSE

Three potential mechanisms were identified by which the effects of the proposed blasting could conceivably interact with the Meason House and potentially cause damage:

- Airblast overpressures,
- Direct ground vibrations, and
- Potential interaction of ground vibrations with additional subsidence in the Mt. Braddock underground mine.

Subsequent sections discuss each of these mechanisms in greater detail.

4.1 AIR BLAST OVERPRESSURES

The detonation of an explosive used in surface mine blasting gives rise to a shock wave which propagates through the air at the velocity of sound. Typical airblast overpressure is measured in units of decibels, abbreviated dB, which are defined mathematically in terms of the actual air pressure in the shock wave as;

$$dB = 20 \text{ Log } (p/p_0)$$

where;

p is the airblast overpressure in any units of pressure such as psi,
p_0 is a standard reference pressure in the same units of pressure as p, and
Log is the logarithm to the base 10.

The use of decibels for measuring airblast overpressure can lead to misinterpretation of ratios of airblast pressures because of the logarithmic function. For example, for a pressure ratio of 2, the decibel level only increases by 20 Log (2) = 6 dB. Thus, an increase in overpressure by only 6 dB results in a doubling of the actual pressure magnitude. Similarly a decrease by 6 dB results in one-half the actual pressure magnitude.

When airblast pressures arrive at a structure, the structure is subjected to an impulsive pressure loading which rises and falls rapidly as the wave passes. The sudden nature of airblast loading can, depending on the magnitude, cause immediate failure of some portion of the structure such as window panes, or it can provide a dynamic loading which causes the structure and its elements to vibrate. It is this latter effect which is responsible for the observation of rattling when structures are subjected to airblast loadings.

Airblast pressures are also noise which is one of the bases for using decibels as a measuring unit for overpressure. While the noise may be inaudible if the frequency content is low and outside the range of human hearing, it nonetheless acts on the human ear with a potential for injury if decibel levels are sufficiently high.

4.2 DIRECT GROUND VIBRATION

Direct ground vibrations are a result of stress waves created by the blast which propagate outward from the explosive source as speed which depend on the specific nature of the wave and the stiffness and mass properties of the material through which it travels. Figure 12 illustrates conceptually the various stress waves created in the earth by the blast detonation.

The stress waves propagating outward from the explosive source give rise to the motion of particles as the wave passes. It is important to recognize that it is not the earth which is propagating with a wave. The motion of particles is a result of the rise and fall of stresses within the earth as the wave passes. This is analogous to water waves which can be visualized as ripples on a pond. The wave propagates outward and the particles of water at any location appear to rise and fall as the wave passes, but the water at any given location does not travel with the wave.

The motion of particles of the earth as a blast wave passes cause a vibratory motion. It is this vibratory motion which is the source of excitation of structures. Vibratory motion arising from blast waves is measured in terms of the peak particle velocity which is the maximum value of the velocity of the particle motion as the wave passes.

4.2.1 Surface Waves

Part of the energy released by a blast creates stress waves which propagate along the surface of the ground in the upper strata. These waves are known as Rayleigh waves and involve elliptical particle motion in a vertical plane. It is as if the particle of soil moves around an ellipse as the wave

passes. The nature of particle motion for Rayleigh waves is such that they create both radial and vertical particle velocities. Rayleigh waves should be generated by blasting at the mine site.

4.2.2 Body Waves

In addition to surface waves, a blast detonation gives rise to body waves which propagate outward from the blast into the earth in all directions. As shown on Figure 12 there are two forms of body waves, a compressional wave (P-wave) and a shear wave (S-Wave). In addition, the shear wave has two components, one polarized in the vertical plane (SV-wave) and one polarized in the horizontal plane (SH-wave).

P-waves propagate at a velocity which is faster than shear waves by about 40 percent in most rock types. The difference in propagation velocity causes a delay between the P-wave arrival at a given point and the S-wave arrivals. The delay increases as the distance between the blast source and the point of interest increases and has the effect of increasing the duration of ground shaking. At the Meason House, the delay between the P-wave and S-wave arrivals is estimated to be on the order of 75 milliseconds[4]. For blast vibration durations from a single delay on the order of a second, the delay of 75 milliseconds is not expected to have a significant effect on the duration of shaking.

A second consideration relative to body waves is that they reflect at surfaces where there is a strong contrast in wave propagation velocities. Waves propagating downward from the blast source can be reflected and return to the surface. As discussed previously, the geology of the mine site and beneath the Meason House is such that the geological strata do not show major velocity contrasts. However, the presence of the Mt Braddock underground mine raises a question concerning possible wave reflections from the roof of the mine.

The Mt. Braddock mine was retreat mined. The roof of the mine was allowed to fall in a planned and controlled manner. Consequently, the mine is not a series of open rooms. Instead, the roof has collapsed into the opening created by removing the coal, filling the opening with broken rock known as "gob". With time the gob consolidates, but until consolidation is complete, the gob present a potential velocity contrast with the upper shales, sandstones and limestones such that reflection of body waves from the roof of the Mt. Braddock mine is at least a possibility.

[4] See Appendix A which presents a dynamic structural analysis of the Meason House.

4.3 STRUCTURAL DYNAMICS ASPECTS OF THE MEASON HOUSE

The response of the Meason House and the potential for damage from blast generated ground vibrations depend on the relationships of the characteristics of the dynamic characteristics of the house to the time dependence of the ground vibrations. While the peak ground velocity is the parameter used for damage correlation studies, it is not the sole determining factor in how a given structure will respond to blast generated ground vibrations. Similarly, the structural characteristics also determine how a structure will respond to airblast waves created by blasting.

The dynamic characteristics which determine the response are expressed in terms of properties of the house known as natural frequencies and modes shapes plus a third characteristic known as damping. All three are properties of the house itself and do not depend on the loading or excitation.

4.3.1 Natural Frequencies

The natural frequencies of a structure may be viewed most simply by means of a simple spring fixed at one end with a weight at the other end. If the weight is displaced from its rest position, it will vibrate. The frequency, or number of times per second the weight makes a complete cycle, returning to its starting position is the natural frequency of the system. Natural frequency is expressed as cycles per second or Hertz, abbreviated Hz. For this simple system the natural frequency is determined by the stiffness of the spring and the mass of the weight where mass is obtained from weight by dividing by the gravitational acceleration constant, 32.2 ft/second squared in the lb-feet-second system. As the stiffness of the spring increases with the mass held constant, the natural frequency increases. As the mass increases with the stiffness of the spring held constant, the frequency decreases.

The situation is similar to the above simple spring-mass system for structures except that the mass or weight of the structure as well as the stiffness is distributed throughout the structure instead of being lumped at the end of a spring. Because of the distribution of mass and stiffness structures have more than one natural frequency. Theoretically, there are an infinite number of natural frequencies for a structure having distributed mass and stiffness, but on a practical basis rarely more than the first few lowest natural frequencies are important. When a structure has more than one natural frequency, the lowest is typically referred to as the fundamental frequency. The term fundamental has its roots in sound where natural frequencies of many sound producing systems such as strings in pianos or violins are integer multiples of the lowest or fundamental frequencies and the higher frequencies are typically called harmonics. For structures involving

bending deflections of structural elements, higher natural frequencies are not integer multiples of the lowest or fundamental frequency.

4.3.2 Mode Shapes

Modes shapes correspond with natural frequencies and are the deformed shape a structure takes when vibrating at one of its natural frequencies. For the simple spring-mass system the mode shape is the simple back and forth motion of the mass as it vibrates. For more complex structural systems, the modes shapes are more complex. The association of natural frequencies and mode shapes is such that for every natural frequency there is one and only one mode shape, although a structure may respond to ground vibration excitation in more than a single mode.

It is the mode shapes which give rise to the structural distortions and produce stresses as the structure is vibrating. Much of the research work performed to investigate damage from blasting vibrations correlates damage with peak ground velocities. This correlation is one of mathematical convenience to researchers through time, but is not the direct cause of damage. It is the dynamic stresses and displacements arising from the mode shape distortions which give rise to damage. Peak ground velocity is a measure of the intensity of the ground shaking which excites the structural mode shapes, but it is not the direct cause of damage. The importance of this distinction will become more apparent in subsequent sections dealing with potential response and damage.

4.3.3 Resonance and Damping

When a simple spring-mass system is excited by means of an oscillating force, the response is greatest when the frequency of excitation coincides exactly with the natural frequency. At frequencies higher or lower that the natural frequency the response is less. The phenomenon in which the frequency of excitation coincides with a natural frequency of a structure is known as resonance.

Mathematically the response of a simple spring-mass system at resonance is infinite. However, in the real world as opposed to the mathematical one, energy is dissipated as vibrations occur. The dissipation is typically in the form of friction and may result from air resistance or other resistance to motion. The dissipation of energy in a vibrating structure is known as damping. One effect of damping is to reduce the response of a structure at resonance. As the damping increases, the response at resonance decreases.

The stress waves produced by blasting have an oscillatory nature and may be thought of as the sum of a number of oscillations at various frequencies. Mathematical methods are available to determine the frequency content of a blast wave. This frequency content plays a crucial role in determining the structural response because of the possibility of resonance with the natural frequencies of a structure. The importance of the frequency aspect is demonstrated by the observation that if two waves having the same peak particle velocity but different frequency contents pass beneath a structure, the structure may respond violently to one and essentially not at all to the other even though the ground velocities are identical. The reason for the difference is dependent upon the frequency content of the wave relative to the natural frequencies of the structure. The more they coincide, the greater will be the response of the structure.

Appendix A presents an analysis of the natural frequencies, mode shapes, and dynamic response of the Meason house to ground vibrations and airblast. Of necessity, it involves technical detail required for the analyses. Appendix A also presents analyses of various damage elements considered such as crack initiation or propagation in the mortar between the limestone blocks and the sagging plaster. It is intended to serve as a reference for this as well as further discussions.

4.4 UNDERGROUND MINE SUBSIDENCE

The Mt. Braddock underground mine also poses a potential threat of damage to the Meason House. The potential threat arises from the nature of subsidence and its effects on structures. While the threat of subsidence damage exists independently of the proposed blasting, it was considered worthwhile to investigate concerns that blasting vibrations could aggravate the subsidence threat.

Subsidence results from the collapse of the roof rock in underground mines into the void created by mining activities. Over time, the collapse works its way upward to the surface resulting in a depression on the surface over the underground void. The damage potential to structures arising from subsidence results, not from the depression, but from the shape of the subsidence profile, particularly near the edges of the depression. At the edges of subsidence profiles the ground surface is curved. The curvature causes differential settlement of structures which are located in the curved zone. Differential settlement then induces strains in the structure which lead to damage.

Because the retreat mining method was used at the Mt. Braddock underground mine, subsidence was both planned and expected. The retreat mining method removes all coal in the retreat panels and induces roof collapse as a planned part of mining. Therefore, all of the shaded

zones shown on Figure 10 may be assumed to have undergone subsidence during or shortly after the mining of the areas.

As shown on Figure 10, the Meason House is located in an area where the coal was left in place to form a protective pillar. The pillar was extended to the northwest to also provide protection for the structures located in front of the house including the Cellurale house which was damaged by subsidence sometime in 1979-1980.[5] The protective pillar surrounding the Cellurale house proved to be inadequate because the subsidence profile extended outward further than anticipated. The subsidence which resulted in damage to the Cellurale house is discussed at greater length in Section 8.0.

The subsidence threat to the Meason House arises from the pillars left in place in the middle of the retreat area behind the house (See Figure 10). Left in place as indicated, these pillars provide roof support which decreases the span of the gob area directly behind and to the right of the Meason House. While roof collapse and subsequent subsidence over the mined-out area would decrease the stresses acting on the remaining pillars compared to no subsidence, the remaining pillars are overstressed compared with lithologic stresses which would exist if no mining had occurred.[6] Over time the overstressed condition can result in pillar collapse which would extend the width of the subsided zone. That extension on the surface could reach the Meason House causing damage. The potential interaction of pillar failure with blasting vibrations is discussed in Section 8.0.

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[5] Claim No. 638, dated 9/24/80. According to the investigation, damage was primarily in the foundation walls which were pressed inward with horizontal joint cracking about one foot below the ground surface. In addition the foundation wall movement caused distortion to the first floor resulting in minor wall and ceiling plaster cracking. An inground swimming pool located 50 feet south of the house was also damaged.

[6] Peng, S. S., **Coal Mine Ground Control**, Wiley-Interscience, New York, 1978, Section 8.4

5.0 PROPOSED BLASTING PLAN AND CRITERIA

The magnitude of ground vibrations and airblast which can be expected to result from a blast depend on the amount of explosive used per delay. A delay is the time interval between the ignition successive rounds. To qualify as a delay on a regulatory basis, the minimum time is 0.008 seconds or 8 milliseconds. For ground vibrations, the peak ground velocity has been established empirically to be a function of the scaled distance, S_{dv},which is defined as;

$$S_{dv} = D/w^{1/2}$$

where;

D is the distance from the blast in feet, and
w is the charge weight per delay in pounds.

For airblast, the peak airblast pressure in decibels has also been established empirically to be a function of another scaled distance, S_{da}, which is defined as;

$$S_{da} = D/w^{1/3}$$

where D and w are the same as for the velocity scaled distance. Thus, the difference between the scaled distance used for velocity and that used for airblast is that for velocity the actual distance is divided by the square root of the charge weight while for airblast it is divided by the cube root of the charge weight.

To estimate the potential for damage to the Meason House from ground vibrations and airblast, the charge weight per delay is required. Based on the blasting plan filed with the permit application the maximum charge weight per delay was defined as 125 pounds with the nearest structure identified as being at a minimum distance of 650 feet. No information was given as to the delay interval nor the number of blasts anticipated over the permit duration.

As mentioned in the introduction, one of the difficulties encountered in estimating potential damage to the Meason House arose from the lack of specific information concerning how the blasts will actually be conducted. Based on the blasting plan, only a scaled distance approach could be used relative to ground vibration criteria. However, based on discussions with the DER, it appears

that additional measures based on monitoring of vibration and airblast levels will be required of the mine operator. While general information was provided relative to the concepts of the additional requirements to be imposed, no specifics were available.

Section 5.1 presents airblast overpressure criteria based on the Pennsylvania Code. The determination of expected values and whether they comply with the criteria as well the probability of damage to the Meason House are discussed in detail in Section 6.0.

Section 5.2 briefly compares the scaled distance developed from the blasting plan information to the Pennsylvania Code criteria. With no additional requirements by the DER, this would be the governing criteria. Under the criteria for ground vibrations in the state code, the DER can stipulate additional criteria when appropriate. Based on discussions with the DER, it has been stated that additional criteria will be imposed Section 5.3 discusses the proposed additional requirements based on monitoring. In subsequent sections where airblast and ground vibrations are discussed in detail, it was assumed that the blasting operations will be conducted using 125 pounds per delay. The effects of any additional criteria on the results of the evaluation of damage potential are discussed as appropriate.

5.1 AIR BLAST CRITERIA

Airblast limits in decibels are given in 25 Pa. Code §87.127(e). The peak airblast overpressure limits depend on the frequency characteristics of the measuring instrument as shown in Table 1.

TABLE 1
AIRBLAST OVERPRESSURE LIMITS

FREQUENCY RESPONSE OF INSTRUMENT	LIMIT (dB)
0.1 Hz or lower flat response	135
0.2 Hz or lower flat response	132
6 Hz or lower flat response	130
C-weighted slow response	109

Section 6.0 discusses airblast criteria relative to expected values at the Meason House based on scaled distance relationships and the charge weight per delay specified in the blasting plan.

5.2 SCALED DISTANCE PLAN AND CRITERIA

A scaled distance, S_{dv}, of 60 is required by 25 Pa. Code § 87.127 (j) for blasting where ground vibrations will not be monitored. Based on the distance to the nearest structure of 650 feet and a charge weight per delay of 125 pounds, the scaled distance from the blasting plan is;

$$S_{dv} = 650/125^{1/2} = 58.1$$

While this is slightly less than the regulatory limit of 60, it essentially satisfies the criteria.

5.3 VIBRATION MONITORING CRITERIA

In lieu of the scaled distance limit of 60 specified by the regulations, 25 Pa. Code §87.127(h) establishes a one inch per second criteria for peak particle velocity when ground vibrations are monitored. Based on discussions with the DER it appears that a monitored approach will be required with the following additional provisions:

- Seismographs will be placed at the Meason House, the Connellsville school, and on the banks of the creek which flows through the mine site. Figure 13 shows the approximate proposed locations for seismographs.

- The maximum peak particle velocity at the Meason House, the Connellsville school and the creek bank will be limited to 0.5 inches per second.

- A test hole will be required and the monitored ground vibrations from the test hole will be analysed using a waveform superposition technique to establish delay intervals which will reduce the frequency content of the blast vibrations for frequencies below approximately 10 Hz.

- The delay interval established by the waveform superposition technique will be used for blasting operations with continued monitoring.

The concept of the waveform superposition technique is to adjust the delay interval such that the ground vibration velocities from succeeding delays cancel portions of the velocities from the preceding delay when superimposed. The technique will be discussed subsequently in greater detail in Section 7.0 relative to its potential mitigating effects on the possibility of damage.

6.0 AIR BLAST OVERPRESSURES

This section discusses airblast overpressures expected to be created by the proposed blasting. Regression analyses[7] were performed to determine overpressure as a function of the cube root scaled distance, S_{da}, defined above, based on data from highwall blasting in coal mines. The statistics of the regression analyses also permitted the evaluation of the probability distribution for airblast levels at the Meason House. Data giving the probability of damage for various overpressures levels was then used to determine an overall probability of damage from a single blast of 1×10^{-7} (one chance in ten million). The effects of multiple blasts anticipated over the life mining activities when they are closest to the Meason House was evaluated from the damage probability for a single blast to give an overall probability of damage from airblast over the duration of mining of approximately 1×10^{-5} (one chance in 100,000) based on a total of 100 blasts. Curves are presented for the probability of damage due to airblast for a fewer number of blasts ranging from 1 to 100.

6.1 CORRELATION OF PEAK OVERPRESSURE WITH SCALED DISTANCE

Data from RI 8485[8] giving measured airblast overpressures for various charge weights per delay and distances were sorted to remove all data except those pertaining to coal mine highwall blasts. This was done to remove bias in the data and ensuing statistics created by blasting situations not directly comparable to the proposed blasting.

A regression analysis was performed on the resulting data to obtain peak airblast overpressures as a function of the cube root scaled distance, S_{da}. The measured values for the 0.1 Hz. peak linear responses given in RI 8485 were chosen as the basis for the regression analyses because more data points existed for that measuring technique. Also, damage probabilities are most readily compared using the 0.1 Hz peak linear response data.

The regression analysis yielded a plot of peak airblast overpressures as a function of scaled distance. The plot is a straight line on a log-log scale (logarithm of velocity vs.logarithm of scaled distance) as shown on Figure 14. The mean regression line, the mean plus one standard deviation

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[7] Regression analysis is a mathematical technique for determining relationships between two or more variables. In the present case they were used to determine the best-fit straight line through data points having considerable scatter.
[8] Siskind, D. E., et. al., **Structure Response and Damage Produced by Airblast from Surface Mining,** U. S. Bureau of Mines, RI 8485, 1980.

line, and the mean plus three standard deviations line are shown on the figure. Appendix B discusses the regression analysis in more detail and presents the regression statistics plus the regression equation for the mean value line shown on Figure 14.

6.2 SINGLE BLAST CRITERIA AND DAMAGE PROBABILITY

Figure 15 shows a map of the Meason House and the mining panels anticipated based on information obtained from the DER.[9] The nearest and farthest panels were identified on the map and distances to the Meason House as shown on the figure were scaled. The minimum and maximum distances are:

- Minimum Distance - 1250 feet
- Maximum Distance - 4050 feet.

In addition, the average distance to the Meason House for blasting behind the house was also scaled as 1600 feet. The distance of 1250 feet constitutes the expected minimum distance at which blasting could occur to the Meason House. The 1600 feet represent the average distance for multiple blasting effects when ground vibrations at the Meason House are expected to control the blasting criteria.

6.2.1 Criteria

Using the distances of 1250 feet and 1600 feet, corresponding cube root scaled distances were calculated based on a charge weight per delay of 125 pounds as specified in the blasting plan as;

$$Sda_{1250} = (1250/125^{1/3}) = 1250/5 = 250$$

$$Sda_{1600} = (1600/125^{1/3}) = 1600/5 = 320.$$

Referring to Figure 14, the peak overpressure corresponding to scaled distances of 250 and 320 are approximately 117 dB and 114 dB, respectively based on the mean curve. These levels are

[9] During a meeting with the DER on 1/9/89, a copy of the most recent bond map was obtained. In addition a map showing mining panels was reviewed but a copy was not obtained. The basis for the panels shown on Figure 15 was a previous bond map obtained from the Meason House owners stamped, "Received April 7, 1988." Based on the map used to prepare Figure 15 and the map viewed in the DER offices on 1/9/89, no significant differences were noted in the mining panel layout.

well below both the Pa. Code criteria of 135 dB and are also below the minimum value cited in RI 8485 of 134 dB which produces damage due to airblast.

The above values were determined from the mean regression line shown on Figure 14 which exhibits a considerable amount of scatter. Assuming the data in distributed according to a lognormal distribution around the mean, probability levels were obtained for various peak overpressure levels based on the statistics of the regression analysis. Figure 16 shows a plot of the probability of exceeding a given level of airblast overpressure which was derived from the assumption of a lognormal distribution. As shown on Figure 16, the probability of exceeding the Pa. Code criteria of 135 dB at the distance of 1250 feet is approximately 0.001 or one chance in a thousand on each blast.

6.2.2 Probability of Damage

The Pa. Code criteria value of 135 dB and the minimum damage value of 134 dB from RI 8485 do not create a black and white situation such that damage will not occur up to the criteria value of 135 dB then always occur once overpressures surpass 135 dB. The minimum damage value is the value at which at least one incident of damage has been observed, but it does not mean that the probability of damage is 100 percent.

Window panes have generally been found to be the most damage sensitive element to airblast overpressures. Table 2 shows the data from Table 12 of RI 8485 which gives airblast overpressures for various structural elements. Table 2 shows multiple entries for the same structural element because of different researchers as reported in RI 8485. Note again that overpressures are given in decibels where an addition of 6 dB implies a doubling of pressure. A value of 140 dB implies twice the pressure as a value of 134 dB.

Values given for plaster are 142 dB, 144 dB, and 148 dB in Table 2. Appendix A presents an analysis of existing stress levels in sagging plaster at the Meason House. It demonstrates that the effect of sagging is to make the plaster damage prone at peak ground velocities which are about one-fourth those of competent plaster because part of the plaster strength is utilized in resisting stresses induced by sagging. On a decibel scale, the ratio 1/4 is equivalent to a decrease of 12 db. Since airblast also produces vibrations which result in damage, the plaster values in Table 2 can be modified to account for the sagging plaster in the Meason House by subtracting 12 dB. This would give a range of 132 dB to 136 dB. The value of 142 dB for new plaster was not modified because

plaster strength increases with time as it ages. Based on these values, the 134 dB criteria based on the RI 8485 is the average.

TABLE 2
AIRBLAST SENSITIVE ELEMENTS
AND OVERPRESSURES FOR DAMAGE

STRUCTURAL ELEMENT	dB
Glass, poorly mounted	151
Glass, poorly mounted	141
Less than 64 sq. ft window area (probability = .001)	136
Glass in general	140
Glass in general	146
Glass (probability=.00001 than 1000 people impacted)	144
3.5 sq. ft window (probability=.0001)	141
Wood frame and concrete walls	143
Panes in greenhouses (0.7% damaged)	140
General plaster	144
General glass	145
Paint fleck falling	134
New plaster	142
General glass	146
39 sq. ft. window	142
General plaster	148
General glass	139
RI 8485 research	134

Using glass as the most sensitive element to airblast, Figure 17 shows curves of probability of damage as a function of airblast levels. Figure 17 was essentially reproduced from Figure 40 of RI 8485. For the criteria level of 135 dB which has a probability of occurrence of 0.001, the probability of damage from Figure 17 is approximately 0.0001 for colonial panes.

The probability of damage for a given airblast overpressure is known as a conditional probability. It is the probability of damage for a given overpressure subject to the condition that the overpressure exists. In the laws of probability, the probability of an event which depends on another conditional event is given by;

$$p(A) = p(A/B) \cdot p(B)$$

where in the present case;

p(A) is the probability of damage from a single blast,

p(B) is the probability of a criteria airblast level of 135 dB determined previously as 0.001, and

p(A/B) is the probability of damage given that a level of 135 dB occurs, or 0.0001 from Figure 17.

Thus the probability of damage from airblast for a single blast is;

$$p(A) = (0.0001)(0.001) = 1 \times 10^{-7}$$

or, about 1 chance in 10 million.

6.3 MULTIPLE BLAST DAMAGE

The probability of damage of 1 chance in 10 million due to airblast is the probability for only one blast. In other words, if only one blast were to be made, the probability of airblast damage to the Meason House would be about 1 in 10 million. However, over the duration of the permit, numerous blasts are expected. They will not all be at a distance of 1250 feet from the Meason House, but will vary from the maximum of 4050 feet to 1250 feet. During the early part of the permit life, mining will be well removed from the Meason House, and will be controlled primary by ground vibration monitoring at the creek bank. During the later stages of mining, the Meason House will constitute the nearest seismograph location such that effects on the Meason House will govern.

The number of blasts and length of time that mining and the proposed blasting will be controlled by the Meason House is unknown. It was assumed that blasting would be controlled by events at the Meason House for a period of two years and that approximately 100 blasts would occur based on one blast per week.

The probability of damage due to repetitive blasting can increase for two reasons:

- Potential cumulative fatigue damage from repetitive blasts, and

- The probability associated with successive random trials of an event which has a probability of resulting in damage.

The first is a result of the behavior of the materials and the second is a result of the random nature of the probability of occurrence for a single blast.

6.3.1 Fatigue

Fatigue is a phenomenon which results in the failure of structure elements at stress levels below their static failure stress when subjected to multiple loading cycles. Since both airblast and ground vibrations produce structure vibrations near or at its lowest resonant frequency, repeated blasting subjects structural elements to repetitive stresses even though levels are below the levels required to cause failure statically.

The subject of fatigue damage from repetitive blasting has only recently been the subject of study.[10] While the effects of sagging on the plaster would reduce the number of cycles for crack propagation from that reported in RI 8896, the results of approximately 56,000 cycles which are equivalent to roughly 28 years of blasting to produce a crack by fatigue, would indicate that fatigue is not a significant threat to potential damage of the Meason House. Based on the assumed 100 blasts, and approximately 10 cycles of peak strains per blast, only 1000 cycles would be experienced. Even if the fatigue life of 56,000 cycles is reduced by 1/4 to 14,000 cycles, the expected 1,000 cycles is well below the fatigue life.

6.3.2 Random Trials

Repeated blasting where each blast has a probability of failure may be considered a repeated random event. When a number of trials of that random event are made and the probability of failure is the same for each trial, the overall probability of failure is governed by a binomial probability distribution. For a probability of failure of 1 in 10 million (1×10^{-7}) for each trial (blast) Figure 16 shows the probability of having at least one failure during one of the repeated trials up to 100 trials, the assumed number of blasts when events at the Meason House would control.

An analogy may be helpful in visualizing the effects of repeated blasting on the overall probability of failure. If a coin is tossed, the probability of getting a head on one toss is 1 in 2 or

[10] Stagg, M. S., et. al., **Effects of Repeated Blasting On a Wood Frame House**, U. S. Bureau of Mines, RI 8896, 1984.

0.5. If the coin is tossed repetitively, the probability increases that at least one head will occur sometime during the repeated tosses. If the coin is tossed a large number of times, it becomes almost a certainty to obtain at least one head.

Figure 18 shows the overall probability of damage due to airblast from repeated blasts where the probability of damage is 1 in 10 million for each blast for 1 to 100 blasts. Using the maximum probability from Figure 18, corresponding to 100 blasts, the overall probability of damage due to airblast is 1×10^{-5} or 1 in 100,000.

The probability of airblast damage from multiple blasts is conservative because the minimum distance was used for each blast. However, the probability is low enough that the additional conservatism has little effect.

7.0 DIRECT GROUND VIBRATIONS

This section discusses the potential for damage to the Meason House from direct ground vibrations resulting from the proposed blasting. Regression analyses were performed on published data to determine the relationship between peak ground velocity and the square-root scaled distance. Statistics of the relationship were also developed from which probability distributions were derived for peak ground velocity at the Meason House.

The assessment of damage probability focussed on two aspects of the house: (1) damage in the form of cracks in the mortar between the limestone blocks, and (2) damage to the internal plaster, particularly falling of the ceiling plaster which has been observed as sagging at places. A structural dynamics analysis was performed to estimate the dynamic response of the house to ground vibration. The analysis resulted in an amplification factor of 1.6 for structural vibrations of the house and was used to predict stresses in the mortar. It was found that a peak ground velocity of 0.2 inch per second results in failure strains in the mortar.[11] Structural analyses of the sagging plaster determined that approximately three-fourths of the strength capacity may be used in resisting sagging stress. This reduced the failure strain in the plaster from 400 microinches per inch to 100 microinches per inch or 50 microinches per inch shear strain which was found to correspond to a particle velocity of 1.0 inch per second.

A statistical analysis of the midwall amplification factors for two-story houses which were in the frequency range of 10 Hz to 30 Hz where the natural frequencies of the Meason House were estimated to lie, provided a probability distribution of amplification factors. This was combined with the probability distribution of peak ground velocity to obtain a distribution for particle velocities internal to the house which was used to determine damage probabilities for the plaster.

Based on the failure strain in the mortar occurring at 0.2 inches per second for additional cracking or extending existing cracks the probability of damage was found to be 0.15 for a single blast at minimum distance and essentially certainty over 100 blasts based on the average blast distance. The probability of small movement of the limestone blocks in the walls of the Meason House due to a single blast was found to be 0.006 based on the minimum distance with a failure

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[11] The German Vibration Standard, DIN 4150 gives a peak ground velocity limit of 0.16 inch per second for buildings with visible damage and cracks in masonry which tends to substantiate the 0.2 inch per second limit determined for the Meason House. It is also noteworth that DIN 4150 sets a peak ground velocity limit of 0.08 inch per second for ruins, and ancient and historic buildings given antiques protection.

ground velocity of 1.0 inch per second. Based on the average distance a probability of 0.01 was found for block movement in 100 blasts. Similar analyses of the failure strain in the plaster and the probability distribution of internal velocities found the probability of plaster damage to be 0.08 or 8 chances in 100 for a single blast. When multiple blast effects on the probability of damage were considered, the probability of plaster damage became 0.975 which is almost certainty.

7.1 PEAK GROUND VELOCITY CORRELATION WITH SCALED DISTANCE

Data from Table 1 of RI 8507 was sorted to eliminate all data that did not pertain directly to coal mine blasting. Regression analyses were performed for peak ground velocities in the horizontal radial, horizontal transverse, and vertical directions as a function of the square-root scaled distance, S_{dv}, defined previously. Figures 19, 20 and 21 show the resulting plots of peak ground velocity as a function of scaled distance for the three components of ground motion.

Regression analysis statistics were used to develop the additional lines shown on Figures 19, 20 and 21 corresponding to one standard deviation and three standard deviations above the mean. These lines incorporate only the standard deviation of the actual data about the mean regression line and do not include additional variation due to standard deviations of the slope or intercept of the mean regression line such that the actual statistical variance would be slightly greater than shown. This could result in inclusion of the point shown on Figure 21 which is slightly outside the three standard deviation limit. Appendix B discusses the regression analyses in greater detail and presents the regression statistics corresponding to each of Figures 19, 20, and 21.

Distances of 1250 feet and 1600 feet and a charge weight per delay of 125 pounds were used in the regression lines given in Appendix B to obtain estimates of the mean peak ground velocities at the Meason House for blasting at those distances. The distance of 1250 feet is the minimum distance from the mining panels to the Meason House as shown on Figure 15. The distance of 1600 feet is an approximate average distance for all blasting in the area where vibration levels at the Meason House are expected to control. The minimum distance was used to calculate the maximum single blast probability level. The average distance was used to calculate multiple blast probabilities. Table 3 shows the mean peak ground velocities at the Meason House based on the both distances.

The regression statistics were used to develop a probability distribution of peak ground velocity for all three components of ground motion. Figure 22 shows plots of the probability

distribution for all three components for distances of 1250 feet and 1600 feet. The underlying probability distribution assumed for Figure 22 was the lognormal distribution.

TABLE 3
MEAN PEAK GROUND VELOCITIES
AT MEASON HOUSE FROM REGRESSION ANALYSES

VELOCITY COMPONENT	V_{1250} (IN./SEC.)	V_{1600} (IN./SEC.)
Horizontal Radial	0.11	0.07
Horizontal Transverse	0.10	0.07
Vertical	0.08	0.06

7.2 FIVE POUND BLASTING AND MONITORED RESULTS

On January 26, 1989, a total of 13 five-pound test holes were shot at the base of the highwall in the panel marked "1" on Figure 15. The blast was monitored with seismographs located at the Meason House, the Connellsville School, and one in the mine pit which was located 80 feet from

TABLE 4[12]
PEAK GROUND VELOCITIES
FROM FIVE-POUND SHOTS

Shot	Distance (Feet)	Charge (lbs)	Sdv ($ft/lb^{1/2}$)	Radial (in/sec)	Trans (in/sec)	Vert (in/sec)
1	80	5	35.8	0.3	0.3	0.4
2	75	5	33.5	0.4	0.4	0.8
3	70	5	31.3	0.6	0.6	1.0
4	65	5	29.1	0.4	0.5	0.9
5	60	5	26.8	0.4	0.4	0.8
6	55	5	24.6	0.4	0.5	1.0
7	50	5	22.4	0.5	0.6	0.8
8	45	5	20.1	0.5	0.5	1.0
9	40	5	17.9	0.4	0.6	0.7
10	35	5	15.7	0.6	0.5	0.9
11	32	5	14.3	1.6	0.6	1.8
12	30	5	13.4	4.0	2.4	3.0
13	32	5	14.3	4.4	4.3	4.0

the blast hole for the first shot. The distance between the hole and the seismograph located in the mine pit varied between 80 feet and 30 feet. Table 4 shows the results for peak ground velocities

[12] Vibra-Tech Engineers, Inc., Seismograph Analysis, Masontown Office, Christopher Resources-Mt. Braddock Mine, 1/28/89, Dated 2/1/89

in the radial, transverse and vertical directions measured in the mine pit. As expected, no vibrations were measured at either the Meason House or the Connellsville school.

Figures 23, 24, and 25 show the peak ground velocities from the five-pound shots plotted on the regression plots of Figures 19, 20, and 21 for the radial, transverse and vertical components of motion, respectively.

Appendix B contains a regression analysis and its statistics of the data obtained from the five-pound shots; however, no attempt was made to test the statistical significance of differences between the regression lines because the amount of data available from the five pound shots was considered too small compared with the RI 8507 database to provide meaningful results. Also, the range of scaled distances involved in the five-pound shots was limited relative to the RI 8507 database.

The plotted points for the five-pound shots shown on Figures 23, 24, and 25 show that the scaled distance peak ground velocities fall within the scatter of the RI 8507 data. Therefore, it was concluded that the five-pound shots are consistent with the larger data base such that the existing regression curves shown on Figures 19, 20, and 21 are valid as a predictor of potential peak ground velocities at the Meason House.

7.3 PROBABILITY OF DAMAGE FROM A SINGLE BLAST

Possible damage to the Meason House from a single blast was viewed in terms of two components:

- The mortar between the limestone blocks comprising the structural walls of the house, and

- The internal plaster, particularly that on the ceiling, some of which is sagging.

These two elements of the Meason House were considered to represent the most damage-prone elements because of their present condition. Structurally, unreinforced masonry walls typically fail first by cracking of the mortar. Only with severe failure strains do the masonry blocks, stone or brick, usually crack. Within the Meason House, the plaster is a damage prone element. In research cited in RI 8507 dealing with blasting damage, rarely are other internal elements considered. Elements such as piping or electrical systems were not considered because

they are not part of the historical aspects of the house; although blast induced failure of such systems is rare. Other possible internal damage considerations might involve original woodwork, some of which is ornate. However, the ductility of wood typically allows considerable straining before failure occurs so that woodwork was not evaluated.

7.3.1 Structural Analysis

The construction and configuration of the Meason House are such that it is difficult to include within the conventional residential structures which have been the subject of blasting damage research.[13] Consequently, a dynamic structural analysis was performed to estimate vibratory response levels and effects on the mortar and plaster.

Appendix A presents the structural analysis and pertinent results. The analysis was conducted using the modal superposition method where by the response of a structure can be represented as the sum of the responses of individual structural modes. Since the blast vibration time histories would occur in the future if blasting is permitted, no time histories were available to perform a modal superposition time history analysis nor to develop acceleration levels associated with peak ground velocities. These two shortcomings are not unusual in earthquake engineering work where the response to future possible earthquakes is sought. The lack of acceleration records was compensated for by assuming a harmonic relationship between the velocity and acceleration, and the lack of time history motions was considered using the standard technique of summing individual modal responses by the square root of the sum of the squares.

The Meason House was modelled mathematically as a two degree of freedom system from which natural frequencies and mode shapes were computed. Individual modal responses were computed for displacements from which total response displacements were computed using the square root of the sum of the squares. The total displacements were then used to compute response velocities based on the harmonic assumption plus forces in the two springs used to mathematically represent the structural stiffness. The spring forces were converted to stresses by dividing by the cross sectional area of the plan view of the structural walls.

[13] On page 58 of RI 8507 the following statement is made as to the results relative to safe vibration levels: "Implicit in these values are assumptions that the structures are cited on a firm foundation, do not exceed two stories, and have the dimensions of typical residences, and that the vibration wave trains are not longer than a few seconds."

7.3.1.1 Structural Response

The response of the structure was used to compute a structural amplification factor which is defined as the ratio of the structural velocity to the peak ground velocity. The results shown in the form of plots in Appendix A yielded a structural amplification factor of 1.6. This is slightly higher than the amplification factors determined from measurements in RI 8507 which have an average around 1.0 by inspection. The higher amplification factor is the result of two phenomena: (1) the structural dynamic characteristics of the Meason House are different from the structures considered in RI 8507, and (2) the natural frequencies of the house occur in the realm where the frequency content of ground vibrations produced by coal mine blasts is greatest. Figure 26 shows a histogram which is typical of the frequency content of coal mine blasts with the range of natural frequencies of the Meason House superimposed.[14]

7.3.1.2 Mortar Strain

The mortar in the Meason House contains existing cracks which enhance the probability of additional cracking. Based on the structural analysis two aspects of mortar damage were considered:

- New or additional cracking of the mortar based on exceeding the elastic strength, and
- Potential movement by sliding of the limestone blocks along existing cracks in the mortar.

New or additional cracking was considered by evaluating the strains in the mortar based on the dynamic response calculated in Appendix A. The structural response analysis showed that structural velocities of approximately 0.2 inches per second produced strain levels in the mortar between the limestone blocks of about 150 microinches/inch which has been cited as the failure strain for crack propagation in mortar.[15]

For movement of the limestone blocks, the mortar was considered to be a friction material with a coefficient of friction of 0.11 based on low value of shear to normal stress as discussed in Appendix A. The normal stress on existing cracks is due to the weight of overlying blocks. Based

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[14] Stagg, M. S. and A. J. Engler, **Measurement of Blast-Induced Ground Vibrations and Seismograph Calibration,** U. S. Bureau of Mines, RI 8506, 1980. Figure 10.
[15] Northwood, T. D. , D. R. Crawford, and A. T. Edward, Blasting Vibrations and Building Damage, **The Engineer, v.215,** No. 5601, May 31, 1963, pp. 973-978, as cited in RI 8507.

on the response analysis it was found that incipient block movement corresponds to a peak ground velocity of about 1.0 inch per second.

7.3.1.3 Plaster Strain

Appendix A also contains an analysis of stresses produced in sagging plaster. The analysis was based on plaster panels of varying spans and computing the maximum stresses and deflections at the center which correspond to the sag of the panel. The computation of stresses and deflections was performed assuming a panel can be mathematically modelled as a plate of uniform thickness subject to a uniform loading over its entire area caused by the weight of the plaster. Appendix A gives plots of maximum stresses and deflections for plates in size ranging from 1 to 10 feet for ratios of the sides of 1 and 2 for a one-inch thick plate.

Combining the maximum stresses and deflections as a function of plate dimensions, maximum stresses were calculated as a function of maximum deflections. From this relationship, the stresses in existing sagging plaster were approximated based on a 1/8-inch sag which was approximated from observations. This corresponds to a stress of approximately 225 psi.

Based on failure stresses reported as approximately 300 psi with accompanying strains of 375 microinches/inch,[16] the existing sagging plaster is stressed to approximately 3/4 of its failure stress. This reduces the margin for additional stresses caused by blasting vibrations to 1/4 of its unstressed strength or 75 psi. Reducing the unstressed failure strain proportionately, the failure strain for existing plaster was taken to be 100 micro/inches per inch which corresponds to a shear strain of 50 microinches per inch.

7.3.2 Probability of Damage

Probabilities of damage for the mortar and plaster of the Meason House were determined based on the particle velocity requirements to produce the failure strains based on stress concentration effects and existing strain levels as determined above. Probability distributions of particle velocity, either of the ground for the mortar between the limestone blocks or internal velocities in the case of the plaster were used to determine probabilities of velocity levels sufficient or greater than those required to cause failure.

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[16] Leigh, B. R., **Lifetime Concept of Plaster Panels Subjected to Sonic Boom**, University of Toronto, Canada, UTIAS-TN-191, July, 1947, as cited in Table 7 of RI 8507.

7.3.2.1 Mortar Damage

Based on a peak ground velocity of 0.2 inches per second, the probability of additional cracking of the Meason House mortar is 0.15 or approximately 1 chance in 7 for blasting at the nearest distance. This compares to a probability of about 0.01 based on the probability studies performed in RI 8507 for conventional structures.[17] One reason for the difference is that the Meason House is considerably stiffer than conventional brick walls of the few structures in the RI 8507 test data which are brick. One reason for the greater stiffness is the stiffness contrast between the mortar and the limestone combined with the difference in size. Very small strains in the composite produce very large strains in the mortar.

Also shown on Figure 22 is the average probability of mortar cracking from a single blast which is 0.07. This probability is based on the ground velocity relationships for a 1600 foot distance and is used in the estimate of multiple blast damage.

The probability of damage in the form of block movement from a single blast at the minimum distance is 0.006 or 6 chances in 1000. This probability is based on the shear to normal stress ratio for failure considering the existing cracked mortar to behave as a friction material with no cohesive strength as discussed in detail in Appendix A. The average probability of block movement based on the 1600-foot distance is 0.0001. This single blast probability is used in the determination of multiple blast damage.

7.3.2.2 Plaster Damage

The probability of plaster damage to the Meason House resulting from a single blast was determined in the same manner as for the mortar strain. A probability distribution was developed for the internal structural velocities and the damage probability determined from the intersection of a line of constant velocity of 1.0 inch per second with the probability curve. The velocity of 1.0 inches per second corresponds to a failure shear strain of 50 microinches/in which is sufficient to cause damage to the existing plaster considering initial stresses in the plaster resulting from sagging.

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[17] The probability analysis of RI 8507 suffers deficiencies in fitting cumulative damage data to a lognormal distribution. Inherent is the lognormal cumulative distribution function is a very low probability of damage in the upper tail. This means that the probability of damage would be very low for very high peak ground velocities. Thus, the use of lognormal plots to fit probability data in RI 8507 appears to be inappropriate.

To compute the internal velocity probability distribution, the following approach was used:

- Amplification factors were determined based on those shown in RI 8507 corresponding to midwall frequencies. Only those amplification factors in the range of approximately 10 Hz. to 30 Hz. were included because the natural frequencies of the Meason House are within this range.

- A probability distribution for amplification factors was constructed based on a statistical analysis of the amplification factor data.

- A probability distribution of amplification factor time peak ground velocity was constructed to yield the probability distribution of internal velocities.

- The probability of damage was found from the intersection of the internal velocity probability distribution and a line of constant internal velocity of 1.0 inch per second which corresponds with the shear strain required to fail the plaster.

Figure 27 shows a plot of amplification factors as a function of frequency taken from RI 8507.[18] The shaded area on Figure 27 shows the range of natural frequencies determined for the Meason House from the dynamic analysis in Appendix A.

The amplification factor data for two-story structures that lie within the shaded area of Figure 27 were analysed statistically to determine the mean and standard deviation. A lognormal distribution was assumed so that the logarithms of the amplification factors were used in the analyses. Appendix B contains the results of the statistical analyses.

Based on the assumption that the amplification factors are lognormally distributed, a probability distribution was constructed for the probability of exceeding a given value of amplification factor. Figure 28 shows the resulting probability distribution.

The internal structural velocity is determined by the product of the amplification factor and the peak ground velocity. Since both the peak ground velocity and amplification factors were taken to

[18] Figure 28 was developed from Figure 40 of RI 8507

be lognormally distributed, the variance[19] of the distribution of the product of amplification factor and peak ground velocity was computed from the following relation:

$$\mathrm{Var}(\mathrm{Log}\ V + \mathrm{Log}\ Q) = \mathrm{Var}(\mathrm{Log}\ V) + \mathrm{Var}(\mathrm{Log}\ Q)$$

where;

Var( ) implies the variance of the random variable within the parentheses,
Log V is the logarithm of the peak ground velocity, and
Log Q is the logarithm of the amplification factor.

The above expression is an implementation of the standard relationship for the variance of the sum of two random variables.[20] The additional term involving the covariance of V and Q typically found in the relationship was zero because V and Q are statistically independent. From the variance of the product of V and Q, the probability distribution of the product was constructed as shown on Figure 29. Two distributions are given on Figure 29, one for the minimum distance of 1250 feet and one for the average distance of 1600 feet. The probabilities determined from the average distance were used in the analysis of multiple blast probabilities.

The probability of plaster damage from a single blast was found from the intersection of the probability distribution curve on Figure 29 with the line of constant velocity of 1.0 inch per second to be 0.08 or 8 chances in 100. For the average distance of 1600 feet, the probability of damage was 0.04.

7.4 MULTIPLE BLAST DAMAGE

The damage probabilities determined in the previous section are for a single blast. As discussed relative to airblast considerations in Section 6.0, mining activities over the course of a permit life involve numerous blasts. Blasting during the early stages of the permit life will be controlled by the DER criteria of a 0.5 inch per second peak ground velocity limit at the Connellsville school and at the creek bank. However, during the latter stages of the planned mining activities, vibration levels at the Meason House will be the controlling factor. While the number of blasts is unknown, the effects of multiple blasting on the overall probability of damage to either the limestone mortar or the plaster was estimated based on one blast per week for two

[19] The variance is the square of the standard deviation
[20] For those readers unfamiliar with logarithms, the variance of a sum is used because the sum, Log V + Log Q is equal to the logarithm of the product V•Q.

years, or a total of 100 blasts. However, overall damage probabilities were computed as a function of the number of blasts so that the effects of more or less blasting can be judged.

As was the case for airblast, damage to the mortar or plaster of the Meason House can result from either of two phenomena:

- Fatigue or cumulative damage from large numbers of cyclic stresses and strains, and
- Repetitive trials of a random event.

7.4.1 Fatigue

While data is limited relative to fatigue damage of either mortar or plaster, that which is available indicates that many more cycles are required before fatigue becomes a major consideration in damage probability. Undoubtedly, the existing condition of the mortar relative to existing cracks and the sagging plaster, will make them more damage-prone through fatigue than if there were no existing cracks or sag in the plaster. However, based on 100 blasts over two years and approximately 10 cycles of peak velocity per blast, the resulting 1000 cycles is less than the number of cycles required for failure as reported in RI 8507. Therefore, aside from the qualitative recognition that cumulative fatigue damage may aggravate the damage probability, no quantitative assessment was possible.

7.4.2 Random Trials

The single blast probabilities determined for the average distance of 1600 feet provided the basis for the determination of multiple blast probabilities. The average distance was used instead of the minimum because not more than a few blasts would be expected at the minimum distance. The average distance of 1600 feet was selected to be representative of the distance for the assumed 100 blasts behind the Meason House.

Each blast constitutes one realization of a random event with a probability of damage. For mortar cracking the probability is 0.07. The probability of sliding of limestone blocks on mortar cracks is 0.0001, and the probability of damage to the plaster is 0.04. With repeated blasting, there are repeated trials of the same random event which as a probability which remains constant for each blast. This situation can be modelled by means of a binomial distribution to compute the overall probability of damage for N blasts where N has been taken to vary from 1 to 100.

Figure 30 shows the overall probability of mortar cracking as a function of the number of blasts based on the binomial distribution. For 100 blasts, the probability of damage to the mortar, principally by extending existing cracks, is essentially 1.0, or it is a certainty to have at least one incident of additional cracking sometime during the 100 blasts. Figure 30 also shows the probability of more than two and more than three damage events occurring.

Figure 31 shows the probability of a limestone block in the walls of the Meason House moving as a function of the number of blasts. The probability for 100 blasts is 0.01 or 1 chance in 100. It should be noted that limestone block sliding does not imply destruction. Typical blast durations would not be long enough to result in any substantial movement. Sliding means that a small permanent deformation could occur between blocks in the wall. Figure 31 shows the probability for more than one event occurring. Probabilities for two or more events occurring are too small to show on the plot.

Figure 32 shows the probability of damage to the plaster in the Meason House as a function of the number of blasts based on a probability of damage of 0.04 for each blast and the binomial distribution. For 100 blasts, the probability of plaster damage becomes approximately 0.97 or almost certainty that more than one damage event occurs. Also shown on Figure 32 are the probabilities of more than two and more than three damage events occurring.

7.5 PROPOSED DER ADDITIONAL CRITERIA

Based on discussions with the DER, the additional criteria anticipated for the proposed blasting has two components:

- Limiting peak ground velocities at the Meason House to 0.5 inches per second, and
- Requiring the use of the waveform superposition concept of determining interval delay times which modify the resulting ground velocities relative to peaks and frequency content to minimize damage potential.

7.5.1 Velocity Limits

Based on the scale distance relationship at a distance of 1250 feet, the mean expected peak ground velocities are approximately 0.1 inch per second. From Figure 22, the probability of exceeding 0.5 inches per second at the Meason House is approximately 0.01 or 1 chance in 100. Thus, if peak ground velocities are consistently higher than 0.1 inch per second and are

consistently near to 0.5 inches per second, the damage probability for both the plaster and the mortar would increase significantly, especially since threshold damage to the mortar is predicted at about 0.2 inches per second.

A consistent peak ground velocity higher than expected from the regression analysis plots based on a charge weight per delay of 125 pounds would indicate the presence of some anomaly in the wave propagation such that the scaled distance relationship used in this analysis would be invalidated. Thus, while monitoring all blasts can serve to validate or invalidate the expected peak ground velocities, a limit of 0.5 inches per second may be too high to accomplish the goal of minimizing the damage potential to the Meason House, and could conceivably result in increased probabilities of damage.

7.5.2 Superposition of Waveforms Concept

The concept of the waveform superposition technique is to determine a delay interval which minimizes the damaging aspects of blasting vibrations by changing the frequency content. The fundamental assumption in the technique is that the ground vibration time history for each successive delay has the same wave form and frequency content. This assumption permits the calculation of an optimum delay interval to modify the characteristics of the resulting superimposed waves.

As an illustration of the concept, an artificial blast wave was simulated based on the sum of three frequency components, 1, 5 and 10 Hz with equal amplitudes. A second identical wave was then superimposed on the first wave after a varying delay period. Figure 33 illustrates the superposition effects for a delay interval of 500 milliseconds which is longer than typical blasting delays, but serves to illustrate the effect of eliminating low frequency content in blast waveforms. As illustrated on Figure 33, the effect of the 500 millisecond delay between the first and second blast was to completely eliminate the 1 Hz. and 5 Hz. components of the wave leaving only the 10 Hz. component. While the low frequency components of the wave were eliminated by superposition, the peaks are not significantly lower. Thus, if this simple example were an actual blast waveform, the effects of the 500 millisecond delay would remove the frequency content below 10 Hz. in the ground velocity with the peak ground velocity essentially unchanged. The 500 millisecond delay resulted in complete removal of the 1 Hz and 5 Hz components because it involves a shift of 1/2 of the 1 Hz. period and 2.5 times the 5 Hz. period. In both cases, it is the half period portion of the shift which effects the complete cancellation. Such simple relationships are not possible for waveforms having a continuous frequency spectrum.

The determination of the optimum delay interval for actual blast waves is more complex than the above illustration, but the principle involved remains the same. Computer programs are used for determining superposition effects with delay intervals varied until an optimum is found which provides the greatest amount of removal of low frequency wave components without increasing the peak ground velocity.

The basis for the application of this technique to the proposed blasting would be a recorded test blast at the production charge weight per delay of 125 pounds. This would provide the basic waveform which would be shifted by varying delays until optimum frequency component reduction is achieved.

The effectiveness of the technique in mitigating potential damage to the Meason House depends to a great degree on how well relatively high frequency components can be removed. For conventional structures, reduction of frequency components below 10 Hz. can remove a significant potential for damage because the natural frequencies of the structures are in the range of 4-6 Hz. Removal of frequency components below 10 Hz removes resonance possibilities for conventional structures. In addition, when a structure is excited by frequencies greater than its natural frequency, the response becomes mass controlled, decreasing as the frequency increases.

For the Meason House the natural frequencies range from approximately 10 Hz to 30 Hz based on the structural analysis performed. Thus, removal of frequency components below 10 Hz by adjusting the delay interval would not be expected to eliminate resonance possibilities. It would, of course, remove the lower frequency, non-resonant response which typically results in the maximum displacements but not necessarily the maximum velocities. Removal of frequency components up to 30 Hz using the superposition method, is expected to be more difficult than for removal up to 10 Hz. A 10 Hz waveform has a period of 0.1 seconds or 100 milliseconds. Thus, the minimum delay time to remove 10 Hz becomes 50 milliseconds based on a shift of one-half the period. To achieve cancellation, the delay interval, D, must have the following relationship to the periods of the frequencies removed;

$$(n + 1/2)\,T = D$$

where n is any integer from 0 to infinity. This shift effectively converts sine components to cosine components, which when superimposed will cancel. Solving the above equation for T and using the fact that frequency is the inverse of the period, frequencies cancelled by a delay of D seconds would be expected to be:

$$f = (n + 1/2)/D$$

For a delay of 50 milliseconds, the frequencies would be

$$f = (n + 1/2)/.050 \text{ or } f = (n + 1/2)\ (20)$$

With n = 0, the 10 Hz component is cancelled and with n=1, the 30 Hz component is cancelled. Intermediate as well as lower frequency components would be modified, but not cancelled.

Consequently, using the wave superposition concept to mitigate potential damage to the Meason House appears to be difficult to achieve. Some benefits may be obtained, but prudence indicates that the effectiveness of the method be demonstrated relative to structures having frequencies in the range of 10 Hz to 30 Hz. before concluding that a significant reduction in damage potential can be achieved.

8.0 UNDERGROUND MINE SUBSIDENCE

The final source of potential damage arises from the possibility of additional subsidence of the Mt. Braddock underground mine and its potential relationship to the proposed blasting. Since retreat mining was used, subsidence was planned and has occurred to some degree. The concern relative to the Meason House arises from pillars which were left in place to the northeast of the house and their potential for collapse which could result in additional subsidence and widening the subsidence profile to the point that strains are induced in the Meason House foundation. A critical question is to what degree, if any, the proposed blasting could create dynamic stresses in the pillars which could either hasten an eventual collapse or cause a collapse which would not have occurred.

8.1 MT. BRADDOCK UNDERGROUND RETREAT MINING

The Mt. Braddock mine was operated until 1978 by U. S. Steel Corporation. Figure 10 shows the map of the underground working superimposed on the topographical map showing the Meason House. The largest shaded area on Figure 10 shows the coal which was removed during the U.S. Steel operations. Subsequently, in 1983 additional mining occurred in the Mt. Braddock mine. Coal remaining near the original mine entries was removed as shown by the two additional shaded areas on Figure 10.

The Mt. Braddock mine used the retreat method of mining in which essentially 100 percent of the coal is removed from active panels with the intent of allowing the roof to fall as mining proceeds. Thus, subsidence from the Mt. Braddock mine was planned as a normal course of mining activities.

Predicting subsidence is not an exact science so that many times the planned subsidence does not occur as planned or as soon as planned. The degree to which actual subsidence compares with planned subsidence depends primarily on the overburden rock condition. If the overburden is composed of weak and fractured rock, roof collapse typically occurs as planned. However, if competent, strong rock strata exist in the overburden, the roof can "hang up" until the stress in the competent strata is sufficient to cause failure and collapse into the opening. While no specific information was reviewed concerning operations at the Mt. Braddock mine, the geology of the area suggests that subsidence should have occurred reasonably as planned. Located on an anticline, the roof strata dip to the northwest such that a fracture pattern would be expected which has a strike to

the northeast. Fractures in overlying rock would be expected to parallel the entry directions running across the page on Figure 10.

While the presence of fractures would have induced subsidence as planned, it can not be known for certain that all of the potential subsidence has occurred. As discussed subsequently, a house located several hundred feet in front of the Meason House was damaged by subsidence which presumably was not planned. Therefore, it is at least reasonable and prudent to question whether all of the possible subsidence has occurred.

In particular, the pillar shown on Figure 10 may pose a threat for future subsidence. The purpose of the pillars is not clear. One possible purpose could have been to reduce the span of the opening in a northeasterly direction behind the Meason House to reduce the spread of the surface subsidence profile. While the pillars accomplish this objective, it is conjecture as to their original purpose. The threat to the Meason House from subsidence results from the following scenario:

- If the pillars have not collapsed, they are providing overburden support which reduces the width of the subsidence profile to the northeast of the Meason House.

- If the pillars, which as a minimum are subjected to greater that lithologic stresses, collapse due to a possible interaction with the surface mine blasting, additional subsidence could occur.

- If the additional subsidence occurs, the surface profile could extend to the Meason House which is located on the order of 200 feet from the edge of the protective pillar left in place beneath the house.

Subsequent sections address the issues relative to this chain of events in an effort to determine to what degree the potential for blasting to interact with the pillars might pose a threat of damage to the Meason House.

8.2 PILLAR STRENGTH AND LOADING

The first issue to be addressed relative to the above chain of events by which blasting might result in subsidence damage to the Meason House is the current state of the pillars left in place as shown on Figure 10. If the pillars are not overstressed, any blasting induced stresses would act on

essentially competent rock and coal strata. If the pillars are greatly overstressed, they have probably collapsed already in which case the threat of additional collapse would not be great.

The area of the pillars is irregular based on the underground mine map. For analytical purposes, a square pillar has been assumed with a width of 150 ft which corresponds to two panels plus the entry between.

8.2.1 Pillar Strength

Peng[21] gives the following relationship for the strength of a square pillar based on the progressive theory of failure:

$$P_s = 4\rho gh(W_b^2 - 3W_bhH \times 10^{-3} + 3h^2H^2 \times 10^{-6})$$

where;

Ps is the pillar strength in tons,
ρg is the weight density of average rock taken as 0.0707 tons per cubic foot,
h is the depth of the overburden in feet,
H is the seam thickness in feet, and
Wb is the seam thickness in feet.

For an approximate depth of overburden of 300 feet and using the density ρg = 0.0707, and a 10-foot thick seam, the strength of the pillar would be;

$$P_s = 4(.0707)(150^2 - 3(150)(300)(10)\times 10^{-3} + 3\times 300^2 10^2 \times 10^{-6}$$
$$= 1.8 \times 10^6 \text{ Tons.}$$

8.2.2 Pillar Loading

Using Peng's relation that a pillar supports the overlying material plus 30 percent of the roof outside the pillar area, the load on the pillar is given by:

$$P_L = (W_b + 0.6h)^2$$
$$= (150 + 0.6(300))^2$$
$$= 2.3 \times 10^6 \text{ Tons}$$

[21]Peng, S.S., **Coal Mine Ground Control**, Wiley Interscience, New York, 1978

8.2.3 Pillar Safety Factor

The pillar load is greater than the pillar strength. The safety factor based on the above calculations would be 1.8/2.3 = 0.78. This indicates that the pillar is overstressed and should have failed or be in the process of failing. However, a more precise analysis based on the actual geometry considering the effects of any neighboring pillars could conceivably result in a pillar safety factor around 1.0. Therefore, it must be concluded that the existing pillar strength is marginal. It could be sufficient that the pillars have not failed. Or, it could be at the point where initial pillar failure has occurred but has not completely collapsed.

Pillar failure typically occurs as a result of shear stresses within the pillar. The stress relieved zone around the periphery of the pillar creates a free surface which tends to spall off as compression stresses increase. The spalling action reduces the area of the pillar available to resist the overburden load resulting in increased stresses and more spalling. This process continues until the pillar completely collapses. The process may occur slowly or may happen suddenly depending on the stress distributions and the fractures within the pillar.

As the pillar spalls, displacements occur which allow the roof to converge over the pillar. The spalling mixes with gob which surrounds the pillar and to some degree helps fill in any voids left around the pillar. As roof convergence occurs over the pillar, the material in the collapsed pillar is consolidated until it tends to become part of the overall gob area left by the mine.

Based on the above analysis, it appears that the pillars shown on Figure 10 could be anywhere in the above process. This complicates an assessment of potential interaction of the pillars with blast induced stresses because the existing stress state is marginal.

8.3 SUBSIDENCE DAMAGE TO CELLURALE HOUSE

As mentioned previously, subsidence at the Mt. Braddock underground mine has resulted in damage to the Cellurale house located several hundred feet in front of the Meason House. Figure 10 shows the specific location. It is presumed that the subsidence which damaged the Cellurale house was not planned and is indicative that either the angle of draw was greater than anticipated or some other unforeseen event occurred.

In an attempt to determine whether analytical subsidence models could be used to assess the potential for damage to the Meason House resulting from subsidence, the opening to the south of the Cellurale home was modelled mathematically. The analysis was performed using the methods

developed by the National Coal Board (NCB) in England and given by Peng.[22] Figure 34 shows the results of the analysis in the form of plots of a symmetric half of the subsidence profile and a symmetric half of the strain profile which results from the ground surface displacements of the subsidence profile. (Distances on Figure 34 are in metric as opposed to English units because the NCB methodology uses dimension-dependent tables. Note that 1 meter = 3.28 feet.)

Figure 34 also locates the Cellurale house on the subsidence profile. As shown, the Cellurale house was located near the edge of the profile where the ground level curvature was approaching zero, but was large enough to induce differential movement in the Cellurale house which resulted in the damage. This is also shown on the plot of strain versus distance from the center of the opening, which is the shaded area to the south of the Cellurale house as shown on Figure 10.

While the predicted strain level at the Cellurale house is somewhat less than would correspond with descriptions of damage, the NCB methodology did predict subsidence damage where it occurred and was judged adequate for examining potential subsidence effects on the Meason House.

8.4 POTENTIAL SUBSIDENCE DAMAGE TO MEASON HOUSE

The NCB method was used to predict subsidence arising from the mined out area behind the Meason House. The span was taken across the area in a northeast-southwest direction between the edge of the protective pillar on which the Meason House sits and the closest of the pillars identified on Figure 10 and discussed previously relative to their stress conditions. Figure 35 shows plots of the symmetric halves of the surface subsidence profile and strain profile (Note units are metric). The location of the Meason House is also shown on the figure at its distance from the center of the underground opening.

As shown, the Meason House is on the order of 20 meters (65 feet) outside of the subsidence profile which was based on the underground opening between the edge of the protective pillar and the pillars to the northeast. This result tends to indicate that the pillars in question are still providing support to the overburden and have not collapsed. The basis for this observation is that if the pillars had collapsed the subsidence profile would have spread further than the prediction shown on Figure 35 and might have reached the Meason House. The lack of subsidence damage at the Meason House coupled compared with a prediction based on the pillars continuing to provide

[22] Ibid

support and the apparent marginal state of the pillars allow the inference that the pillars have not collapsed.

The same analysis would indicate that if the pillars do collapse, the subsidence profile, which is presently fairly near the Meason House, would extend outward with possible damage to the house resulting. The analysis of subsidence without the pillars could not be performed because the span of the underground opening exceeds the range of data in the NCB tables. It should be noted that the mining subsequent to U.S. Steel operations which occurred in 1983-1984 has further increased the span of the potential subsidence zone if the pillars collapse.

The subsidence analysis that resulted in Figure 35 was based on a seam thickness of three meters (10 feet). As shown on the figure, the maximum subsidence at the center of the profile is less than 2.5 meters for the width of the opening considered. As the width of the opening increases, the percentage of total seam thickness which reaches the surface as subsidence increases. The results on Figure 35 would indicate that at least 0.5 meters (1.6 feet) could still subside if the opening width increases. In subsiding an additional 1.6 feet at the center, the edges of the profile would expand outward. This outward expansion is what poses the threat to the Meason House.

8.5 INTERACTION BETWEEN BLASTING AND SUBSIDENCE

Based on the results of the pillar stress analysis and the subsidence profile analyses, it appears that the pillars to the northeast of the Meason House have not collapsed, but are in a potentially marginal stress state with a factor of safety near 1.0. This situation creates the worst potential for damage to the Meason House from subsidence as well as difficulties in attempting to assess quantitative damage probabilities.

To examine potential detrimental effects on the pillars resulting from blasting, two approaches were used:

- A discussion of pertinent literature which could provide at least a qualitative idea of the potential for blasting aggravating pillar collapse, and

- A more quantitative approach based on limited data relative to effects of blasting vibrations on underground openings.

8.5.1 Literature on Blasting Vibration Effects on Underground Structures

Very little research has been conducted relative to damaging effects on underground structures, and what has been done generally lacks a quantitative assessment necessary to assess probabilities. The following discussion does not represent an exhaustive literature survey, but merely comments relative to literature which was readily available for this study.

At the outset it is important to recognize that the issue is not one of surface blasting effects on underground mines located some distance below the surface mine. The geological structure consisting of the coal outcrop near the location of the surface mine is such that blasting of overburden rock will be blasting in the roof strata of the underground mine. The geologic section shown on Figure 8 helps to clarify this situation. The dip towards the northwest of the seam and overlying rock strata are such that while the seam is approximately 400 feet beneath the Meason House, both the seam and the sandstone and shales in the roof outcrop around the location of the surface mine. Thus, the rock strata which form the overburden for the surface mine are the roof strata for the underground mine. This was observable during the performance of the five pound test shots near a former air shaft for the underground mine. The opening into the mine at the base of the mine pit illustrated that the rock to be blasted is the roof strata of the mine.

Blasting in the mine roof strata is different from surface mine blasting where vibration waves have to be transmitted downward to reach an underground mine. In the latter case, multiple reflections caused by the layered nature of typical coal geology decrease the downward propagation of vibrational energy such that vibration amplitudes at depth are lower than would be expected in the approximately horizontal propagation within one geologic strata.

The greater attenuation of vibrations with depth compared with surface vibrations was demonstrated by Jensen, et. al.[23] in the study of vibrations in an underground mine in Kentucky from blasting in a surface mine approximately 140 feet above the underground mine. The only path for vibrations was through the overlying strata. At the Mt. Braddock surface mine, vibrational energy will not have to propagate downward through alternating layers to reach the mine. Therefore, charge weight per delay - scaled distance relationships can be expected to result in greater peak ground velocities at a given distance than the relations given by Jensen.

---

[23] Jensen, D. E., et. al., **Underground Vibrations from Surface Blasting at Jenny Mine, Kentucky**, NTIS PB80-168925, U.S. Dept. of Commerce, Nov, 1979.

In a discussion of underground blasting induced damage, Jensen cites a work by Tincelin and Sinou[24] who monitored deterioration in mine roofs near production blasts. According to Jensen, they observed that strain levels associated with blasting were considerably larger than those associated with continous miner operations. In another study referenced by Jensen, Isaacson[25] indicated that rock bursts initiated on planes of weakness away from openings were caused by the addition of dynamic stresses from blasting to high static stresses. While this would be comparable to the situation at the pillars, no quantitative data was presented in Jensen. Peng[26] cites work by Habenicht and Scott[27] which shows significantly decreased roof bolt loads as a result of blasting vibrations, indicating interaction between vibrational strains and roof strata; however, no quantitative data is presented in Peng.

While limited and without quantitative data, some literature is available which as a minimum establishes a potential link between ground vibrations created by blasting and damage to underground structures. The reference to failures caused by the addition of dynamic stresses from blasting to high static stresses is particularly appropriate because damage levels associated with ground vibration velocities would seem highly dependent on existing stress conditions and the margin of strength. Analogous to the case of the mortar and plaster where existing conditions lowered margins for additional strains, higher than normal existing stress conditions in the pillars would also lower margins for additional blast-induced stresses. If the pillars have a factor of safety of 1.0 or slightly less, the margins for additional stresses could be very small.

8.5.2 Estimates of Damage Probability

Damage probabilities were estimated based on the probability of damage to the pillars which could result in incipient pillar collapse and subsequent subsidence damage to the Meason House as the surface subsidence profile spreads.

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[24] Reference unavailable due to missing page in Jensen report. This may be a worthwhile reference to obtain for future consideration as Jensen indicates correlations were made between damage levels and peak ground velocities, but does not cite the results.

[25] Isaacson, E. Q., "Stress Waves Resulting from Rock Failure", **Fourth Symposium on Rock Mechanics, Bulletin of the Mineral Industries Experiment Station**, Pennsylvania State University, University Park, 1961, pp. 153-161, cited in Jensen.

[26] Loc. cit.

[27] Habenicht, H. and J. J. Scott, "The Influence of Shock Waves on the Stability of Rock Bolt Anchorages," Presented at the Annual Meeting of AIME, New York, Feb 24-Mar 3, 1966, Preprint No. 66-FM-17

8.5.2.1 Damage Threshold

Data which correlates peak ground motion from blasting with damage is extremely limited. In the Air Force Manual for Hardened Structures[28], damage in the form of roof spalling is given as at an acceleration level of approximately 8g where g is the acceleration of gravity.

A problem with this value of acceleration required to produce damage is the condition of the rock structures on which the data was acquired. Dynamic stresses and strains produced by blasting are superimposed on existing static stress states in rock which can be expected to vary considerably from one structure to the next. This variation of existing static stresses is one of the major contributors to the scatter in damage data for all structures because the dynamic stresses are superimposed on existing static stresses to cause failure and rarely, if ever, are attempts made to quantify existing static stresses.

In the case of the pillars shown on Figure 10, existing stresses would appear to be marginal with respect to strength. Thus, existing static stresses have utilized almost all of the strength capacity available. The reserve strength for the addition of even small dynamic stresses would appear to be small and less than recommended damage threshold values for rock structures with more reserve capacity.

In an effort to provide a quantitative measure of the probability of damage to the pillars caused by blasting, an approach similar, but less rigorous to that used for the mortar and plaster was followed. Safety factors provide the margin of excess capacity in the design of underground openings and are typically defined as;

$$F_s = \sigma_u/\sigma_w$$

where;

F_s is the safety factor,
σ_u is the ultimate strength capacity of an element, and
σ_w is the working strength for that element which is used in design.

[28] Crawford, R. E., C. J. Higgins and E. H. Bultman, **The Air Force Manual for the Design and Analysis of Hardened Structures**, AFWL-TR-102, Air Force Weapons Laboratory, Kirtland AFB, New Mexico, 1974.

If dynamic stresses from blasting are anticipated in design, they are included in the working stress. If they are are not anticipated, they work against the margin provided by the factor of safety. This is one of the principal reasons for the use of safety factors, provisions for unanticipated events. If a safety factor is α, the margin for added dynamic stresses which were not considered in design is,

$$\sigma_d = \sigma_u(\alpha - 1)/\alpha$$

Where;

σ_d is the additional dynamic stress created by blasting.

Thus, if a safety factor is 1.5 which is characteristic of engineering designs, the additional margin for dynamic stresses is $0.5/1.5 = 0.333\ \sigma_u$. If the factor of safety is 1.1, the additional margin for dynamic stresses becomes $0.1/1.1 = .091\sigma_u$. The additional dynamic stresses in the latter case as indicated by the peak ground acceleration, assuming linear behavior, would be 0.091/0.333 = 0.273 or 27.3 percent of the dynamic stresses in the former case.[29]

Based on the analysis of pillar stresses, an existing factor of safety of 1.1 would be conservative. Assuming the typical underground structures for which use is intended would use a factor of safety of at least 1.5 in the design, a peak acceleration of 25% of the published data for the pillars would not seem unreasonable. The percentage could be smaller if the factors of safety for structures at which data has been obtained were higher than 1.5 or the factor of safety for the pillars were lower than 1.1. Thus, the above damage level would become 2g.

8.5.2.2 Scaled Distance Relationship

For damage level acceleration comparison, the following scaled distance relationship given by Olson, et. al.[30] was used to determine acceleration as a function of distance from the blast:

$$AW^{1/2} = 50{,}000(D/W^{1/2})^{-2.21}$$

where;

A is the peak-to-peak ground acceleration,

[29] This effect is well known in the area of fatigue due to alternating stresses where the presences of a mean static stress reduces the allowable alternating stress in a fatigue life evaluation.

[30] Olson, J. J., et. al., **Mine Roof Vibrations from Underground Blasts**, U. S. Bureau of Mines, RI 7330

W is the charge weight per delay,
D is the distance from the blast in feet.

In logarithmic form the above equation becomes:

$$\text{Log}\,(AW^{1/2}) = \text{Log}\ 50{,}000 - 2.21\ \text{Log}(D/W^{1/2})$$

Based on the logarithmic form, Olson cites a standard deviation around the mean regression curve of 0.356. The combination of the scaled distance relationship and the standard deviation permits the generation of a probability distribution for peak ground acceleration.

This relation was used instead of the relationship used for peak surficial ground velocities because it was developed based on blasting in underground mines. It has been recognized that blast wave propagation in underground rock strata is different from that on the surface. As discussed previously, the geological nature of the proposed blasting site is such that the surface mine blasting will occur in the rock strata which form the roof of the mine.

Figure 36 shows a map of the mining areas with the locations of the pillars superimposed. Based on scaling distances from the map, the minimum distance to the pillars is approximately 720 feet. In addition, a distance of 1,000 feet was used as the average distance for the assessment of multiple blast damage probabilities.

8.5.2.3 Damage Probabilities

Figure 37 shows the probability distribution for peak accelerations at the pillar locations for distances of 720 feet and 1,000 feet based on the above scaled distance relationship. As shown on the figure, the probability of pillar damage from a single blast based on the minimum distance of 720 feet is 0.004 or 4 chances in 1,000. For the average distance of 1,000 feet, the single blast probability is 0.0002 or 2 chances in 10,000. The latter probability was used in the calculation of the probability of damage from multiple blasts based on the random trial model used for airblast and direct ground vibrations. Figure 38 shows a plot of the probability of damage to the pillars as a function of the number of blasts. For the assumed 100 blasts, the probability of pillar damage is 0.02.

9.0 CONCLUSIONS

Table 5 shows a summary of the damage probabilities for the five sources of damage from blasting which were considered. Basically the results in Table 5 are self explanatory, and no attempt has been made to interpret whether the probabilities pose acceptable or unacceptable risks.

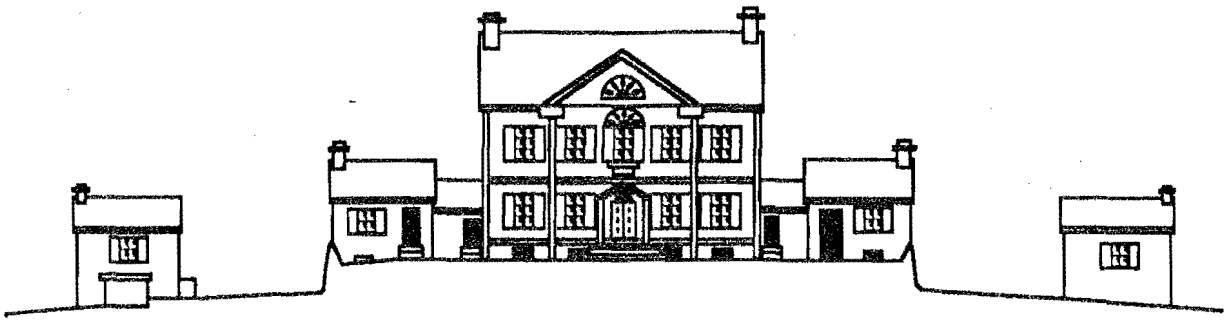
TABLE 5
SUMMARY DAMAGE PROBABILITIES

CAUSE	DAMAGE	PROBABILITY 1 BLAST	PROBABILITY 100 BLASTS
Airblast	Windows	1×10^{-7}	1×10^{-5}
Ground Vibrations	Mortar Cracking	0.15	0.999
Ground Vibrations	Block Sliding	0.006	0.01
Ground Vibrations	Plaster Damage	0.08	0.975
Subsidence	All	0.004	0.02

It should be re-emphasized that probabilities given in Table 5 are the result of numerous assumptions required for their derivation. There is little basis for verifying the results beyond comparisons with empircal data when it is available and scrutiny of the assumptions on which they are based. Where data exists, the comparison has been favorable toward verifying the assumptions. In addtion, the assumptions are consistent with prudent engineering practice and rooted in sound principles of dynamic analysis which have been thoroughly scrutinized in the field of earthquake engineering. One of the goals of this report, beyond the assessment of quantitative probabilities, has been to focus attention on the issues involved. It is hoped that the presentation has succeeded in that goal.

The analytical models used to estimate the effects of existing conditions on damage probabilities represent a more rigorous approach than is typically applied to conventional residential structures. As stated in the introduction, the historic nature of the Meason House as well as its different structural characteristics and existing conditions from conventional residential structures was thought to warrant a more detailed evaluation.

FIGURES



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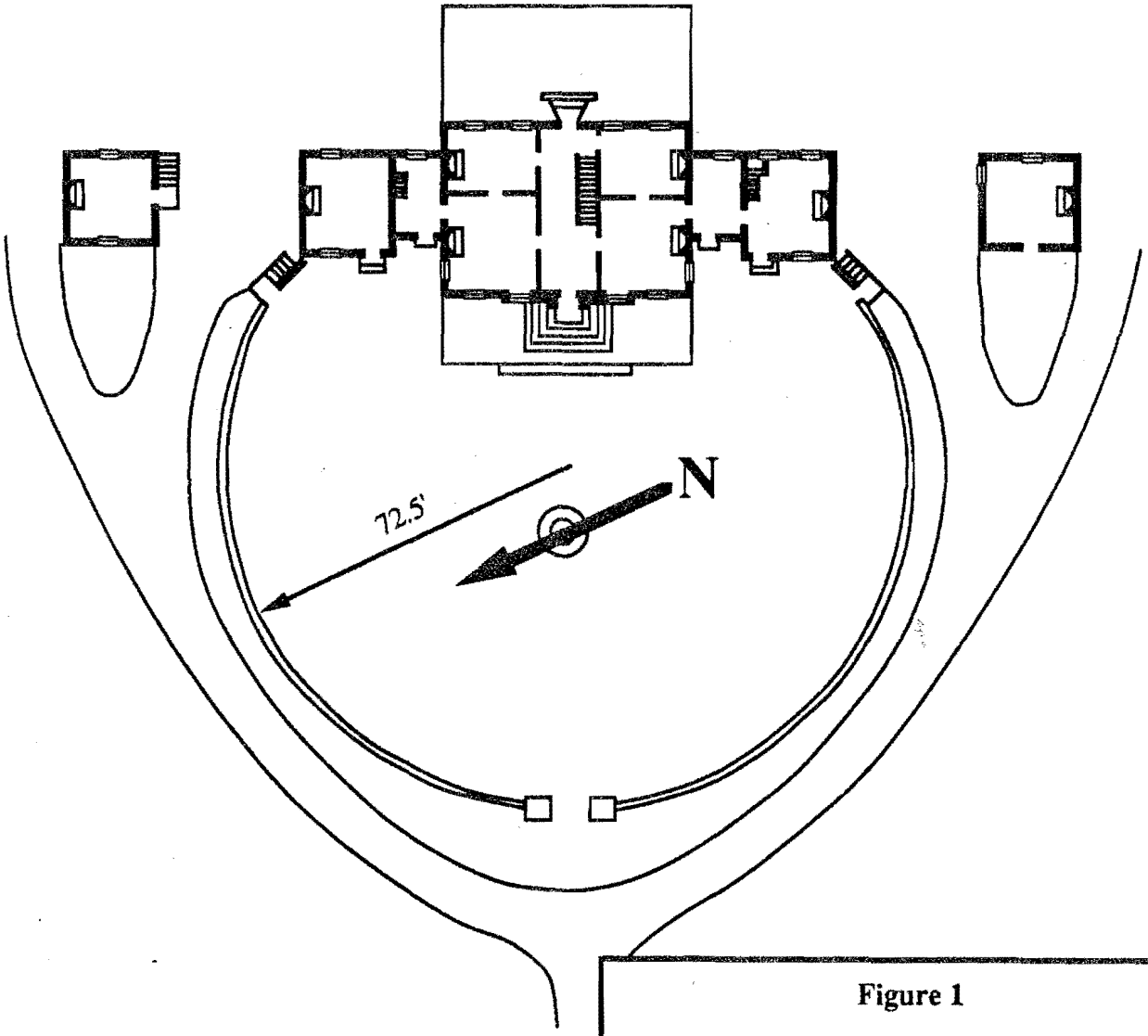

NOTE:
Reproduced by computer scanning figure from "The Early Architecture of Western Pennsylvania."

Figure 1

MEASON HOUSE AND GROUNDS

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NOTES:

1. All dimensions are approximate and were obtained by scaling the drawing based on the scale shown on the figure from "The Early Architecture of Western Pennsylvania"

2. Dimensions as shown were used in the mathematical analysis for natural frequencies

SCALE: 3" = 50'

Figure 2

MEASON HOUSE FRONT ELEVATION

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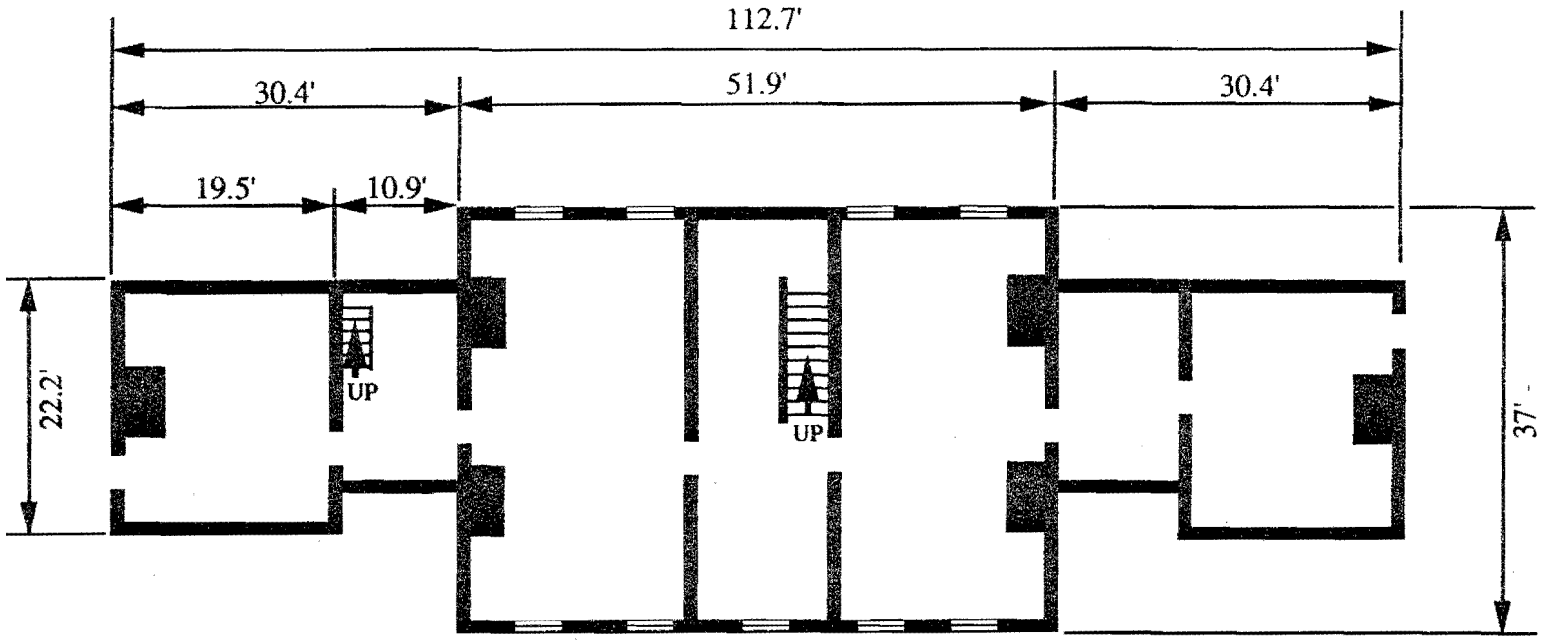

NOTES:

1. All dimensions are approximate and were obtained by scaling the drawing based on the scale shown on the figure from "The Early Architecture of Western Pennsylvania"

2. Dimensions as shown were used in the mathematical analysis for natural frequencies

SCALE: 3" = 50'

Figure 3

MEASON HOUSE
BASEMENT / FOUNDATION PLAN

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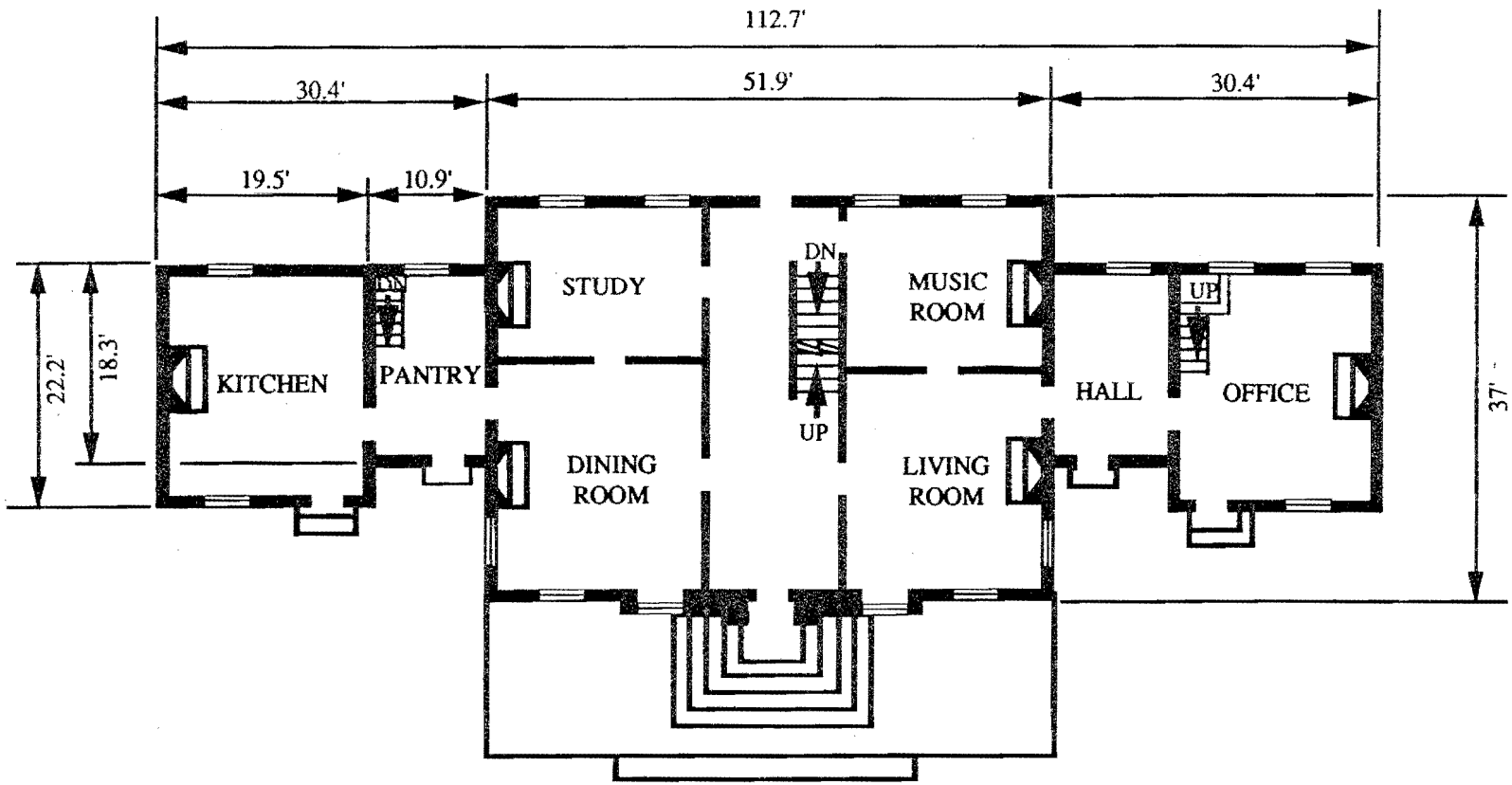

NOTES:

1. All dimensions are approximate and were obtained by scaling the drawing based on the scale shown on the figure from "The Early Architecture of Western Pennsylvania"

2. Dimensions as shown were used in the mathematical analysis for natural frequencies

SCALE: 3" = 50'

Figure 4

MEASON HOUSE FIRST FLOOR PLAN

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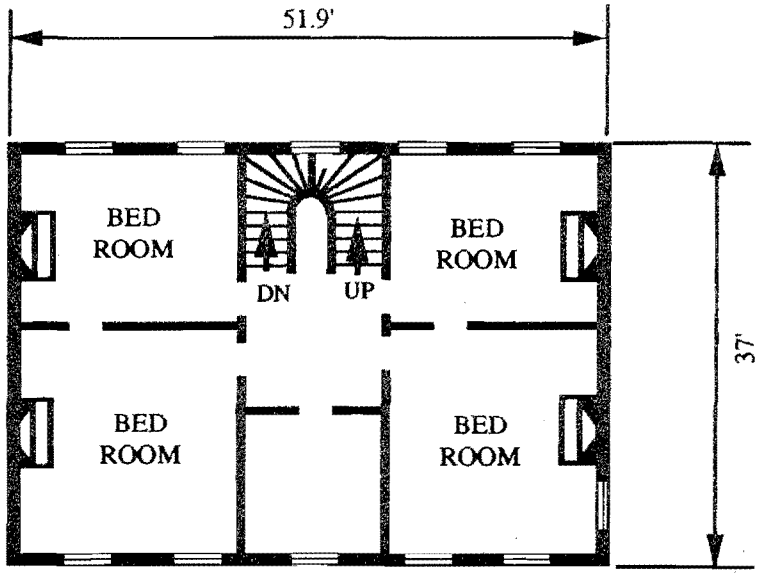

NOTES:

1. All dimensions are approximate and were obtained by scaling the drawing based on the scale shown on the figure from "The Early Architecture of Western Pennsylvania"

2. Dimensions as shown were used in the mathematical analysis for natural frequencies

SCALE: 3" = 50'

Figure 5

MEASON HOUSE
SECOND FLOOR PLAN

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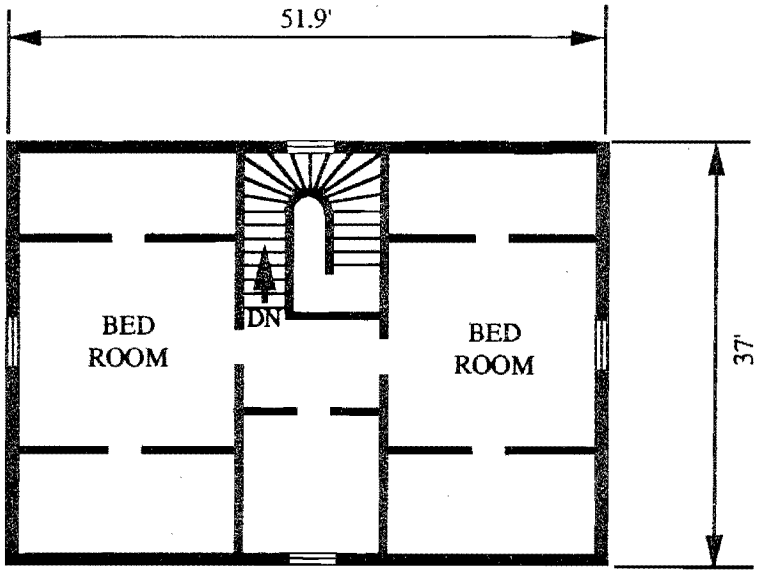

NOTES:

1. All dimensions are approximate and were obtained by scaling the drawing based on the scale shown on the figure from "The Early Architecture of Western Pennsylvania"

2. Dimensions as shown were used in the mathematical analysis for natural frequencies

SCALE: 3" = 50'

Figure 6

MEASON HOUSE
THIRD FLOOR PLAN

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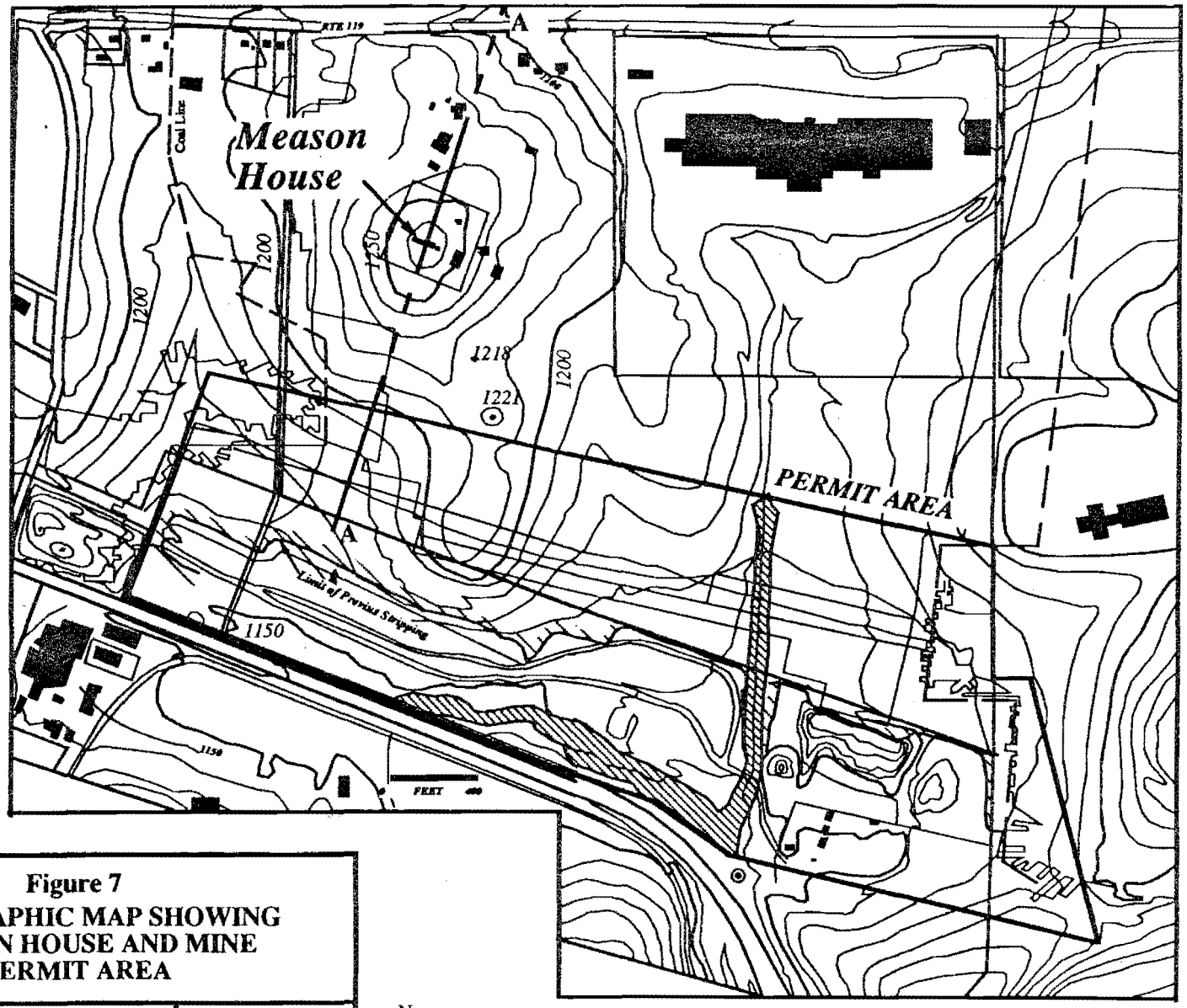

Figure 7
TOPOGRAPHIC MAP SHOWING MEASON HOUSE AND MINE PERMIT AREA

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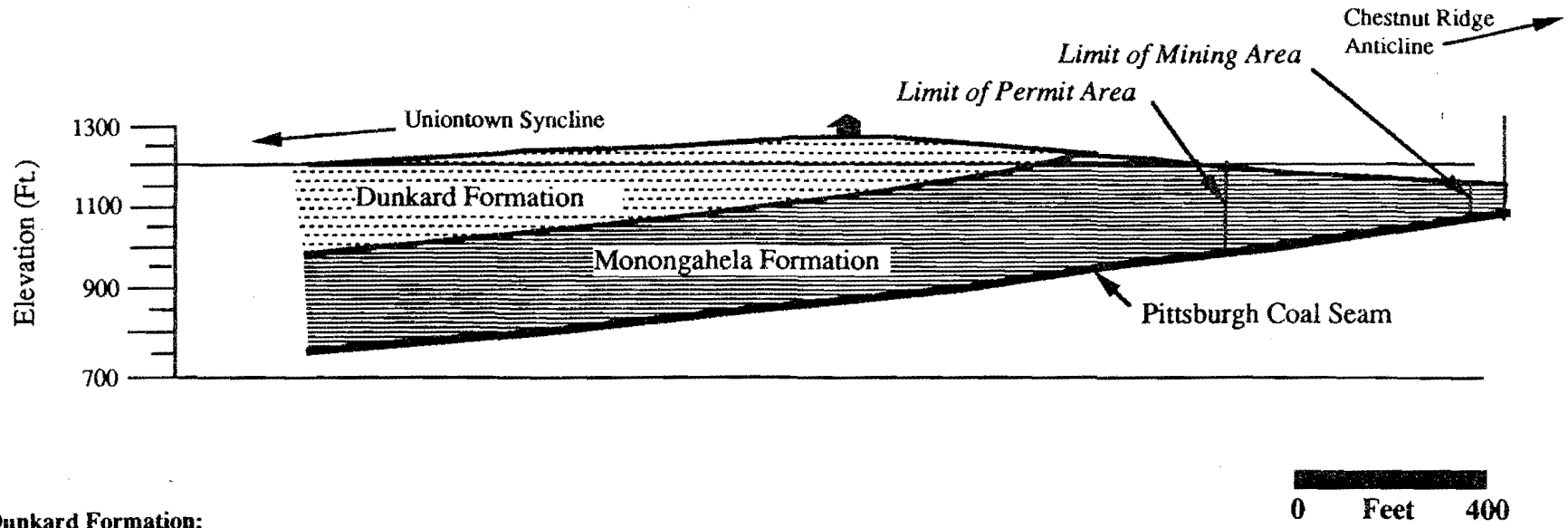
Note:
Unofficial map, for information only. Scanned from Bond Map obtained from DER on January 9, 1989.



Dunkard Formation:
Sandy shales with coarse sandstone and thin limestone strata.

Monogohela Formation:
Sandstones, shale, limestone and coal. Marked regionally by Waynesburg coal at the top (not present at site) and Pittsburgh coal at the bottom with a thick overlying sandstone layer (Pittsburgh sandstone). Erosion preceded deposition of sandstone and removed some shales so that sandstone is directly overlying coal some places. At other places the sandstone does not exist.

NOTES:
1. Cross section constructed graphically from intersections of Section A-A line with topographic contours and Pittsburgh seam contours.

2. Contact between Dunkard and Monogahela formations shown approximately for illustration purposes only.

3. Cross section is presented for conceptual illustration only and is not intended to be an accurate portrayal of detailed stratigraphy.

Figure 8
SECTION A-A
ILLUSTRATING GEOLOGY
UNDERLYING MEASON HOUSE

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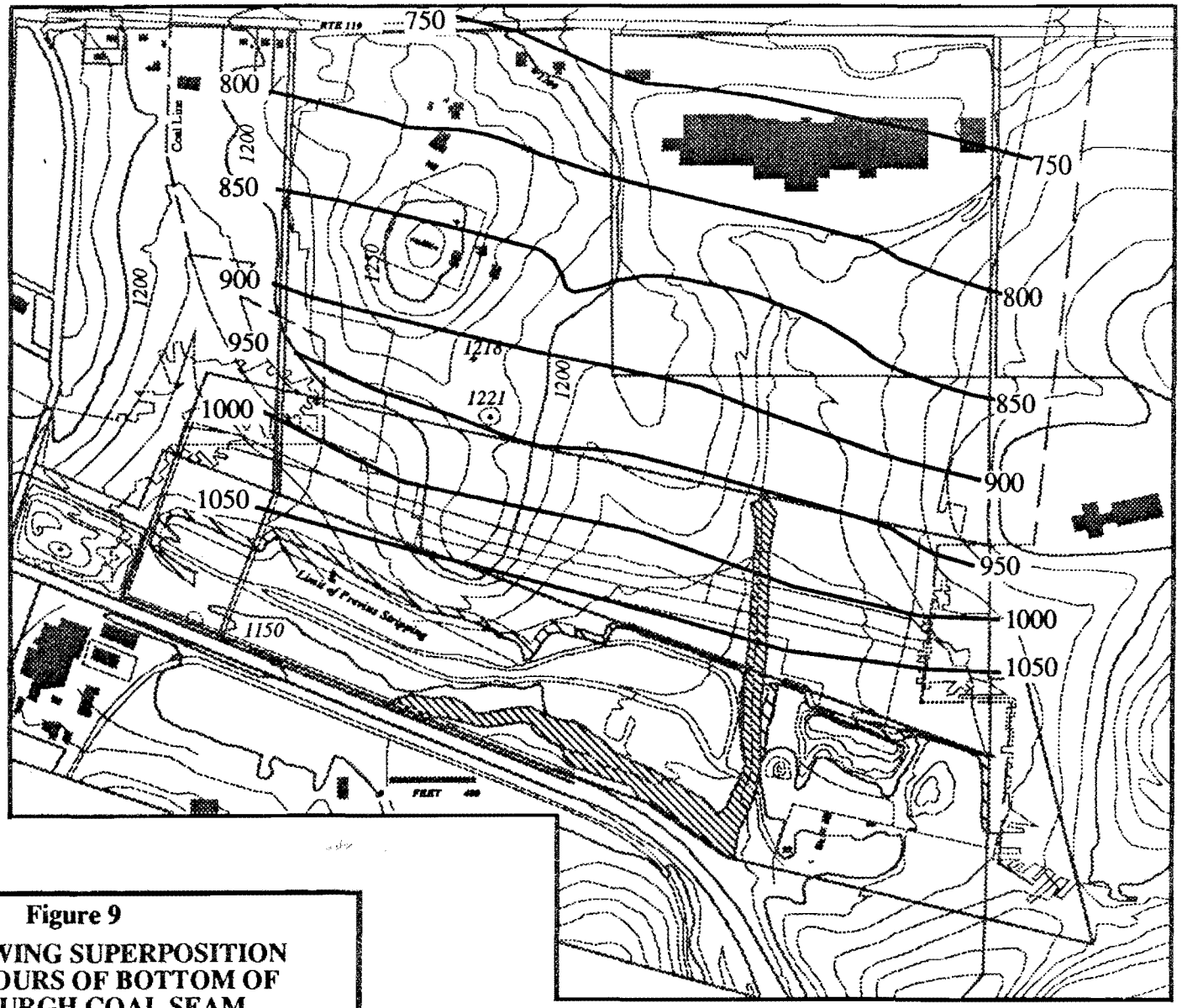

Figure 9

MAP SHOWING SUPERPOSITION OF CONTOURS OF BOTTOM OF PITTSBURGH COAL SEAM

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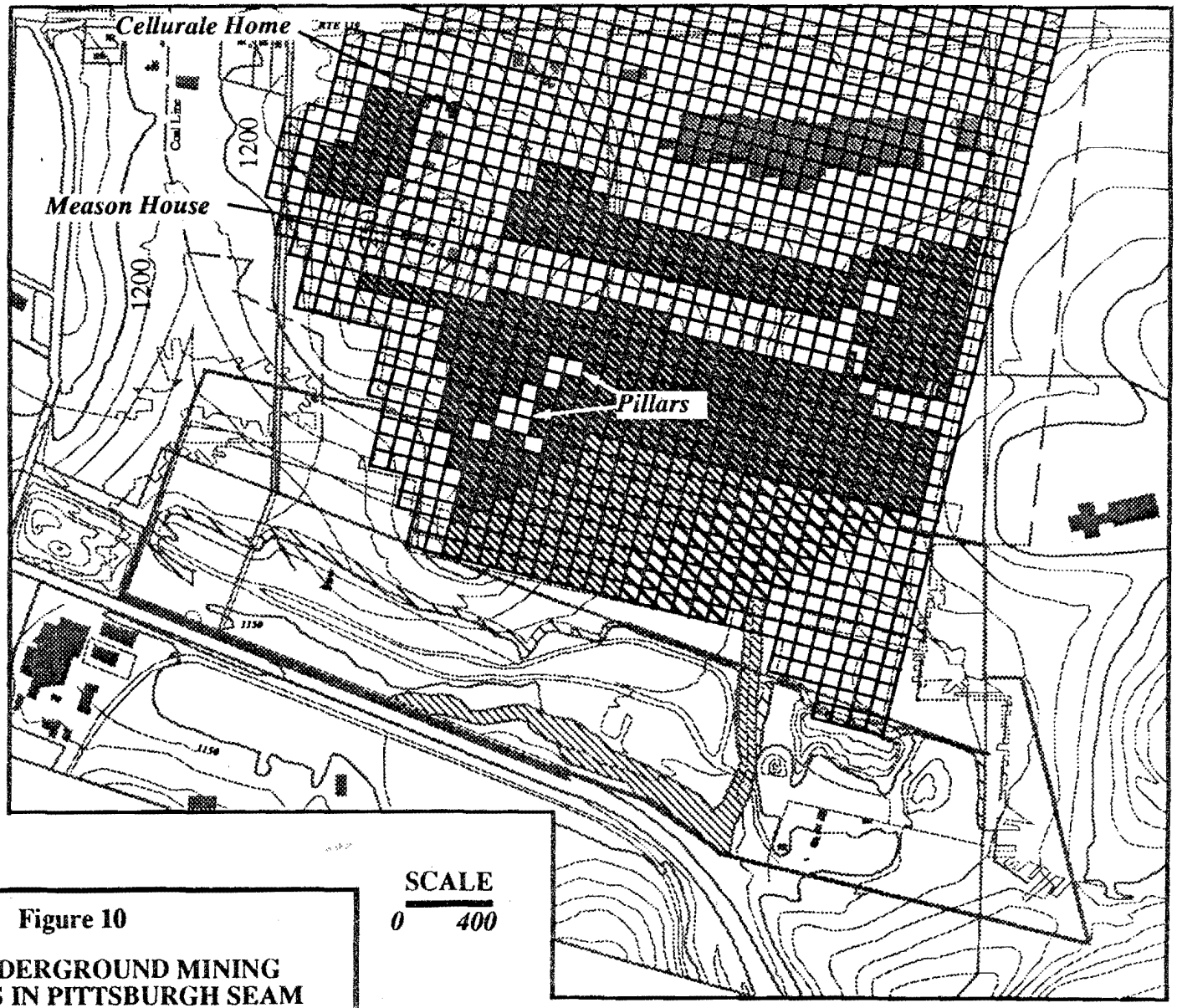
Note:
Unofficial map, for information only. Scanned from Bond Map obtained from DER on January 9, 1989.



U. S. Steel mining to 1978

Mining from 10/1/83 to 2/28/84

Mining from 3/1/84 to 8/30/84

SCALE

0 400

Figure 10

MAP OF UNDERGROUND MINING ACTIVITIES IN PITTSBURGH SEAM

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Note:
Unofficial map, for information only. Scanned from Bond Map obtained from DER on January 9, 1989.

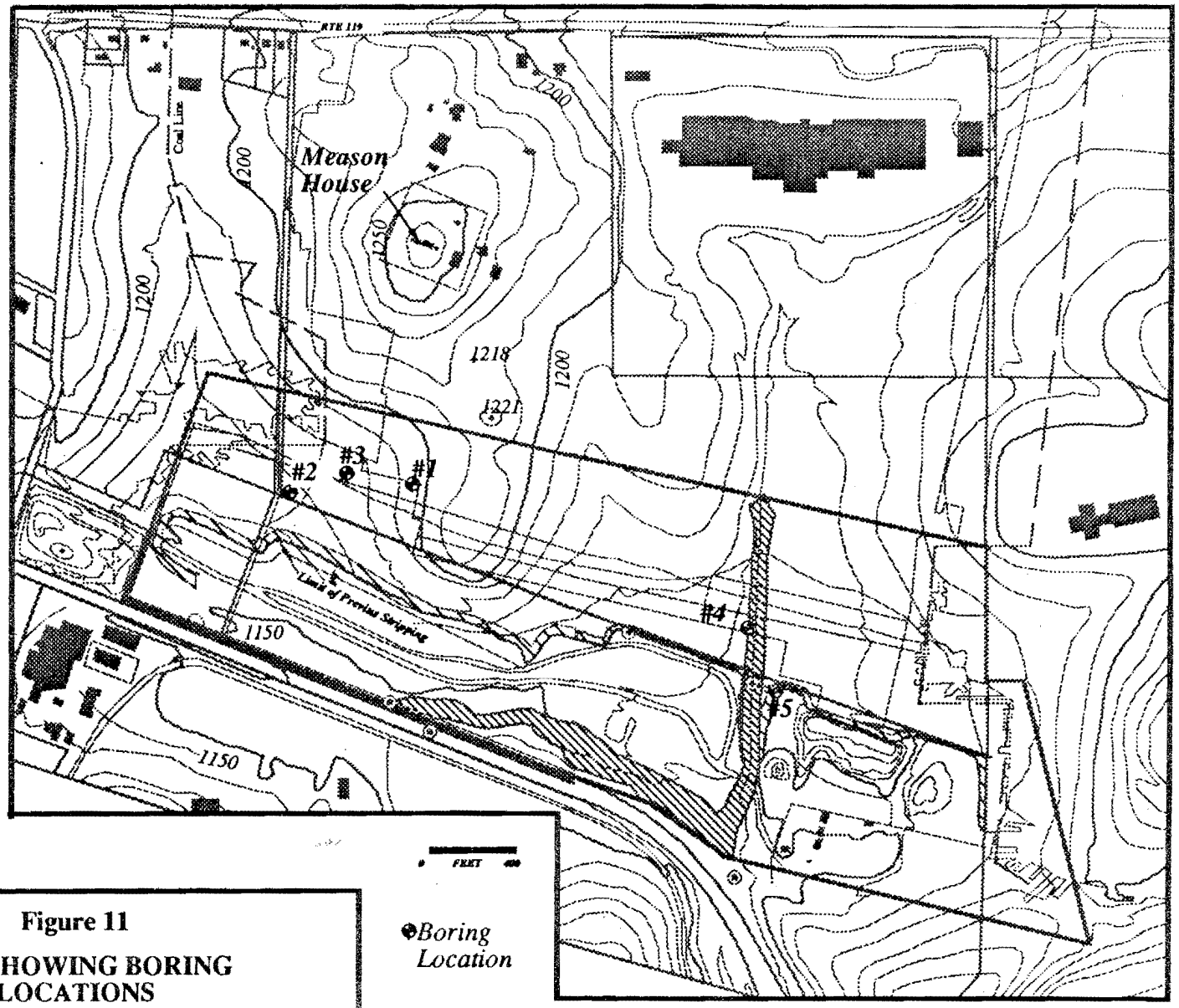

Figure 11

MAP SHOWING BORING LOCATIONS

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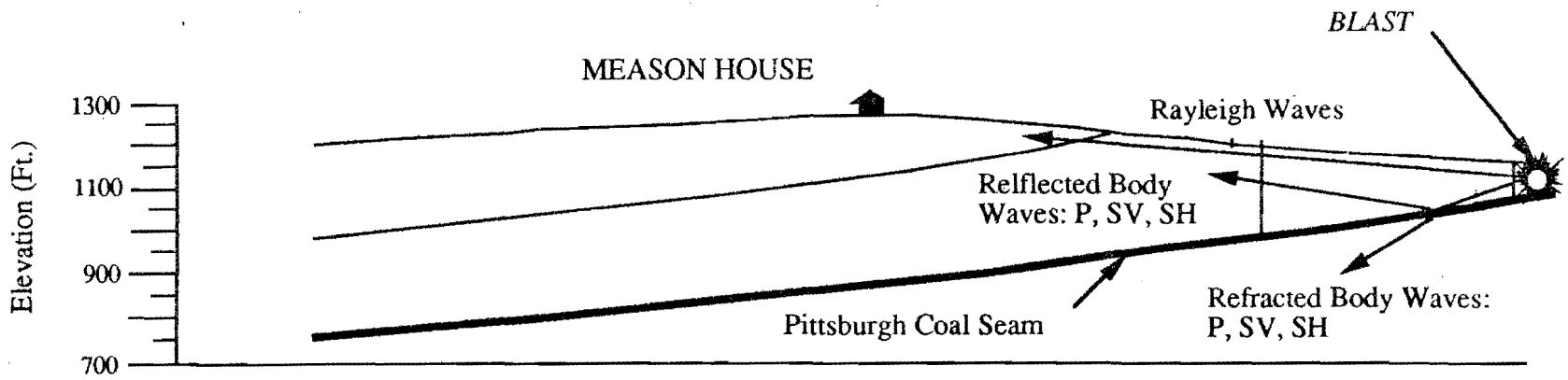
Boring Location

Note:
Unofficial map, for information only. Scanned from Bond Map obtained from DER on January 9, 1989.



SV - Wave:
Shear stress wave polarized in a vertical plane.

SH - Wave:
Shear stress wave polarized in a horizontal plane.

P - Wave:
Compression stress wave involving back and forth ground motion parallel to the direction of wave propagation.

Rayleigh Wave:
Surface wave involving particle motion in an ellipse

0 Feet 400

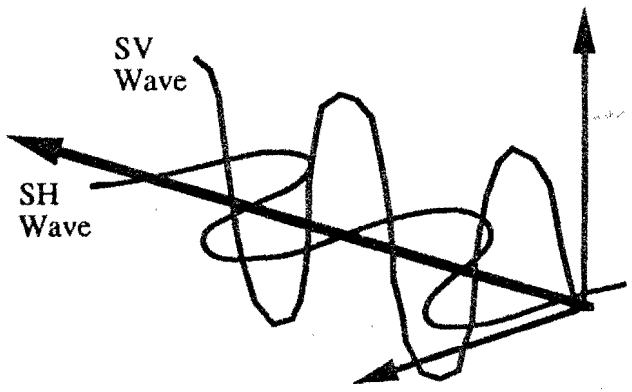


Figure 12
SECTION A-A
ILLUSTRATING STRESS WAVES
RESULTING FROM BLASTING

DONALD E. SHAW, P.E.	1/16/89

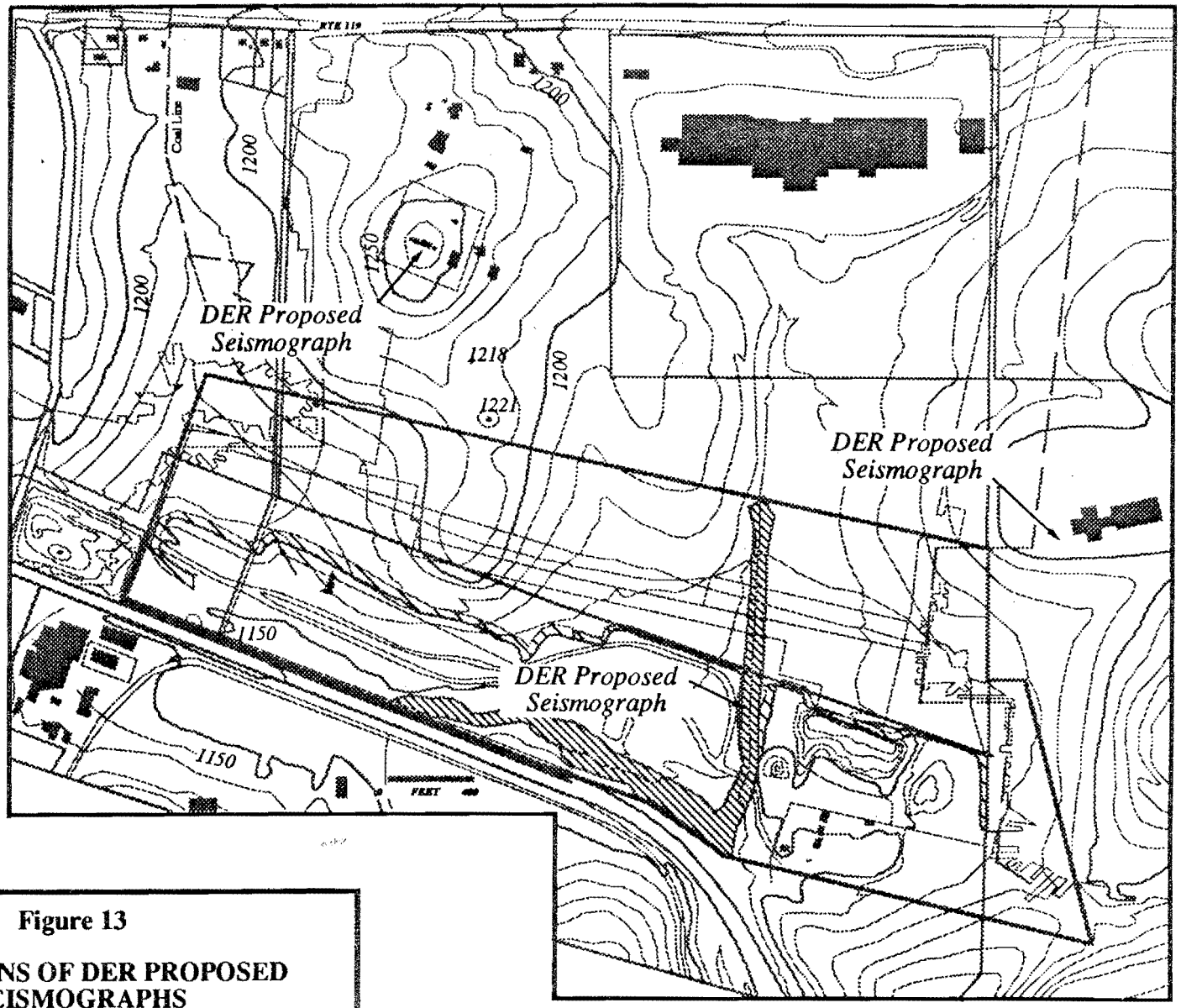


Figure 13

LOCATIONS OF DER PROPOSED SEISMOGRAPHS

DONALD E. SHAW, P.E.	1/12/89

Note:
Unofficial map, for information only. Scanned from Bond Map obtained from DER on January 9, 1989.

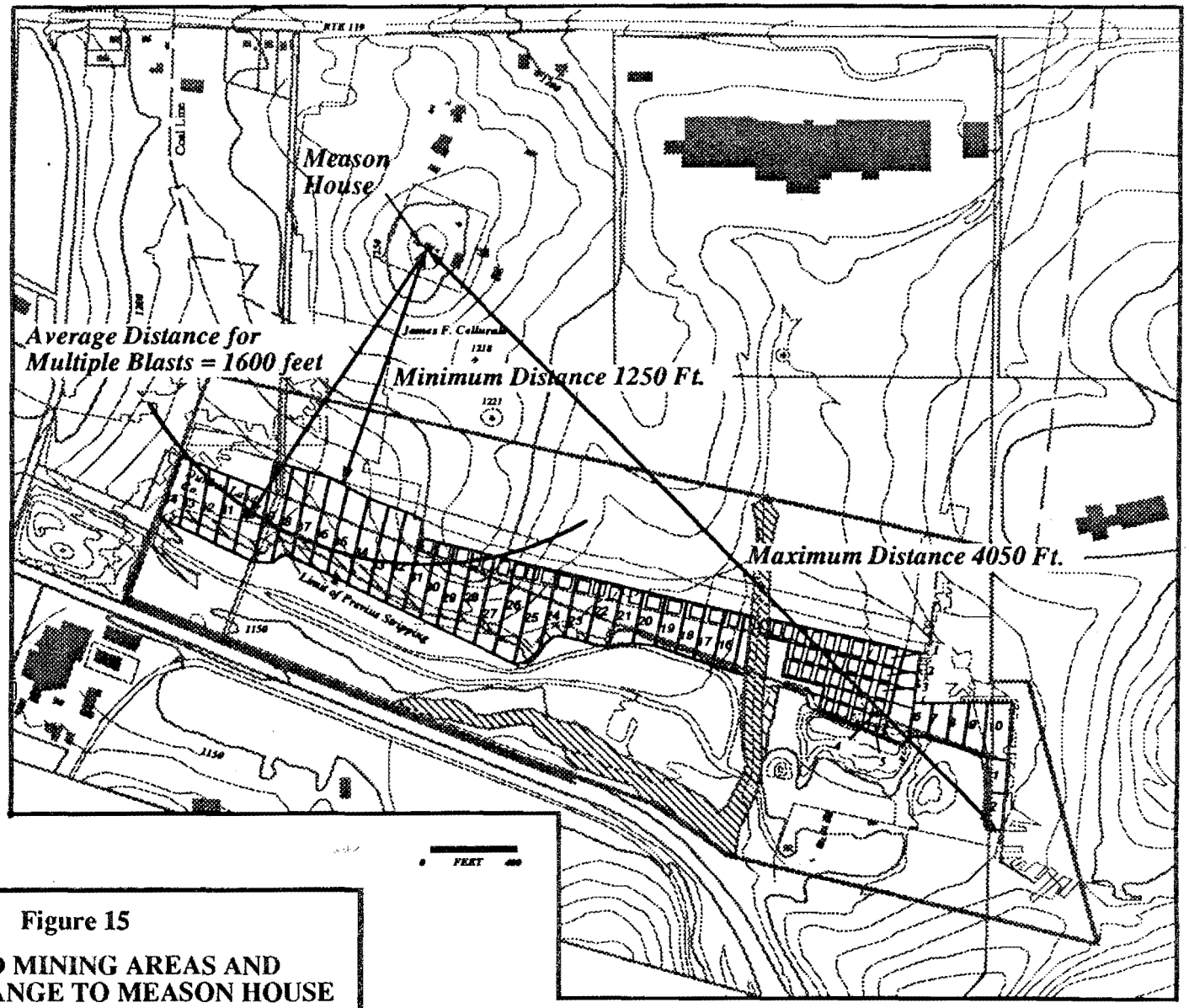
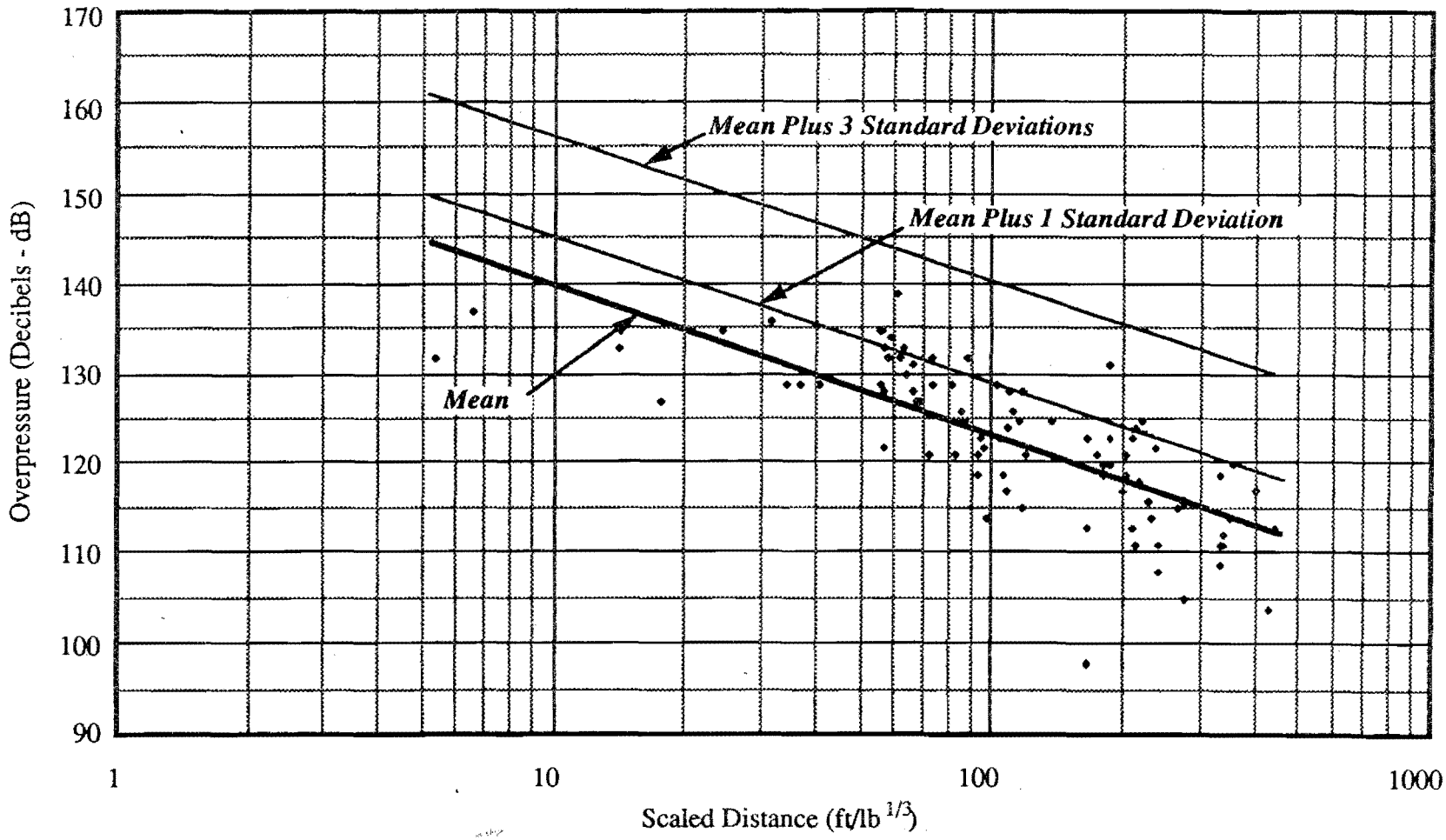


Figure 15

PLANNED MINING AREAS AND DISTANCE RANGE TO MEASON HOUSE

DONALD E. SHAW, P.E.	1/13/89

Note:
Unofficial map, for information only. Scanned from Bond Map obtained from DER on January 9, 1989. Map plan overlaid from Bond Map stamped "Received April 7, 1988.



NOTE:
Developed from data in Siskind, D.E.,et. al., **Structure Response and Damage Produced by Airblast From Surface Mining**, U. S. Bureau of Mines, RI8485, 1980, for coal mine highwall blasts.

Figure 14

AIRBLAST OVERPRESSURE VS. SCALED DISTANCE

DONALD E. SHAW, P.E.	1/16/89

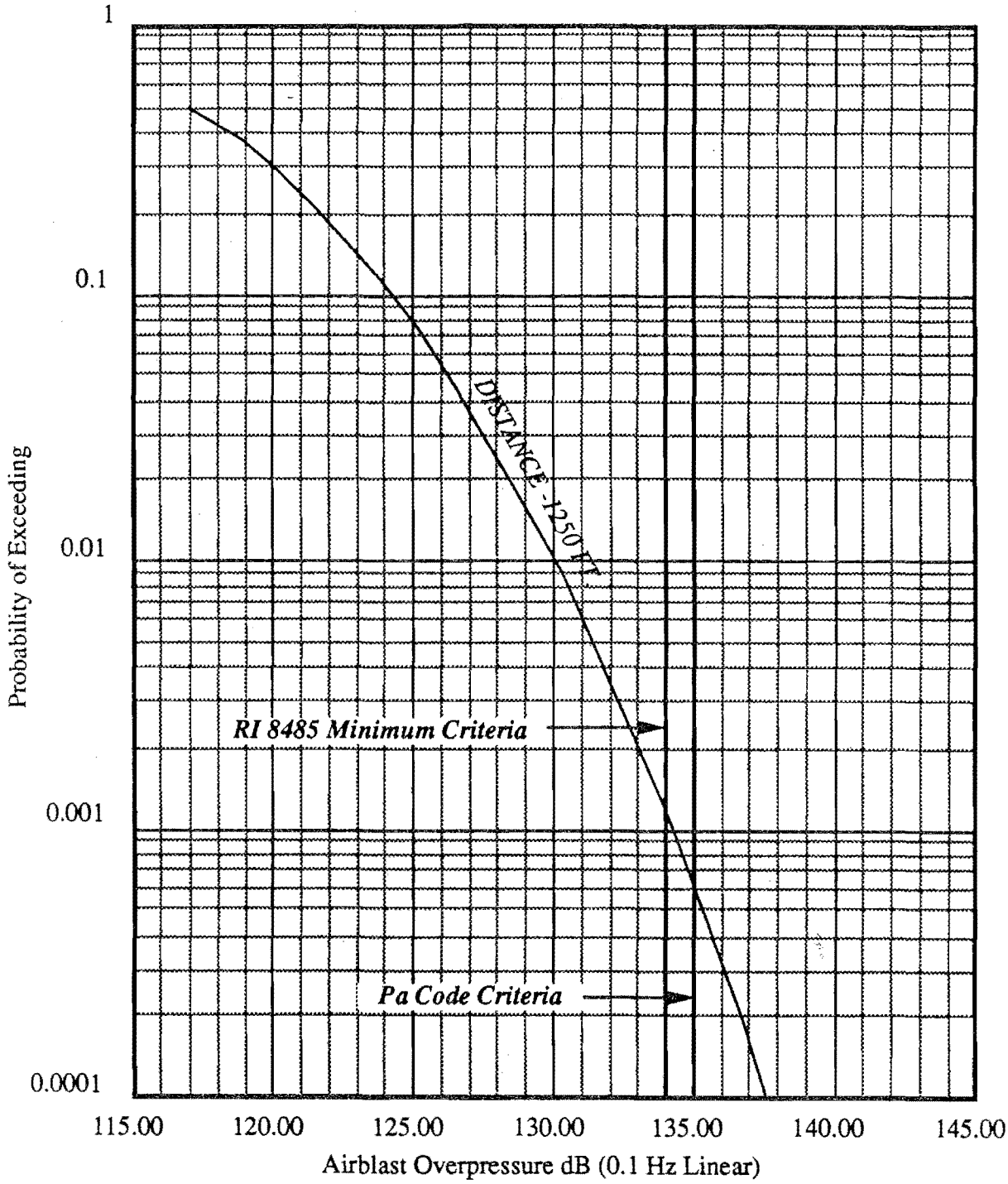


Figure 16

PROBABILITY OF PEAK AIRBLAST OVERPRESSURE FROM A SINGLE BLAST

DONALD E. SHAW, P.E.	1/23/89

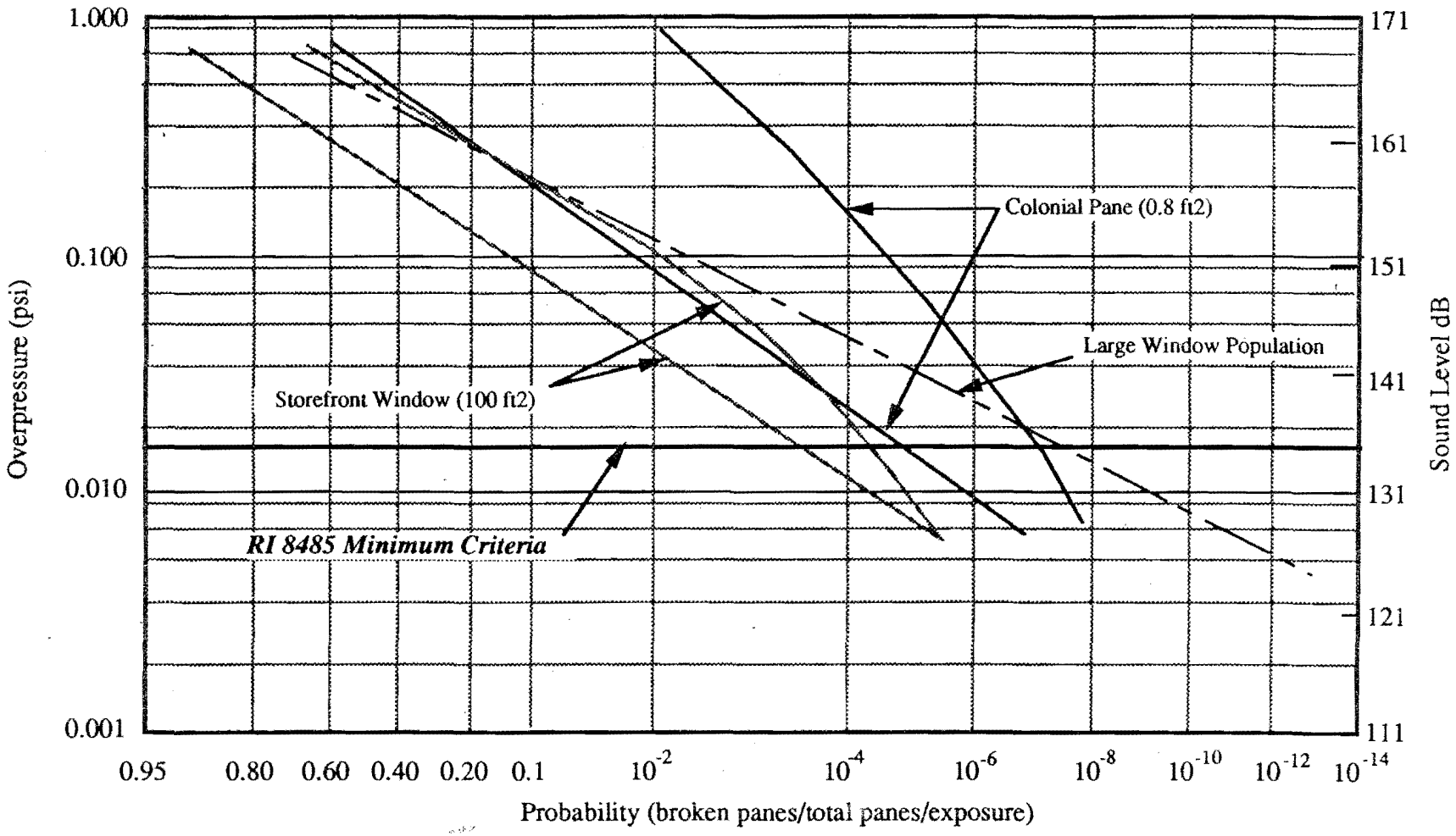


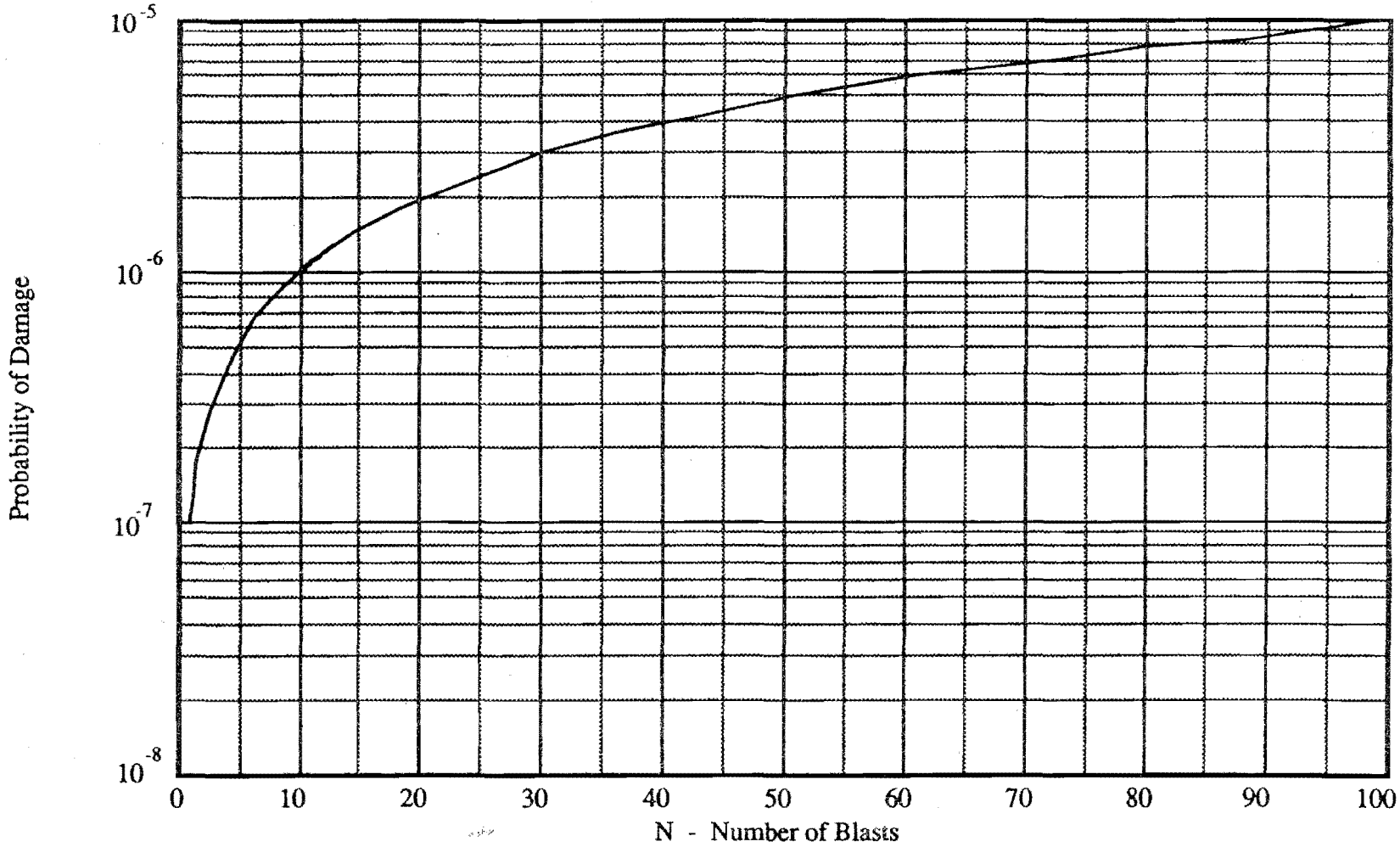
Figure 17

PROBABILITY OF DAMAGE VS. AIRBLAST OVERPRESSURE FROM A SINGLE BLAST

DONALD E. SHAW, P.E.	1/23/89

NOTES:

1. After Siskind, D. E., et al, **Structural Response and Damage Produced by Airblast from Surface Mining, RI8485**, U. S. Bureau of Mines, 1980, Figure 40



NOTE:
1. Based on binomial distribution for repeated random trials with a probability of damage for each blast of 1×10^{-7}.

Figure 18
PROBABILITY OF DAMAGE FROM AIRBLAST OVERPRESSURE VS. NUMBER OF BLASTS

DONALD E. SHAW, P.E.	1/23/89

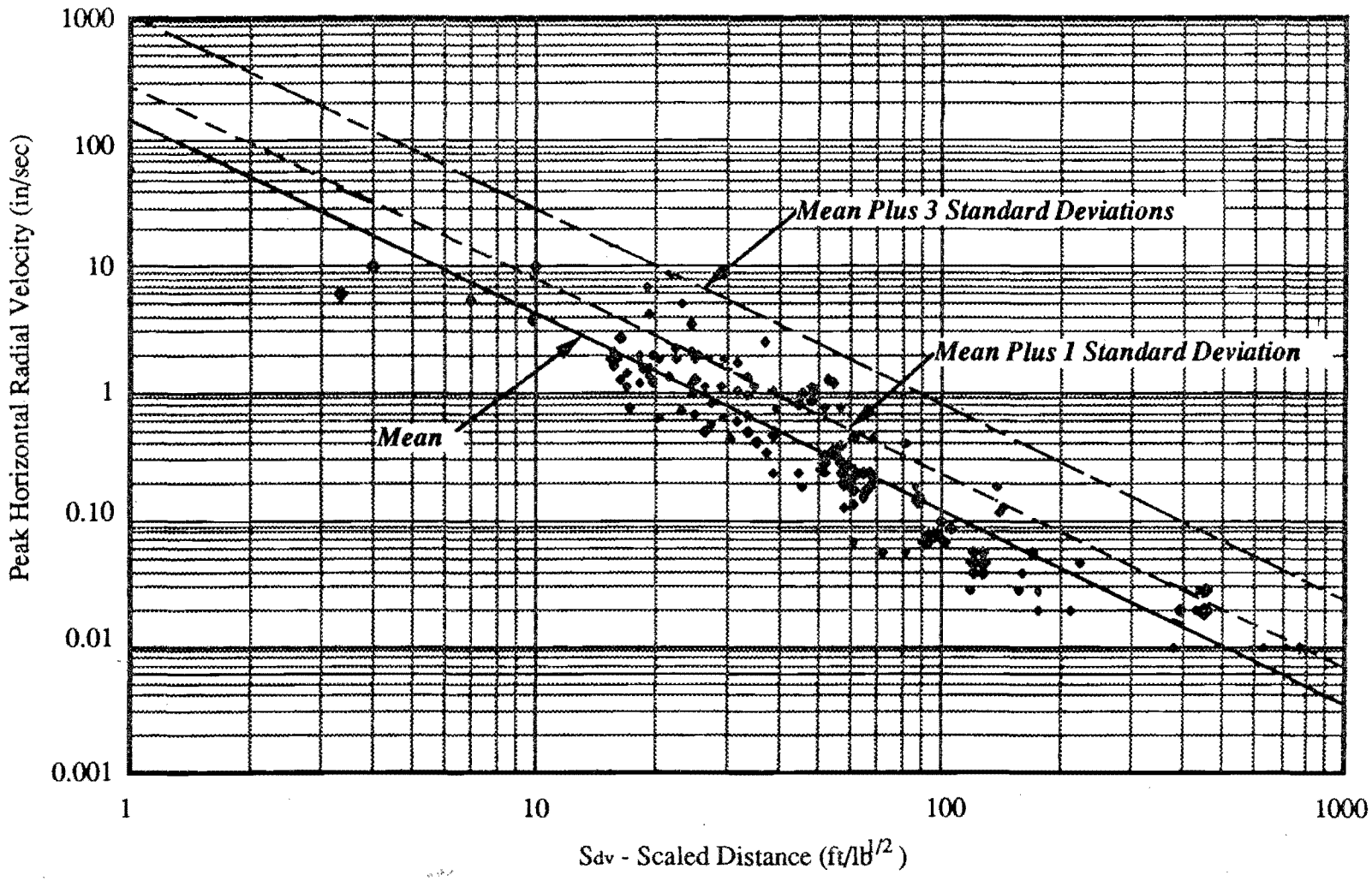


Figure 19

PEAK HORIZONTAL RADIAL VELOCITY VS. SCALED DISTANCE

DONALD E. SHAW, P.E.	1/9/89

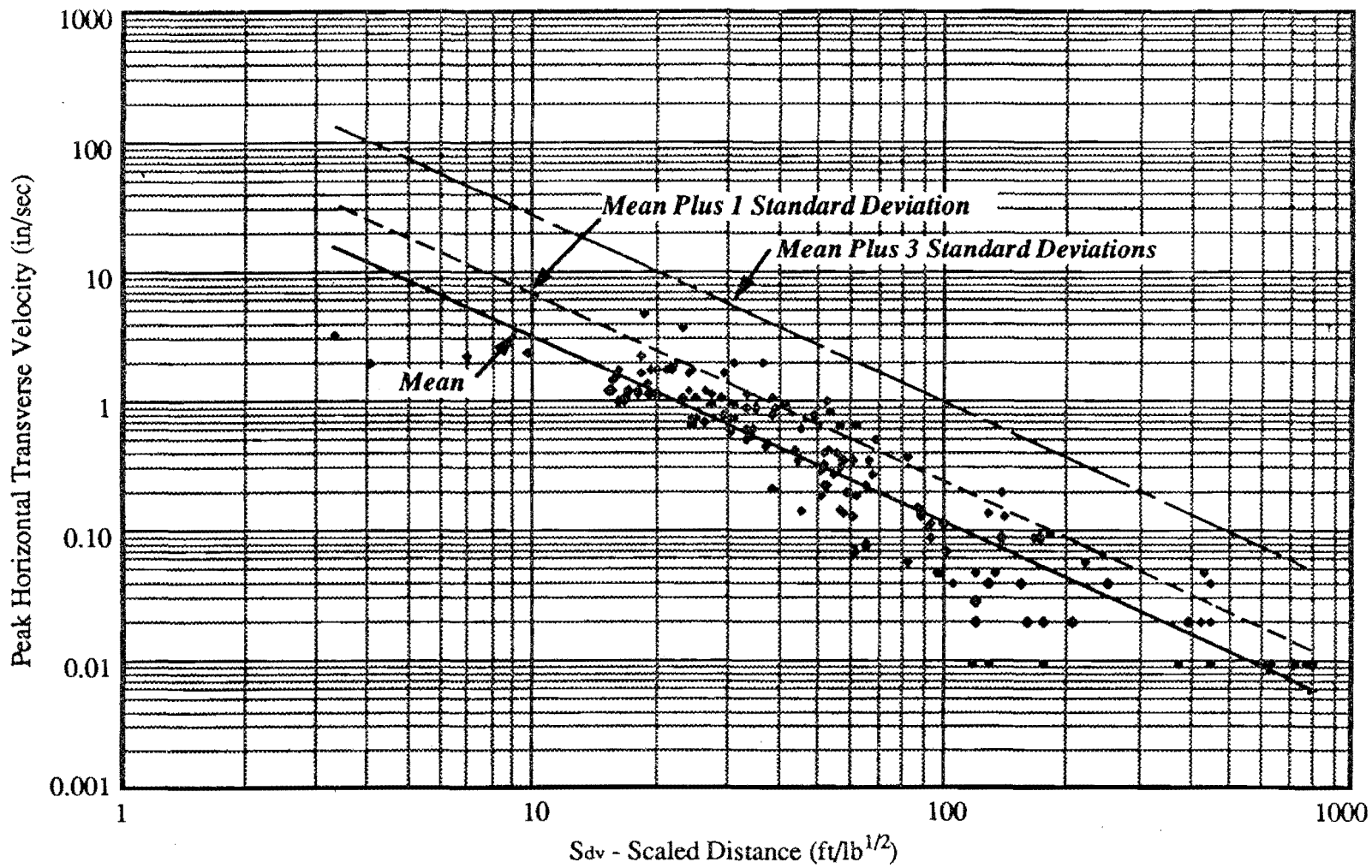


Figure 20

PEAK HORIZONTAL TRANSVERSE VELOCITY VS. SCALED DISTANCE

DONALD E. SHAW, P.E.	1/9/89

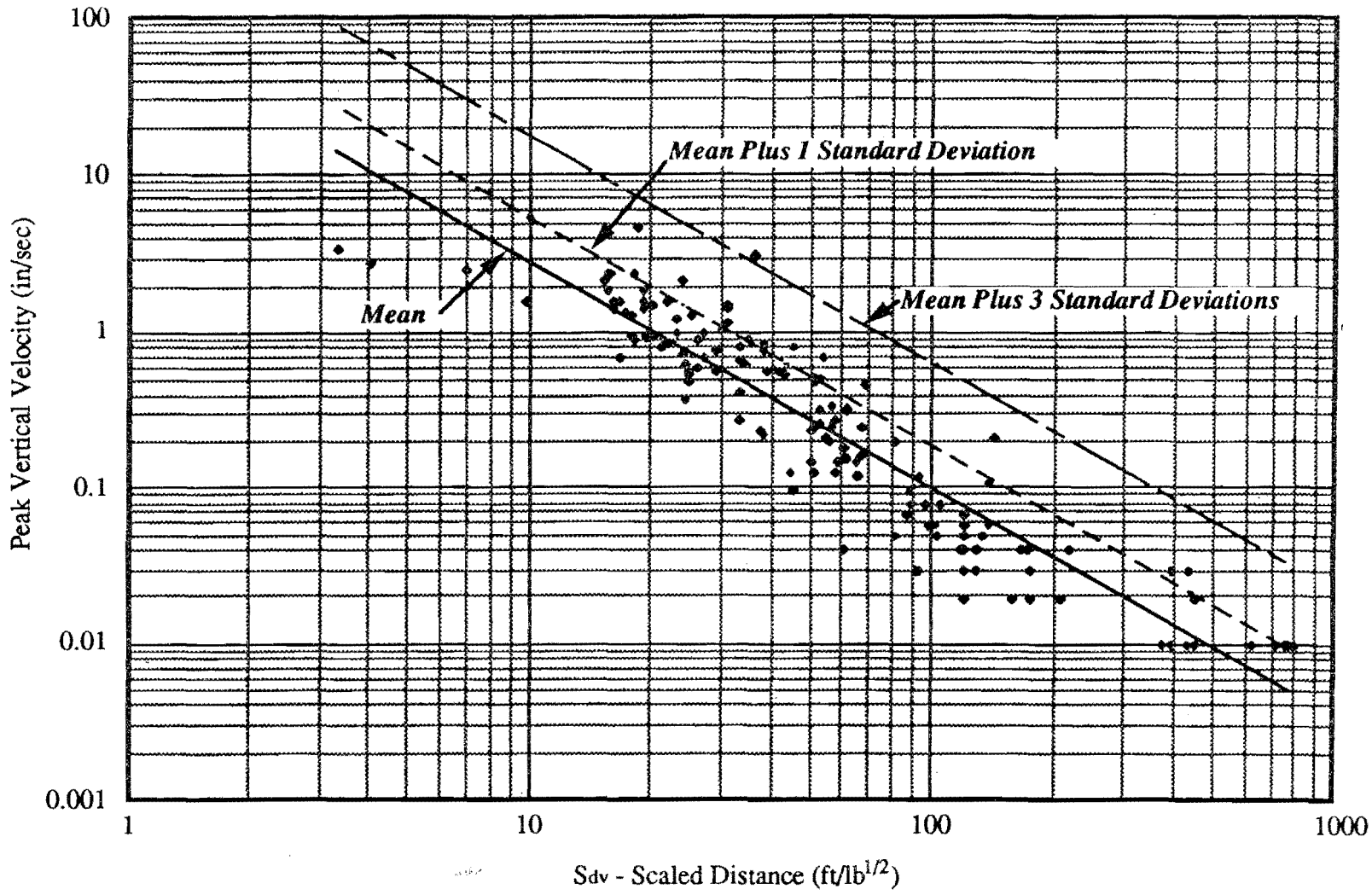
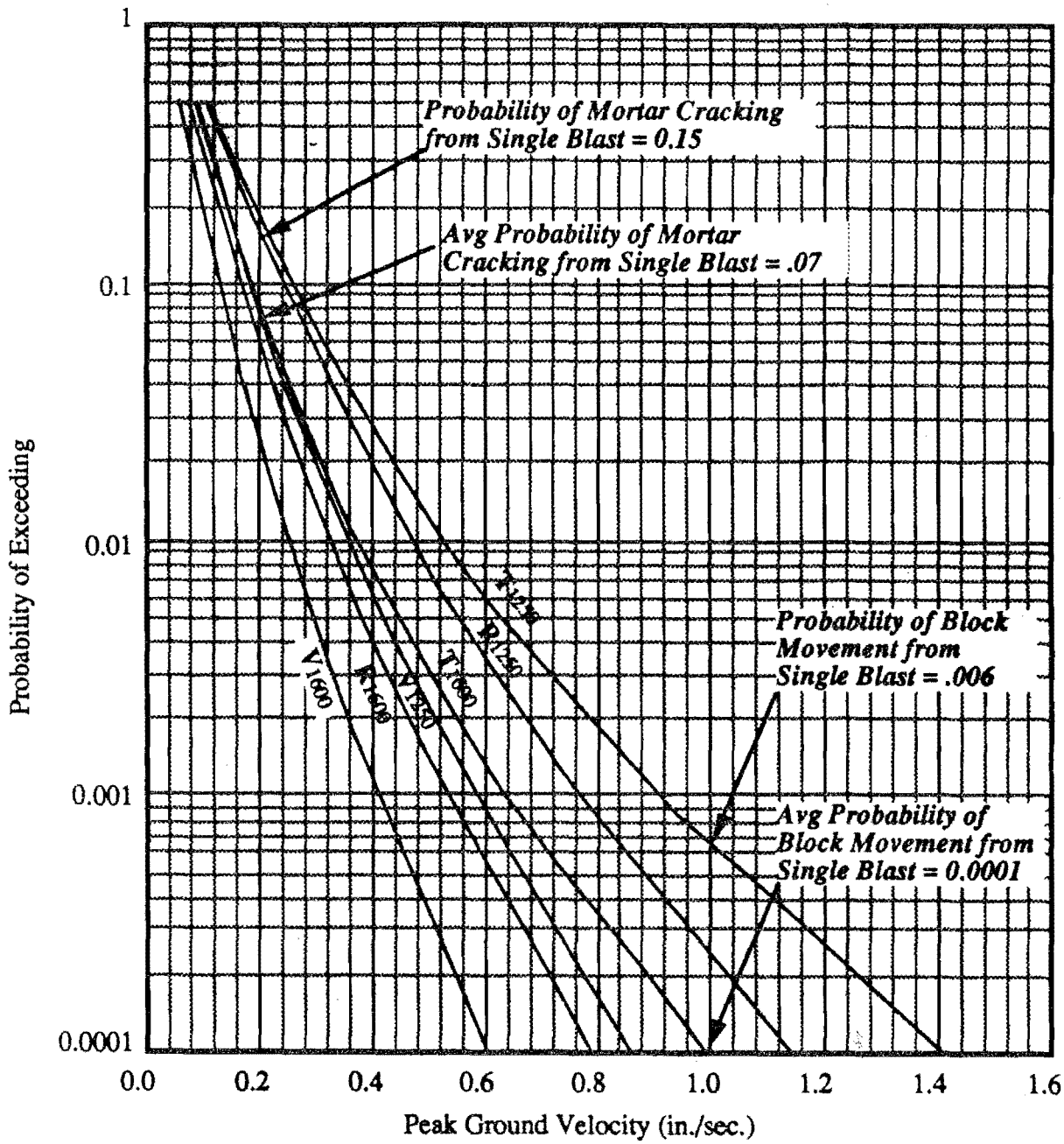


Figure 21

PEAK VERTICAL VELOCITY VS. SCALED DISTANCE

DONALD E. SHAW, P.E.	1/9/89



LEGEND
R1250 = Radial Velocity at 1250 Feet From Blast
T1250 = Transverse Velocity at 1250 Feet From Blast
V1250 = Vertical Velocity at 1250 Feet From Blast
R1600 = Radial Velocity 1600 Feet From Blast
T1600 = Transverse Velocity 1600 Feet From Blast
V1600 = Vertical Velocity 1600 Feet From Blast

Figure 22
PROBABILITY OF PEAK GROUND VELOCITY FROM A SINGLE BLAST

DONALD E. SHAW, P.E.	1/9/89

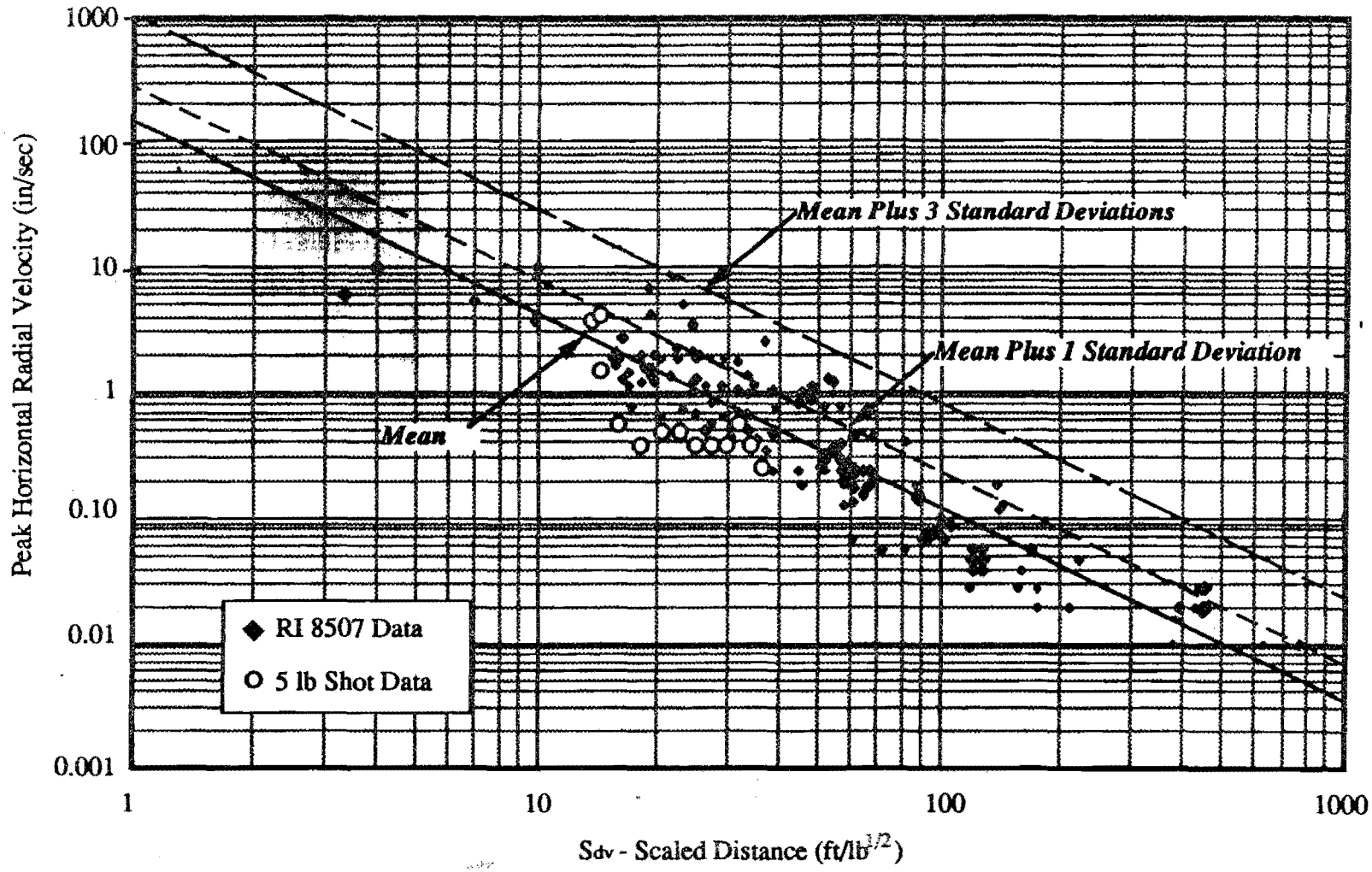


Figure 23

5 LB SHOT DATA COMPARISON HORIZONTAL RADIAL VELOCITY

DONALD E. SHAW, P.E.	2/3/89

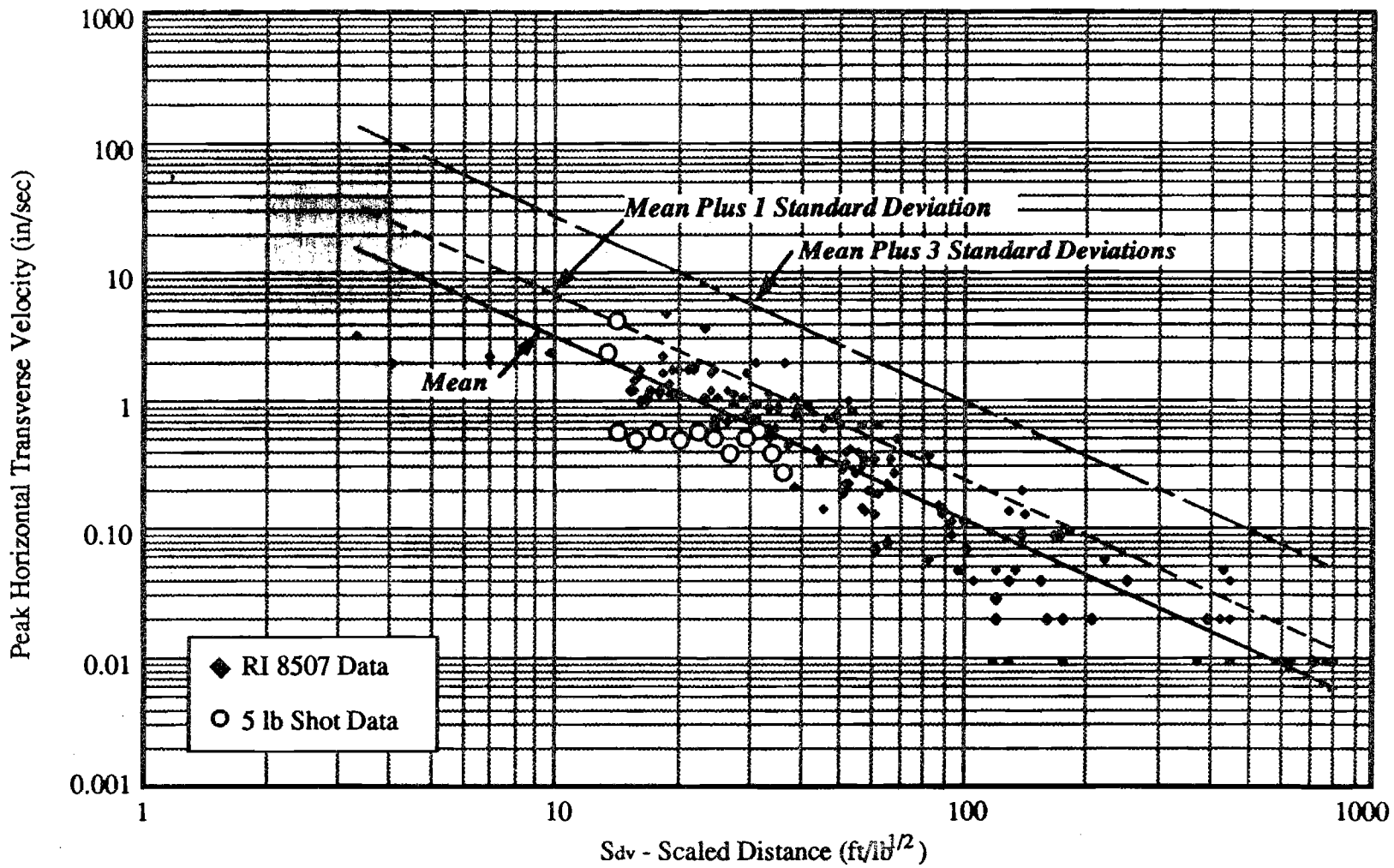


Figure 24
5 LB SHOT DATA COMPARISON HORIZONTAL TRANSVERSE VELOCITY

DONALD E. SHAW, P.E.	2/3/89

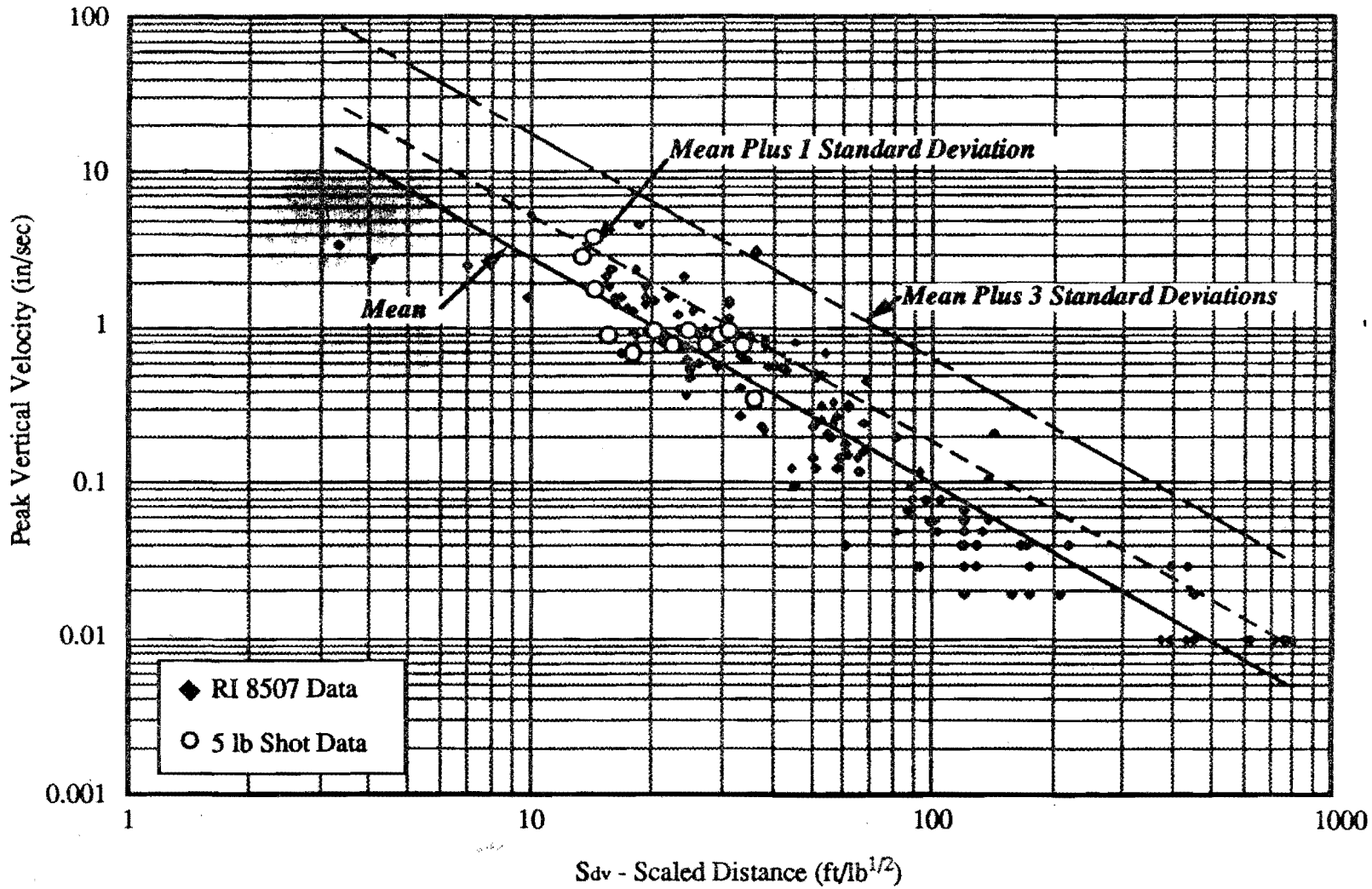
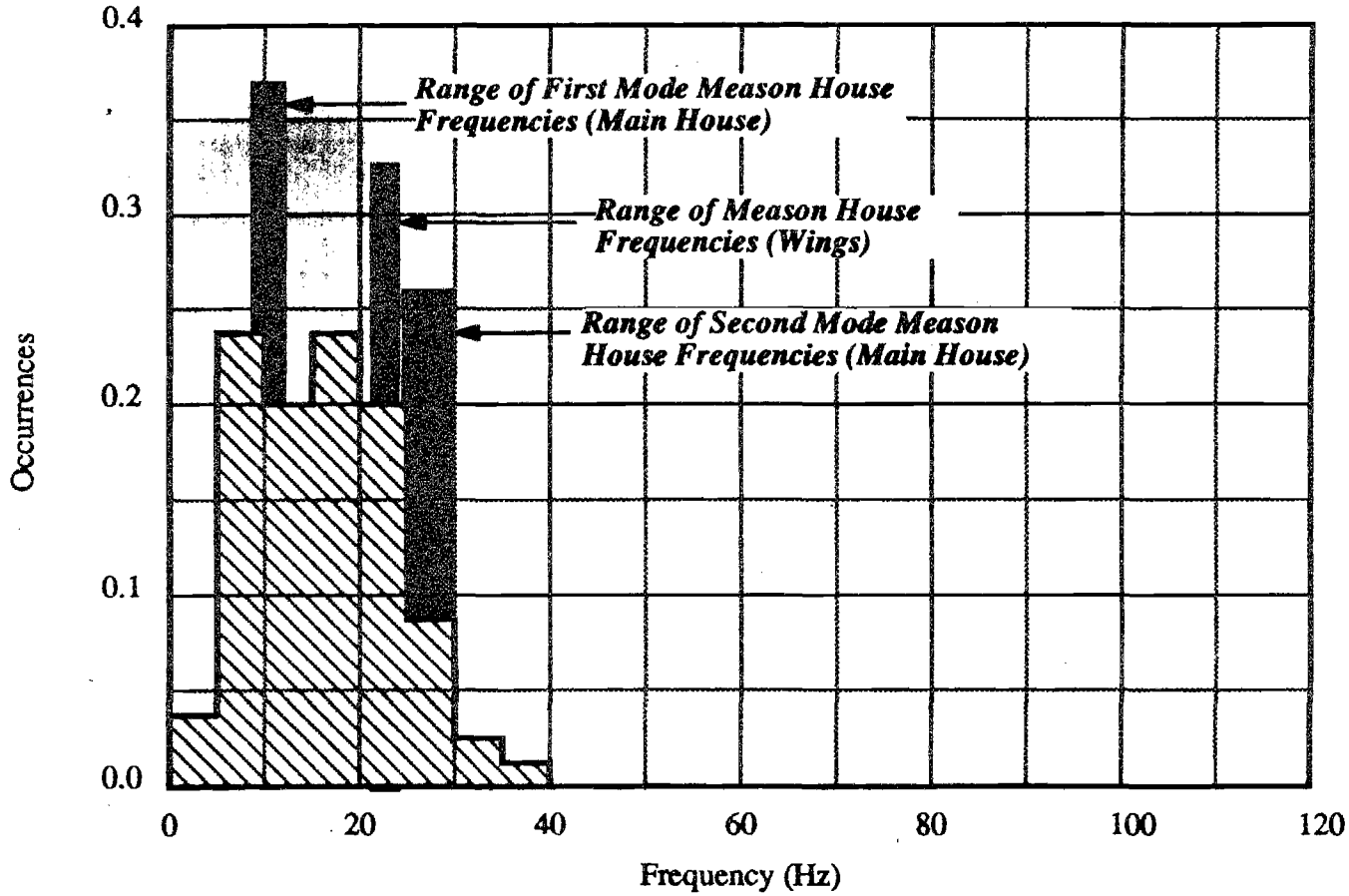


Figure 25

5 LB SHOT DATA COMPARISON VERTICAL VELOCITY

DONALD E. SHAW, P.E.	1/9/89

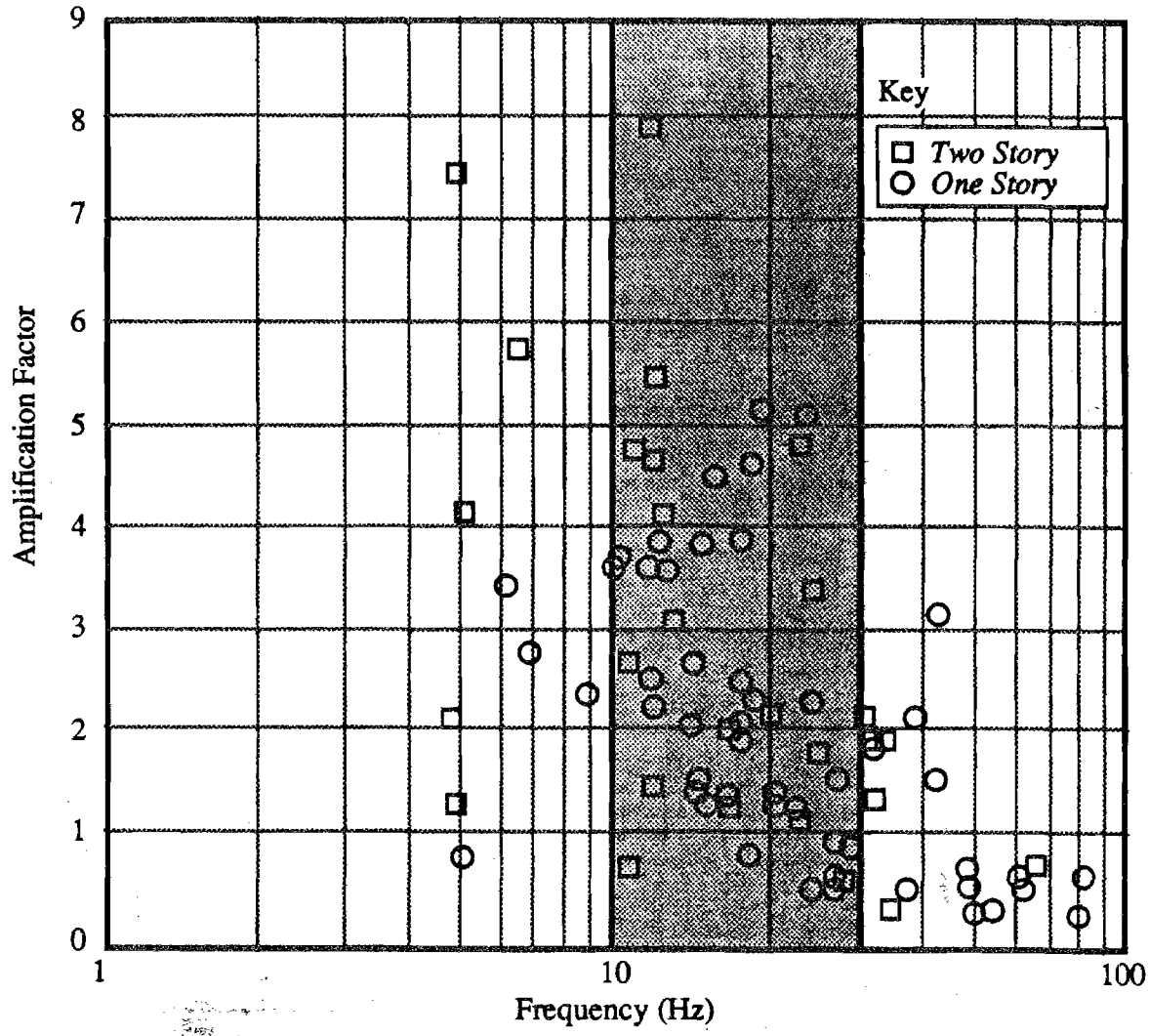


NOTE:
Histogram taken from Figure 10, Stagg, M. S. and A. J. Engler, Measurement of **Blast Induced Ground Vibrations and Seismograph Calibration**, U.S. Bureau of Mines, RI 8506, 1980

Figure 26

HISTOGRAM OF BLAST VIBRATIONS FROM COAL MINES

DONALD E. SHAW, P.E.	1/12/89



NOTES:

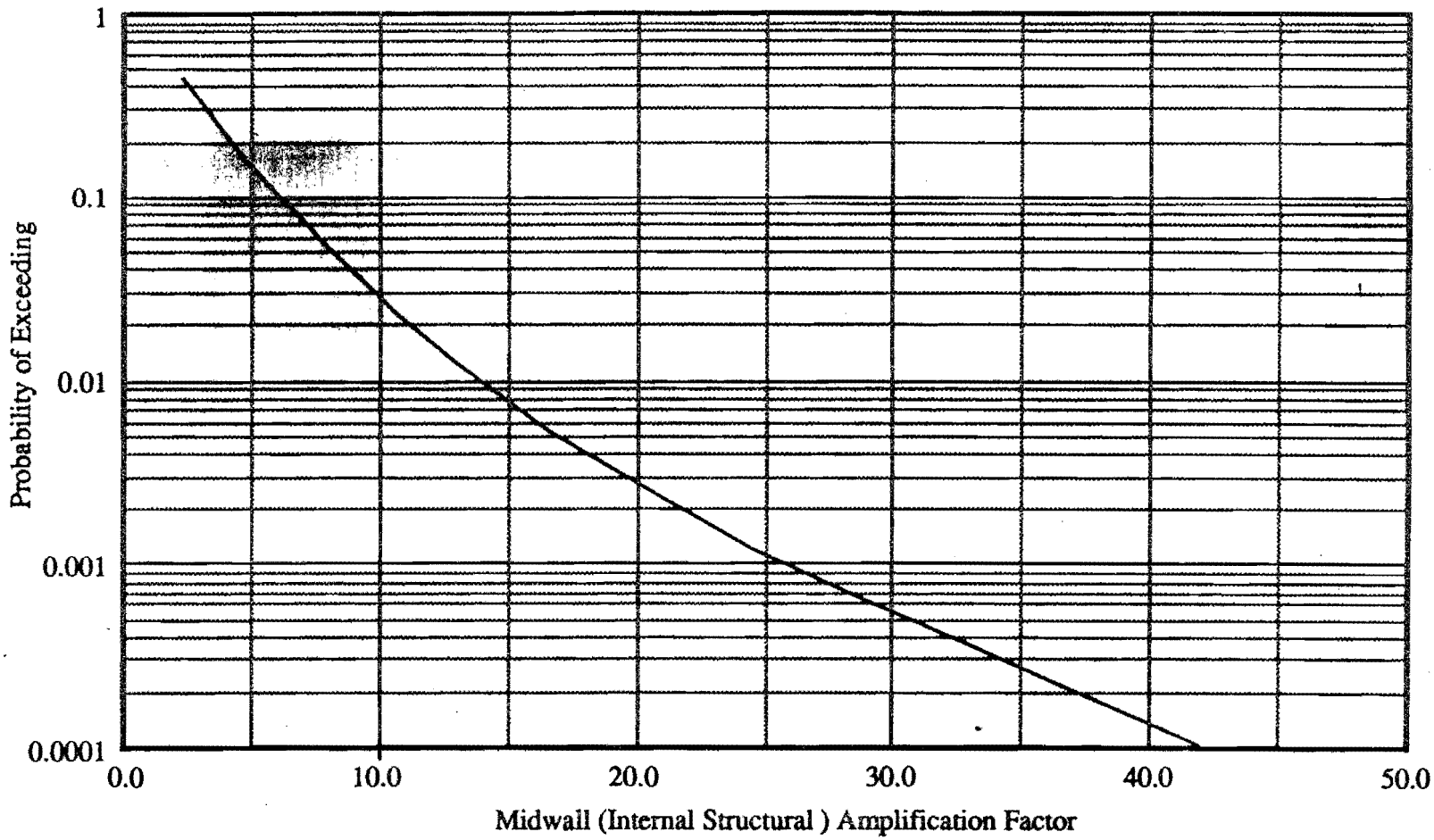
1. Reproduced from Figure 40, Siskind, D. E., et. al., **Structure Response and Damage by Ground Vibration from Surface Mine Blasting**, U. S. Bureau of Mines, RI 8507, 1980.

2. Shaded areas added and represent frequency ranges for the Meason House based on analysis.

Figure 27

AMPLIFICATION FACTORS FOR MID-WALL VIBRATION

DONALD E. SHAW, P.E.	**1/23/89**



NOTES:

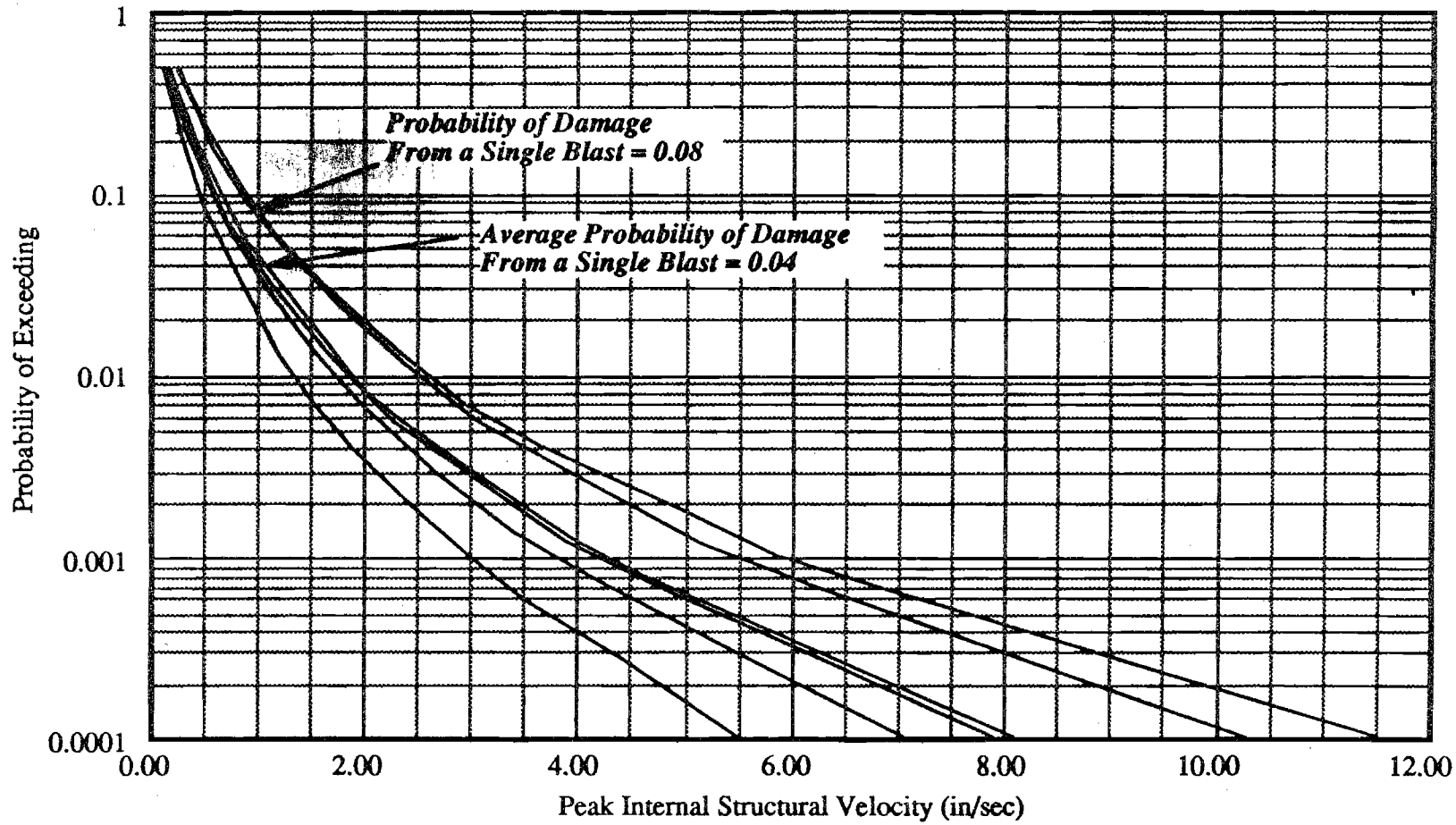
1. Amplification Factor is multiplied time peak ground velocity to obtain peak structural velocity and accounts for flexibility of internal elements of the house.

2. Based on data from RI 8507 for midwall amplification factors in frequency range from 10 to 20 Hz for two-story structures.

Figure 28

PROBABILITY DISTRIBUTION MIDWALL AMPLIFICATION FACTOR

DONALD E. SHAW, P.E.	2/2/89



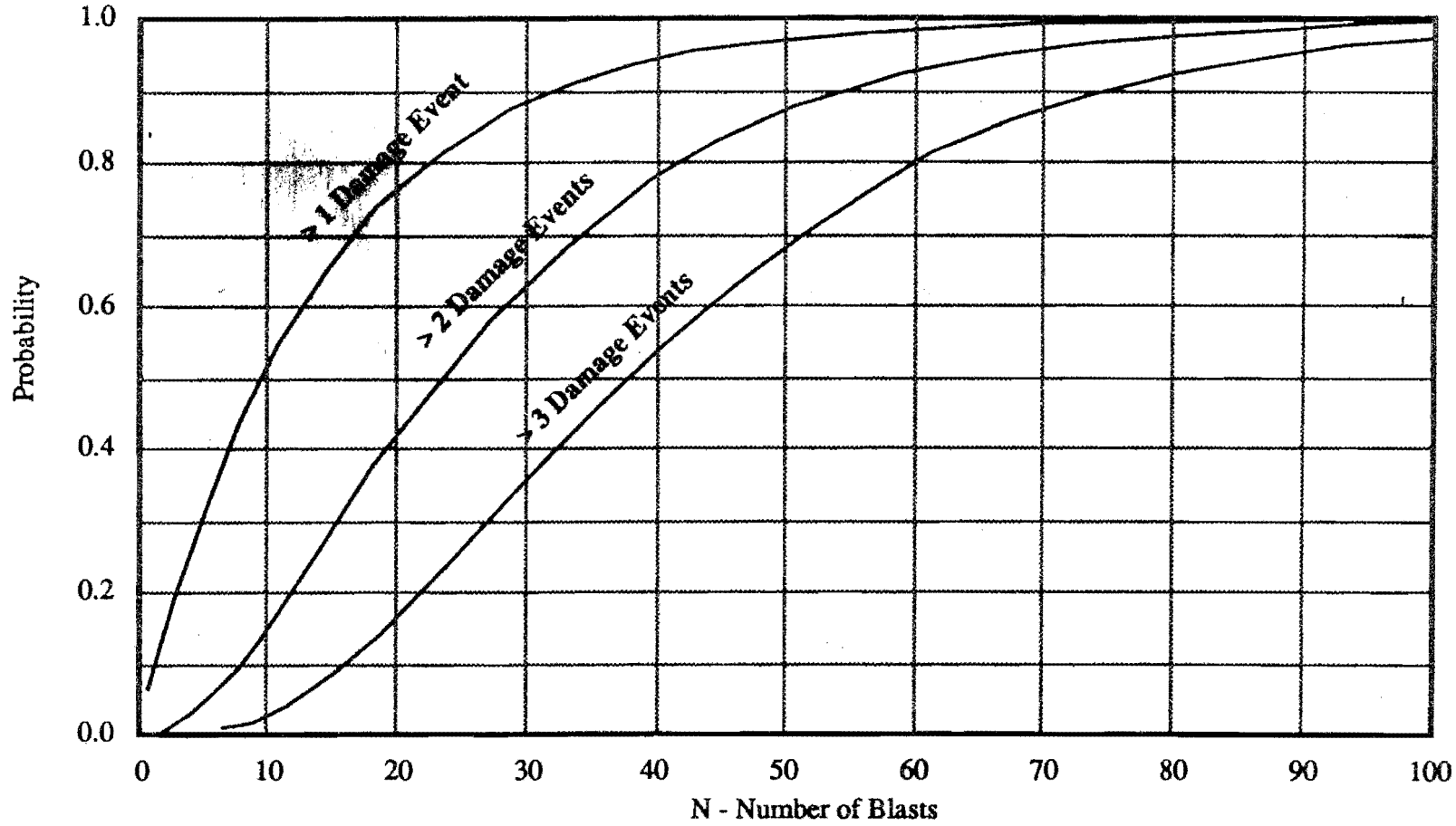
NOTES:

1. Internal structural velocity is obtained from the ground velocity times the structural amplification factor. It accounts for greater flexibility of non-structural internal systems such as plaster and lathe, internal walls and ceilings, etc.

Figure 29

PROBABILITY DISTRIBUTION OF PEAK INTERNAL STRUCTURAL VELOCITY

DONALD E. SHAW, P.E.	2/2/89

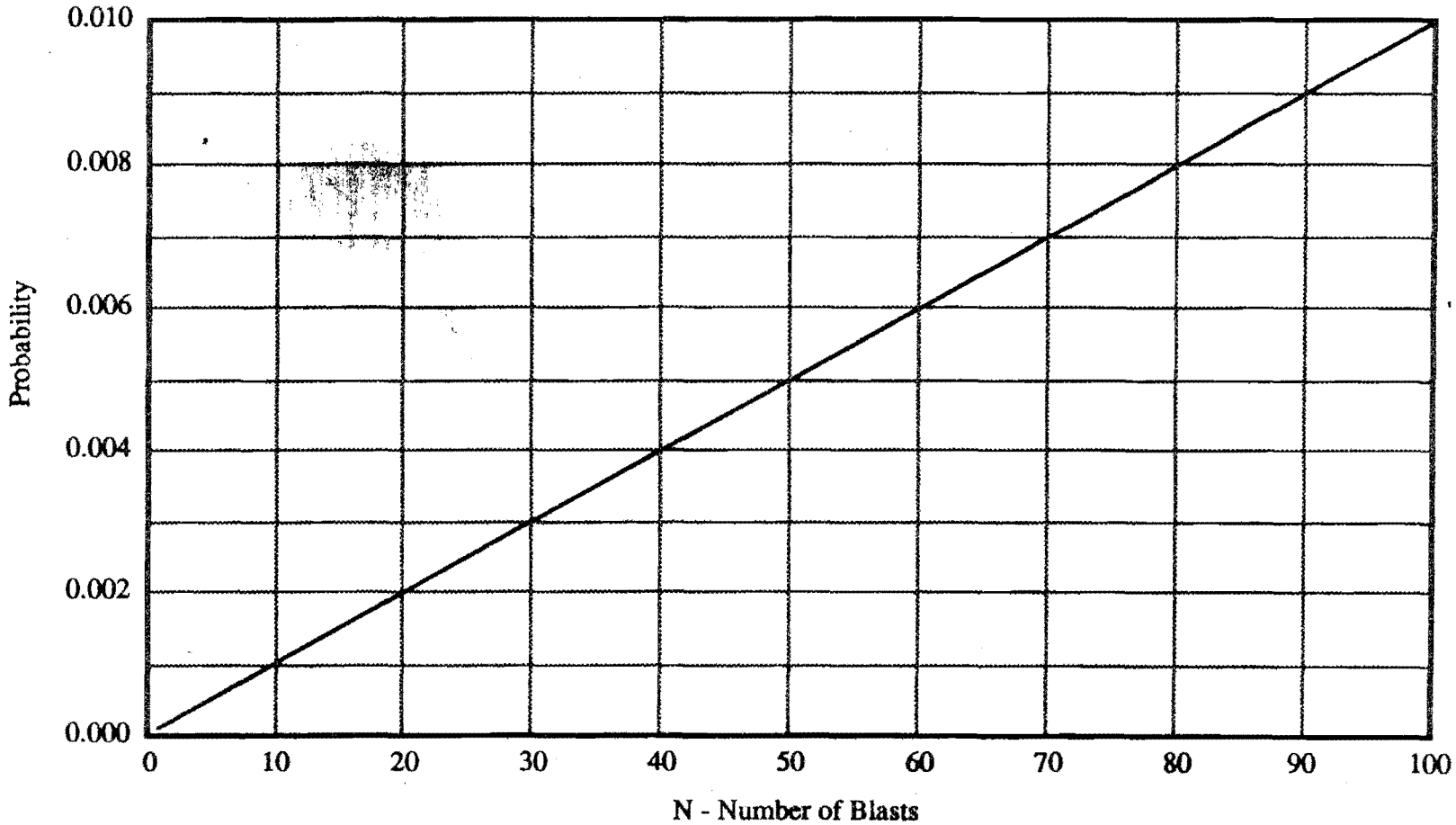


NOTE:
Based on a Binomial Distribution assuming each blast is indepedent with an average probability of damage of 0.07 based on the average distance of 1600 feet.

Figure 30

MORTAR CRACKING PROBABILITY VS. NUMBER OF BLASTS

DONALD E. SHAW, P.E.	2/3/89

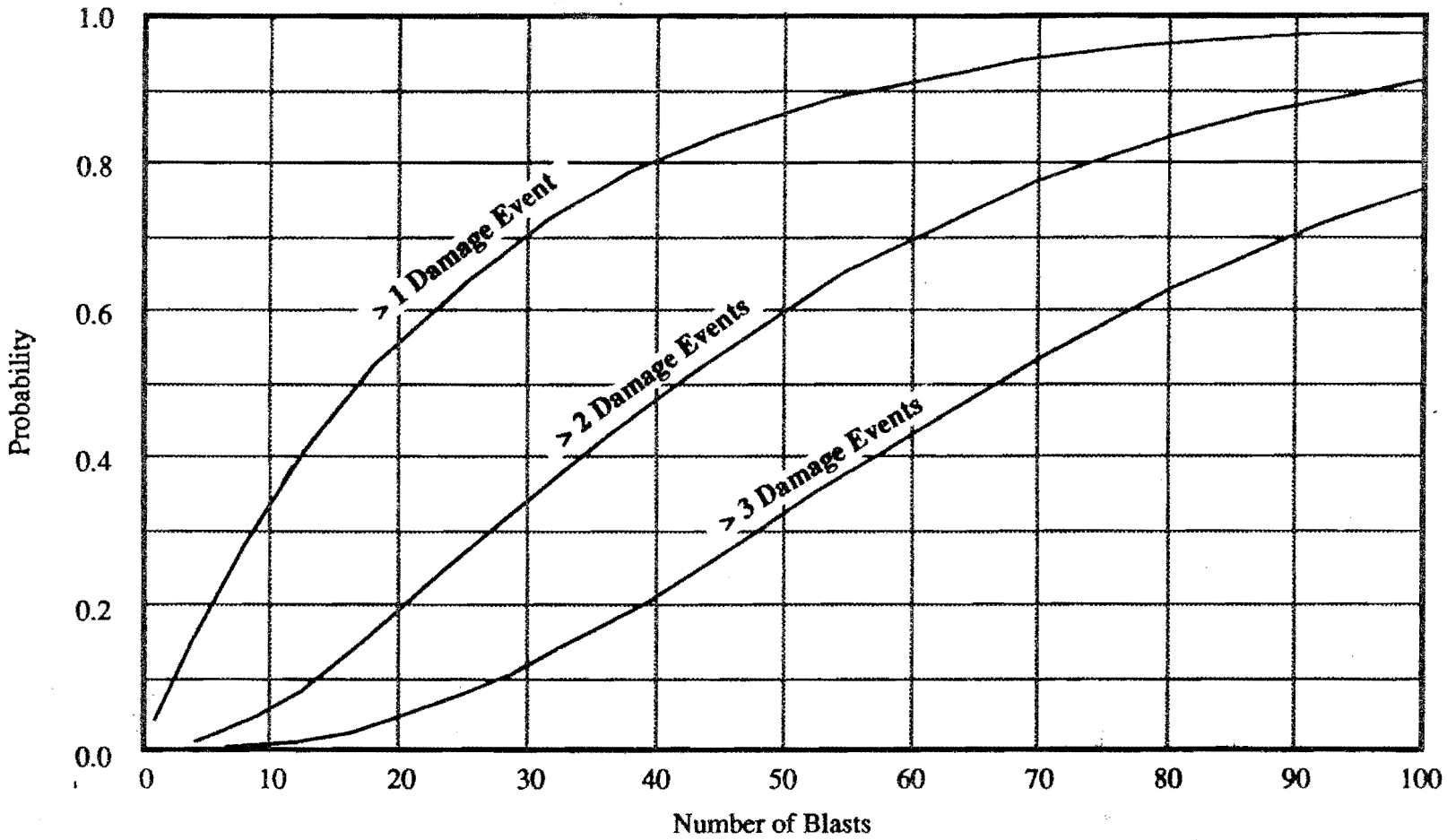


NOTE:
Based on a Binomial Distribution assuming each blast is indepedent with an average probability of damage of 0.0001 based on the average distance of 1600 feet.

Figure 31

LIMESTONE BLOCK SLIDING PROBABILITY VS. NUMBER OF BLASTS

DONALD E. SHAW, P.E.	2/3/89



NOTE:
Based on a Binomial Distribution assuming each blast is independent with a probability of damage of 0.04 based on the distance of 1600 feet

Figure 32

PLASTER DAMAGE PROBABILITY VS. NUMBER OF BLASTS

DONALD E. SHAW, P.E.	2/3/89

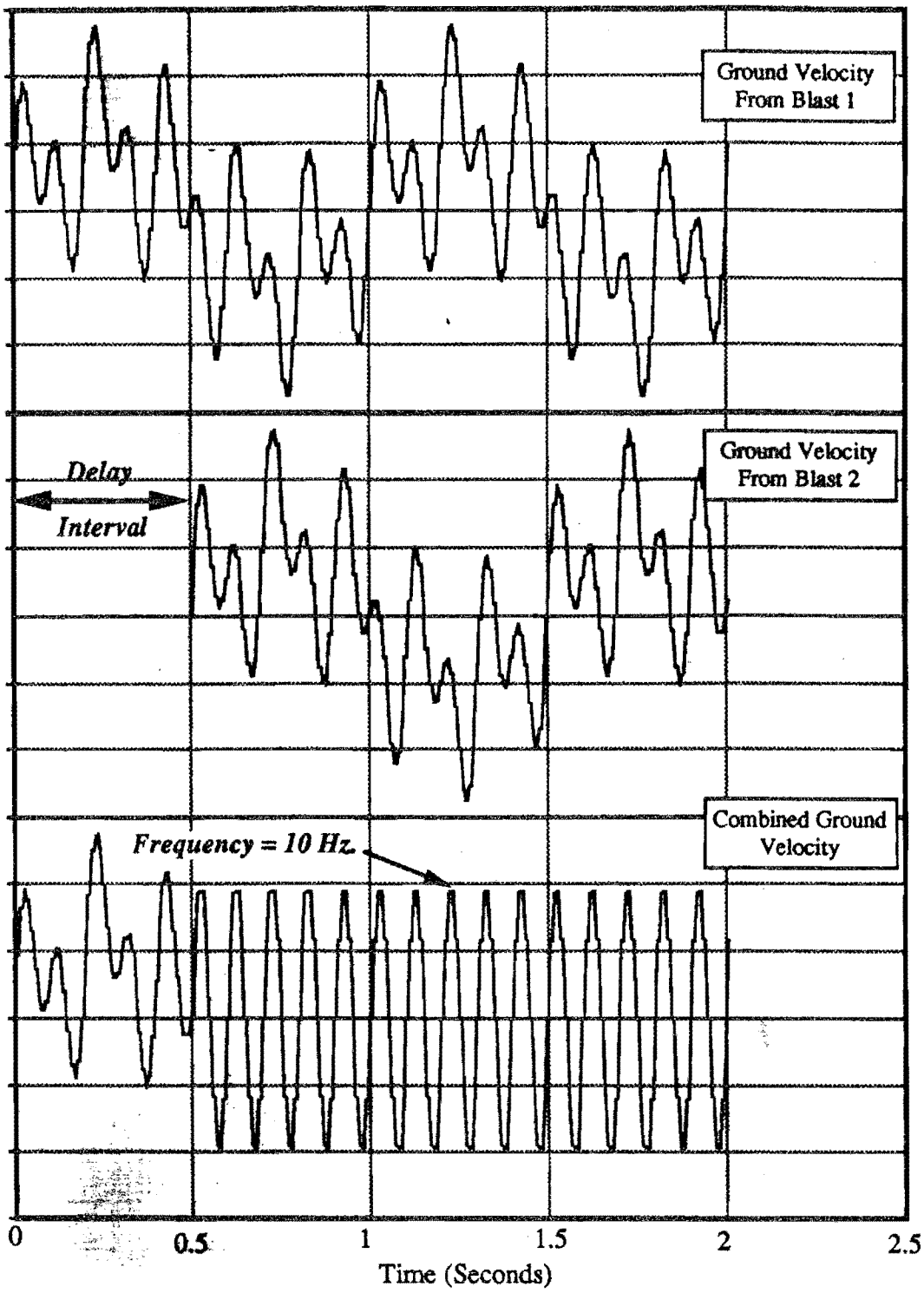
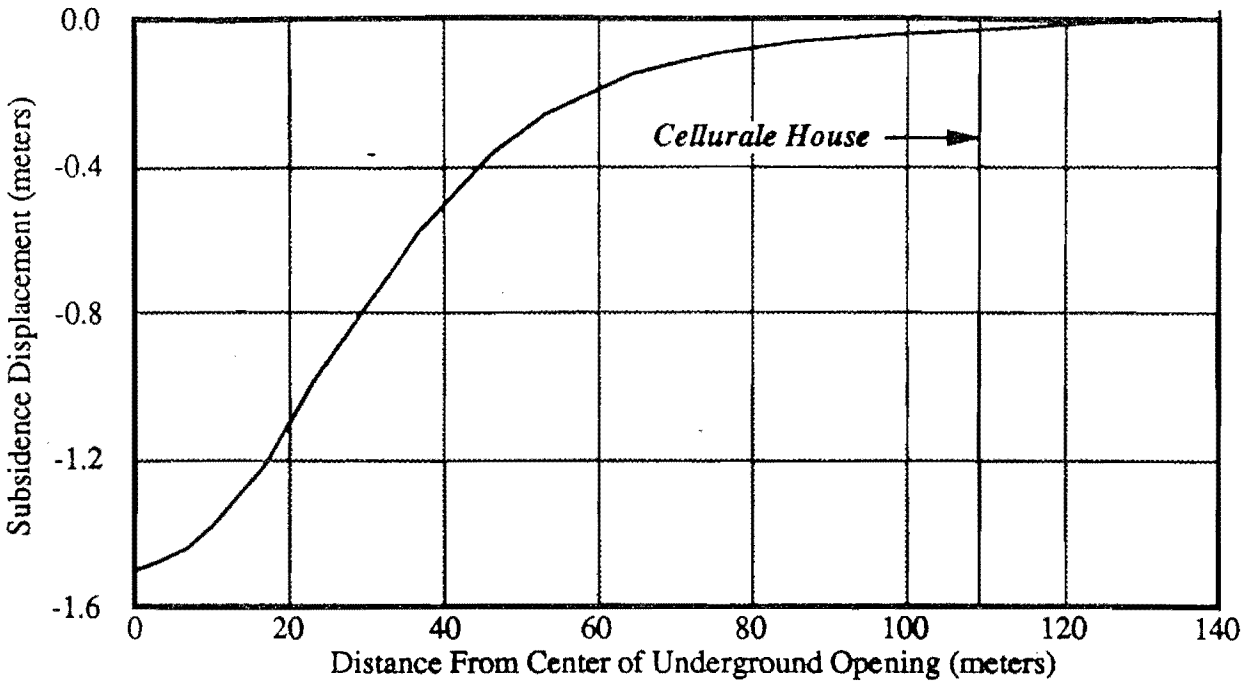


Figure 33

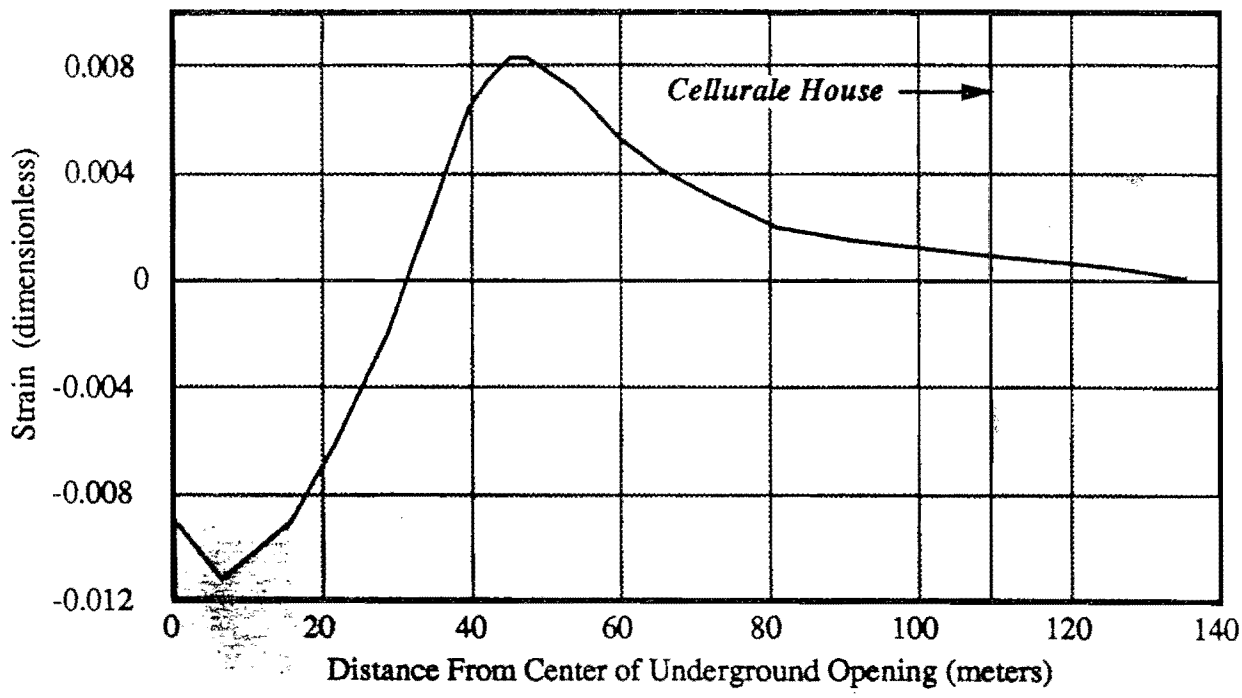
EXAMPLE OF BLAST WAVE DELAY SUPERPOSITION

NOTE:
Waveforms are artificial to simulate a blast wave for illustration only. Actual waveforms are more complex.

DONALD E. SHAW, P.E.	1/9/89



(A) SUBSIDENCE DISPLACEMENT



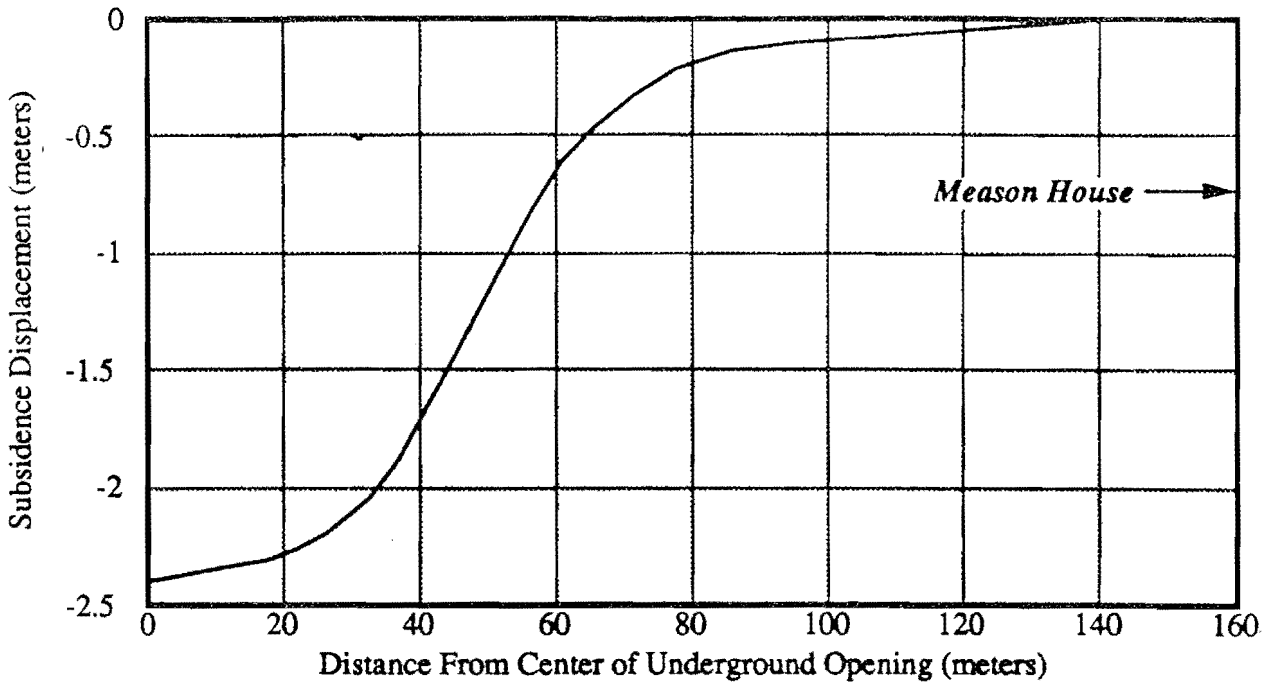
(B) SUBSIDENCE STRAIN

NOTE:
Panel width = 85.7 m
Panel depth = 134.1 m
Seam thickness = 3 m
Determined by graphical method given in Peng, S.S., **Coal Mine Ground Control**, Wiley Interscience, New York, 1978 after method of National Coal Board.

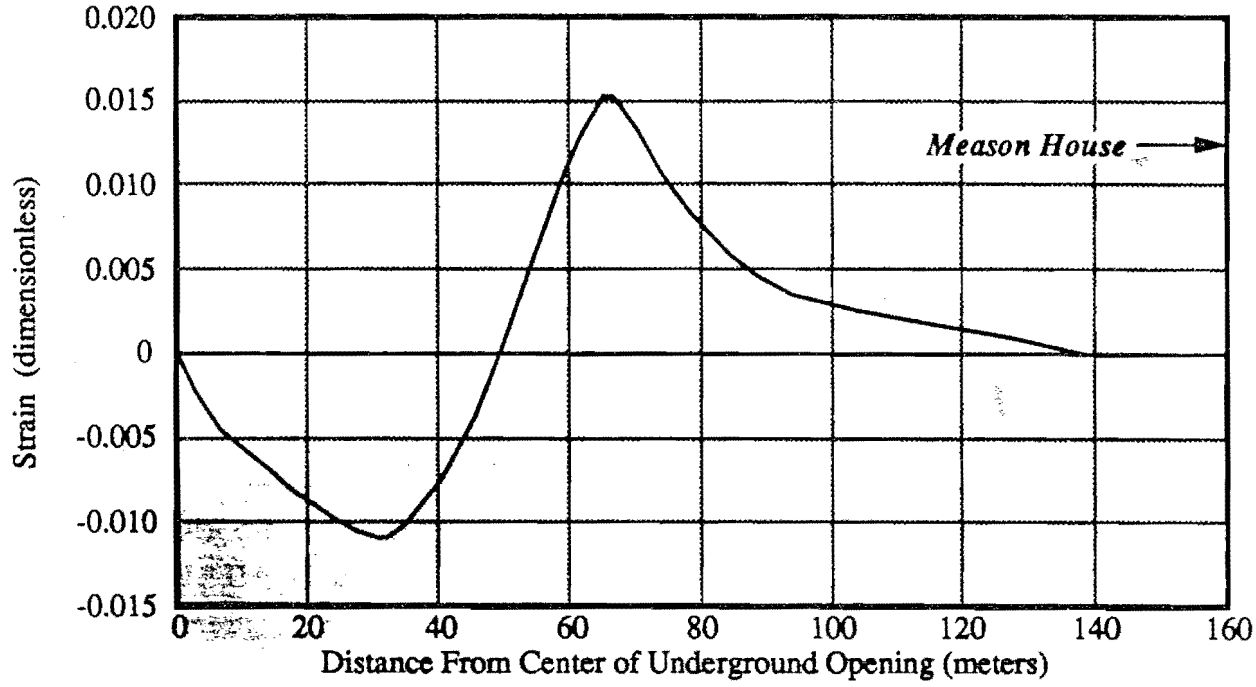
Figure 34

SUBSIDENCE AND STRAIN PROFILES NEAR CELLURALE HOUSE

DONALD E. SHAW, P.E.	1/16/89



(A) SUBSIDENCE DISPLACEMENT



(B) SUBSIDENCE STRAIN

NOTE:
Panel width = 137 m
Panel depth = 108 m
Seam thickness = 3 m
Determined by graphical method given in Peng, S.S., **Coal Mine Ground Control,** Wiley Interscience, New York, 1978 after method of National Coal Board.

Figure 35

SUBSIDENCE AND STRAIN PROFILES NEAR MEASON HOUSE

DONALD E. SHAW, P.E.	1/16/89

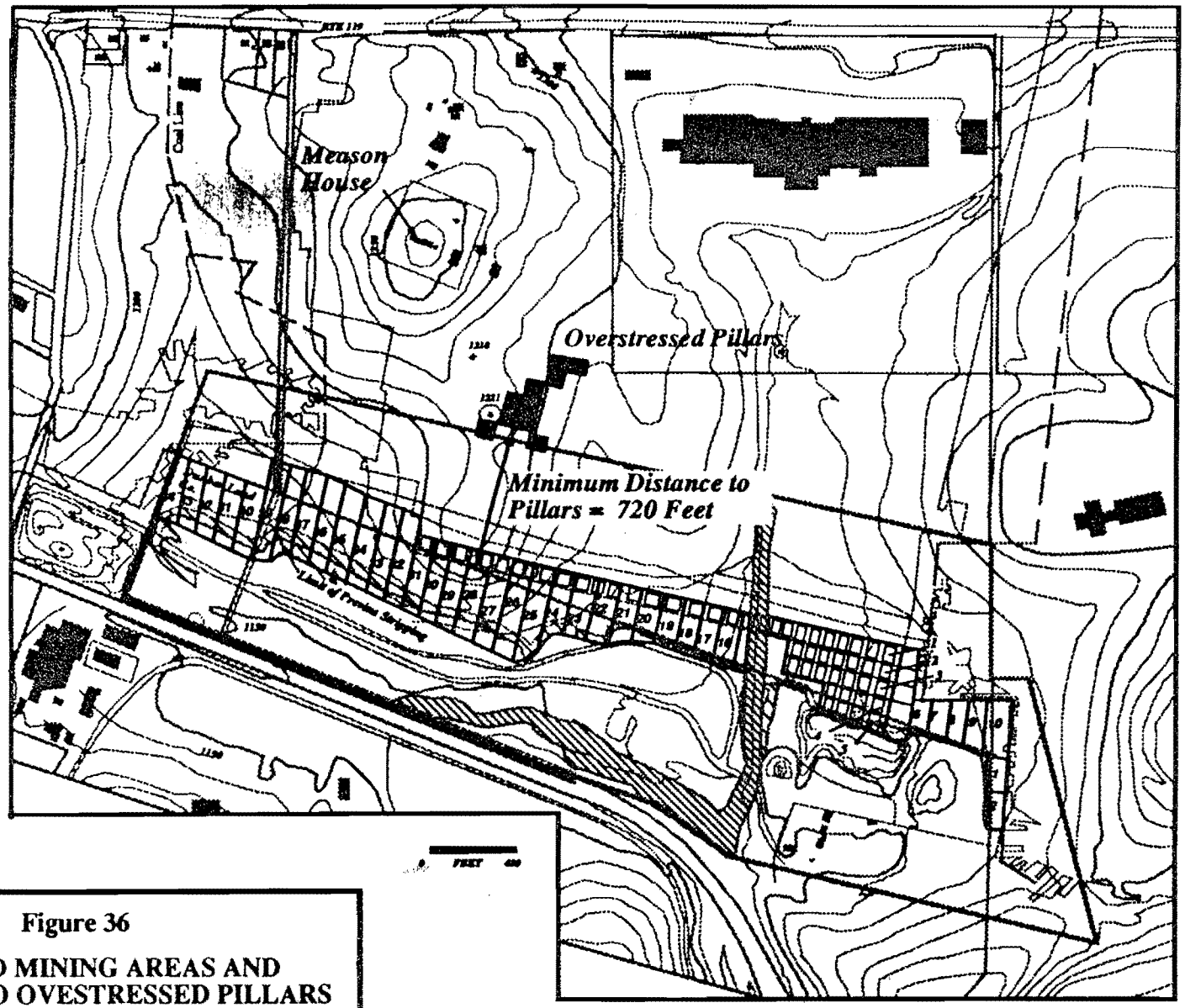
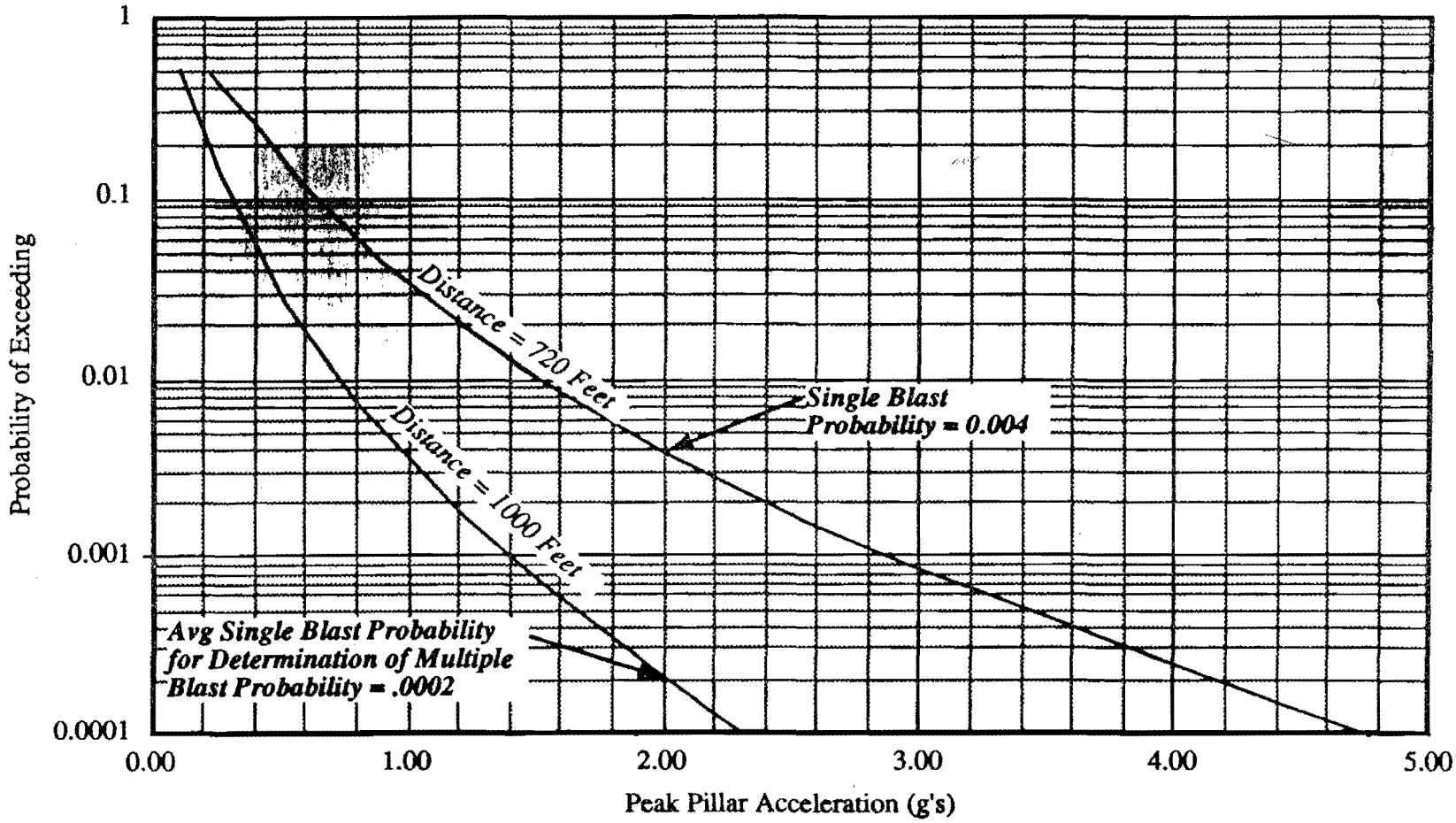


Figure 36

PLANNED MINING AREAS AND DISTANCE TO OVESTRESSED PILLARS

DONALD E. SHAW, P.E.	2/3/89

Note:
Unofficial map, for information only. Scanned from Bond Map obtained from DER on January 9, 1989. Map plan overlaid from Bond Map stamped "Received April 7, 1988.

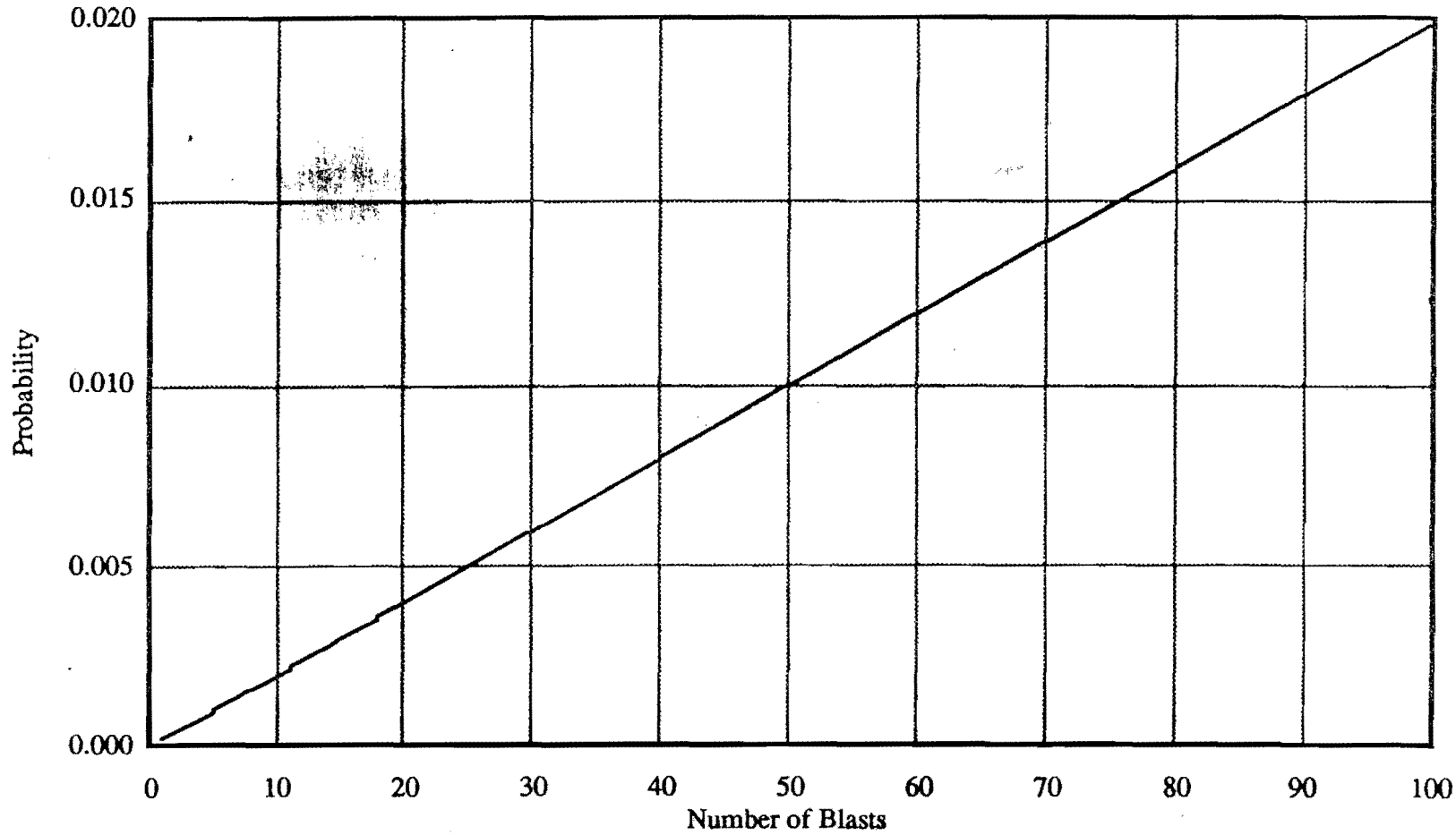


NOTES:
1.Based on scaled distance regression line from Olson, et. al. **Mine Roof Vibrations from Underground Blasts**, U. S. Bureau of Mines, RI 7330.

Figure 37

PROBABILITY DISTRIBUTION OF PEAK ACCELERATION AT PILLARS

DONALD E. SHAW, P.E.	2/2/89



NOTE:
Based on a Binomial Distribution assuming each blast is independent with a probability of damage of 0.0002 based on the distance of 1000 feet

Figure 38

PILLAR DAMAGE PROBABILITY VS. NUMBER OF BLASTS

DONALD E. SHAW, P.E.	2/3/89

APPENDIX A
STRUCTURAL ANALYSIS
OF MEASON HOUSE

APPENDIX A
STRUCTURAL ANALYSIS
OF MEASON HOUSE

The potential for damage to the Meason House as a result of blasting is a function of the nature of the blast waves, the dynamic characteristics of the house, and the strength of various elements of the house subject to damage. While research has been performed relative to damage potential from blasting and forms the basis of regulatory requirements for blasting, it is primarily statistical in nature. In an effort to compare the statistical data available in the literature to the specific case of the Meason House, analyses were performed. Section 1.0 discusses analyses performed to approximate the natural frequencies of the house which are a primary element in determining its response to blasting vibrations. Section 2.0 presents an analysis of the potential response of the Meason House to blasting vibrations based on the natural frequencies and the characteristics of the structure to predict strain levels in the mortar interfaces between the limestone blocks. Effects of existing cracks in the mortar on increasing the potential for damage are also addressed. Finally, Section 3.0 examines the susceptibility of existing plaster, which in some areas is sagging, to blasting damage by attempting to quantify the effects of existing conditions on strength margins.

1.0 NATURAL FREQUENCY ANALYSIS

A structural dynamics model was used to estimate the natural frequencies of the Meason House because of the importance of the relationship between the frequency content of blast induced waves and the natural frequencies of the house to the dynamic response of the structure. The model used a two degree of freedom system for the two principal excitation directions as shown in Figure A.1. The springs shown on the figure represent the stiffness elements of the house which were assumed to be the shear stiffness of the limestone walls. The mass elements shown on the figure represent the weight (mass) of the walls of the house and the roof.

While the house would possess many more than two degrees of freedom per direction of excitation, the higher frequency modes would not be expected to contribute significantly to the overall structural response. The use of two degrees of freedom for the main, three-story portion of the house generally exceeds the approach used for conventional frame structures which typically consider only a single degree of freedom system per direction. The additional degree of freedom

was considered warranted based on the height of the house and the expectation that second mode natural frequencies would coincide with frequency ranges of concern relative to the frequency content of blast waves. -

Subsequent sections discuss the approach used in calculating the estimated natural frequencies in greater detail.

1.1 STIFFNESS ELEMENTS

The stiffness of the Meason House to horizontal motion is provided principally by the shear stiffness of the outer limestone walls. Bending or flexural stiffness of the walls was considered negligible. The stiffness as a shear beam is given by:

$$k = GA/H$$

where;

G is the shear modulus of the material comprising the wall.
A is the area of the wall, and
H is the height of the wall.

1.1.1 Shear Modulus

The Shear Modulus of the wall is a property of the material comprising the wall. For the outer walls of the Meason House the walls are a composite of the limestone block and the mortar between the blocks. The Shear Modulus of limestone was obtained from the Modulus of Elasticity through the relationship;

$$G = E/2(1 + \nu)$$

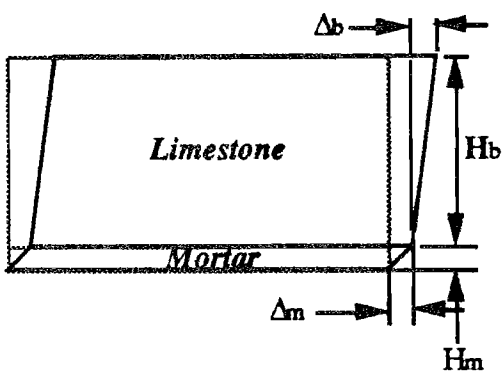
where;

E is the Modulus of Elasticity, and
ν is Poissons Ratio, taken to be 0.25 for limestone.

Based on an Elastic Modulus for limestone ranging between 3×10^6 pounds per square inch (psi) and 6×10^6 psi with an average value of 4.5×10^6 psi, G ranges from 1.73×10^8 pounds per square foot (psf) to 3.46×10^8 psf.

To account for the effects of the mortar between the blocks on the composite modulus of the walls the above moduli were divided by a factor of 10. Page[1] in a study of the stiffness of brick walls shows reductions on the order of 1/2 to 1/10 in the modulus of the brick to account for the modulus of the mortar which is more flexible. While minor cracks are visible in the mortar at various locations, the asperities at the block-mortar interface coupled with the weight acting of the blocks should provide sufficient shear resistance to mobilize some mortar stiffness as long as the strains remain small. If strains do not remain small, the stiffness will decrease, but this was considered unimportant since large strains required to mobilize non-linear behavior would constitute damage to the walls.

The following sketch shows an element consisting of a layer of limestone block and mortar. The stiffness of the combined limestone and mortar constitute a series combination of stiffness such that the overall stiffness is given by;



$$K = \frac{K_b K_m}{K_b + K_m}$$

$$= \frac{G_b G_m}{H_m G_b + H_b G_m}$$

$$G = \frac{(H_m + H_b) G_b G_m}{H_m G_b + H_b G_m}$$

$$= \frac{12.25(4000)1.8 \times 10^6}{0.25(1.8 \times 10^6) + 12(4000)}$$

$$= 177{,}108 \sim 1.8 \times 10^5 \sim 0.1\ Gb$$

where;

K is the overall stiffness of the mortar block combination,

K_b is the shear stiffness of the limestone (GbA/Hb)

K_m is the shear stiffness of the mortar (GmA/Hm)

A is the cross sectional area which is the same for both the mortar and limestone,

G_b is the shear modulus of the limestone, taken as 1.8 x 106 psi,

G_m is the shear modulus of the mortar, taken as 4,000 psi,

H_b is the height of the limestone block, taken as 12 inches, and

[1] Page, A. W., "Finite Element Model for Masonry", **Journal of the Structural Division**, American Society of Civil Engineers, v. 104, No. ST8, Aug, 1978, pp. 1267-1285

H_m is the height of the mortar, taken as 1/4-inch, and
G is the composite shear modulus.

The area of the wall varied depending on the direction of excitation. Since the bending stiffness perpendicular to the plane of the walls is negligible, for the analysis of the natural frequencies for X-direction excitation, as shown on Figure A.1, the area was computed based on the sum of the areas of the front and back walls. The side walls were neglected. For the computation for Y-direction excitation, the area was computed based on the area of the two side walls, and the front and back walls were neglected.

Based on observations made at the Meason House, it was judged that it would behave dynamically as three separate structures, the main portion of the house and each of the two wing structures. The walls forming the connecting halls between the main part of the house and the wing structures were considered to be inadequately connected to the main house walls and wing structure walls for the entire lower story to act as a composite. However, if these connecting hallways do contribute to the stiffness of the lower floor of the structure, the effect would be to increase the natural frequencies moving them towards the higher frequency components of the blast wave.

The height, H, in each spring was scaled from the elevation of Meason House shown in Figure 2 of the report.

1.1.2 Mass Elements

The value of each of the two mass points used was calculated using conventional methods for lumped mass dynamic structural modeling. The items included in each mass were as follows:

- In the main portion of the house mass m1 included the weight of half the height of all four walls between the ground and the level of m1 plus the weight of half the height of the walls between the level of m1 and the level of m2.

- Mass m2 in the main portion of the house included the weight of half the height of the walls above the level of m1 plus the weight of the walls beneath the gables on each side of the house plus the weight of the roof.

- For the wing structures only a single degree of freedom system was used and mass m1 included the weight of half the height of all four walls plus the walls beneath the gables on each side plus the weight of the roof.

1.3 NATURAL FREQUENCIES AND MODES SHAPES

Figure A.1 shows a table of the natural frequencies estimated for the Meason House. Figure A.2 shows normalized mode shapes calculated for the Meason House corresponding to the main portion of the house.

The table of natural frequencies on Figure A.1 shows frequencies for X and Y direction excitations. As shown, the frequencies are almost the same because the plan dimensions of the house are nearly the same in the X and Y directions. The frequencies of the wing structures are high because of their relatively low height. They also are nearly the same in the two directions because the plan dimensions are similar.

The mode shapes shown on Figure A.2 are normalized relative to the masses such that they are identical for both X and Y direction excitation.

2.0 STRUCTURAL DYNAMICS ANALYSIS

As an aid to considering the potential for structural damage to the Meason House, a dynamic analysis was performed based on the structural model used to calculate the natural frequencies. The model uses a method known as the lumped mass modal superposition method to estimate the dynamic response of the house when subjected to base excitation caused by ground vibrations.

2.1 BASE EXCITATION

When a structure is subjected to blasting vibrations its response is a result of shaking of the base or foundation. In conventional dynamic analysis techniques used extensively in earthquake engineering, the input ground motion is given in terms of an acceleration. This creates a problem for the analysis of the Meason House because ground vibrations are given in terms of peak ground velocity instead of acceleration. The accelerations can be obtained from the velocity time histories obtained from seismographs by differentiation if the records are available. For the present analysis

time history velocity records were not available and assumptions were required to relate the peak ground velocity to the peak ground acceleration required for the analysis.

It was assumed that the acceleration for each mode could be obtained by multiplying the peak ground velocity by the modal natural frequency. An approximation similar to this has been examined by investigators and found to represent a reasonable approximation. The structural response was computed from the mode shapes using the modal superposition method. The displacements relative to the ground were calculated from;

$$X_j = a \sum_{i=1}^{2} \Gamma_i \eta_i \Phi_{ij}$$

where;

X_j is the relative displacement of mass M_j,
Φ_{ij} are the displacements of Mass M_j in mode i,
η_i is the generalized coordinate for the i-th mode,
a is the ground acceleration, and
Γ_i is the participation factor in the i-th mode given by;

$$\Gamma_i \sum_{j=1}^{2} M_j \Phi_{ij}^2$$

when the mode shapes have been normalized relative to the masses, as was done for those shown on Figure A.2.

When time history motions are available, the generalized coordinates, η_i, are found as the response of a single degree of freedom system having a natural frequency equal to the structural natural frequency, ω_i, to the time history motion. When they are not available, the generalized coordinates are taken as amplification factors representing the maximax response of the single degree of freedom to the time history input. For damped structures subject to harmonic excitation, the generalized modal coordinate is $1/2\zeta$ where z is the ratio of actual damping to critical damping. Using a damping ratio of $\zeta = 12.5\%$, the generalized coordinate for each mode was taken a 4.

Again, when time histories are available, the individual modal responses are combined at each time point. When they are not available they can be combined using the square-root-of-the-sum-of-the-squares method (SRSS) which has been used extensively in the seismic analysis of nuclear power plants. The SRSS method was used to combine the individual amplified modal responses for the Meason House.

Shear stresses in the walls were obtained from the displacements of the two masses, M_1 and M_2 determined from the modal responses according to:

$$\tau_1 = K_i X_1/A$$

$$\tau_2 = K_2(X_2-X_1)/A$$

where;

τ_1 is the shear stress acting between the first and second floor, and
τ_2 is the shear stress acting above the second floor.

Peak structural velocities were computed by adding the velocities relative to the ground to the peak ground velocities. Peak relative velocities were approximated by multiplying the relative displacements times the modal frequencies based on the harmonic assumption.

2.2 COMPARISON OF STRUCTURAL PEAK VELOCITY TO PEAK GROUND VELOCITY

Research on damage caused by ground vibrations from blasting have compared the measured peak velocity in structures to the measured peak ground velocity. For comparison of the analytical results plots of peak structural velocity to peak ground velocity were made. Figure A.3 shows these plots. Also shown on Figure A.3 is a plot of peak structural velocity as a function of peak ground velocity taken from RI 8507. The data for corner measurements in structures was taken from RI 8507 to minimize the effects of additional amplified response caused by frequencies and modes of structural appurtenances such as walls, ceilings, etc. As shown on Figure A.3 the comparison of the analytically predicted velocities as a function of peak ground velocity compare reasonably well with the plot from RI 8507 which represents a regression curve through statistical data points.

The plot for the Meason House show that higher velocities would be expected than those measured in RI 8507. The higher natural frequencies of the Meason House compared with the

natural frequencies of conventional structures which formed the data base used for the analyses in RI 8507 create more potential for dynamic amplification because of resonance effects.

2.3 STRAIN RESPONSE

Stresses are created during the dynamic response to blasting vibrations as a result of structural deformations. For the Meason House, the weakest element of the primary load carrying structure is the mortar with the joints between the limestone blocks. This is characteristic of masonry construction and is evidenced by the existing cracks observable at some places in the mortar between the blocks.

When structures respond to horizontal ground vibrations created by blasting, either radial or transverse, horizontal stresses are created as the structure deforms. For masonry construction, these stresses are in shear across the block interfaces and result in shear strains within the mortar. Two sets of shear strains were computed. The first corresponds to the average strain across the composite mortar and limestone blocks. The second corresponds to the strain in the mortar. These two strains were computed from the stresses determined in the response analysis based on the following relations:

$$\gamma a = \tau/G, \text{ and}$$

$$\gamma m = \tau/Gm$$

where;

γ_a is the average strain across the composite,
γ_m is the strain across the mortar,
G is the shear modulus of the composite, and
G_m is the shear modulus of the mortar.

Average and mortar shear strains were computed for both the bottom and top sections of the structural walls. Figure A.4(A) shows plots of composite and mortar shear strains at the locations of the two mass points which represent:

- The base of the structure which would approximately coincide with the top of the foundation, and

- The location of mass M1 in the structural model which is at the approximate location of the base of second floor.

The strains shown in Figure A.4 were computed based on a mathematical idealization of the Meason House which, while conforming reasonably well relative to stiffness and mass characteristics, is less valid relative to stress distributions within the walls as a result of deformations induced by ground vibrations. As a consequence of the mathematical model, the strains at the above locations are constant throughout the lower and upper portions of the wall. In other words, the strains at the base of the first floor are the same up to the base of the second floor. And, the strains at the base of the second floor are the same to the top of the structure. This idealization creates a discontinuity at the location of mass M1 which is a result of the inertial force at this point. In actuality, the strains will be distributed continuously from the foundation level to the top of the structure.

Figure A.4 shows strain plots as if there were no cracks in the existing mortar. These strain plots would be appropriate in the case of new construction for which there are no existing cracks. However, cracks do exist along the mortar-block interface in the Meason House.

The existence of cracks can result in stress concentrations at the tip of the crack. These stress concentrations are the result of discontinuities created by the presence of cracks and are the source of crack propagation. Thus, the presence of cracks in the masonry of the Meason House can be expected to result in stress concentration effects with the potential for propagating existing cracks more than creating new ones. Theoretically the stress concentration at the tip of a crack in an elastic continuum is infinite. A rigorous consideration of the fracture mechanics aspects of existing cracks would require information relative to the fracture toughness of the mortar. Since no information is available, stress intensity comparisons to fracture toughness was not made. Instead, it is noted that the consideration of potential ground velocities which could cause damage in the form of additional cracks could occur at peak ground velocities less than shown on Figure A.4.

In addition to strain produced due to elastic deformations, the presence of existing cracks in the mortar raised concern relative to structural damage in the form of movement of the blocks relative to one another along existing cracks. It was assumed that the mortar behaves as a friction material in that the failure envelope is of the form:

$$\tau = \tau_s + \mu\sigma_n$$

where;

τ_s is the elastic strength of the material analogous to cohesion in soils,

μ is a coëfficient of friction, and

σ_n is the normal stress (vertical) due to gravity acting across a crack.

For existing cracks, the elastic strength can be taken as zero such that the mortar behaves as a cohesionless material with a failure envelope;

$$\tau = \mu\sigma_n$$

Page[2] gives three values for μ, 0.87, 0.66 and 0.11 corresponding to three regions of the stress-strain curve. The last, 0.11, corresponds to the region where τ/σ is low which is characteristic of the Meason House. Also in view of the age of the house, the most conservative of the three seemed appropriate. Figure A.5 shows a plot of τ/σ for the same locations as the strain plots of Figure A.4. The vertical stress, σ, was computed based on the weight of the overlying structure. The shear stress was taken from the above relationships.

Without the effects of stress concentration due to existing cracks, Figure A.4 shows that the failure strain is reached at the base of the house at a peak ground velocity of 0.2 inches per second. This value is based on the results of Northwood who found failure at strains of 150 microinches/inch in mortar.[3] From Figure A.5, the at which the τ/σ ratio reaches the limit value of 0.11 is approximately 1.0 inch per second.

2.4 COMBINED EXCITATION DIRECTIONS

The results presented for the structural dynamics analysis were based on an excitation direction corresponding to the transverse axis of the house which would be caused primarily by transverse waves arising from blasting. In addition radial waves would provide base excitation of the house in a front-to-back direction. Since the natural frequencies are essentially the same for the two directions and the mode shapes are the same, the response of the house to radial excitation would be essentially the same as for the shear wave excitation analysed.

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[2] Ibid

[3] Northwood, T. D., R. Crawford, and A.T. Edwards, "Blasting Vibrations and Building Damage," **The Engineer**, v.215, No. 5601, May 31, 1963, pp. 973-978

If the traverse vibrations are the result of shear waves and the radial vibrations are the result of compression waves, the two different wave trains would travel at different velocities with the compression waves arriving first because the compression wave velocity is larger than the shear wave velocity by the following ratio,

$$Vc/Vs = [2(1 + v)]^{1/2}$$

where;

Vc is the compression wave velocity,
Vs is the shear wave velocity, and
v is Poisson's ratio which is about 0.25 for rock.

Thus the ratio of compression wave velocity to shear wave velocity can be approximated as;

$$Vc/Vc = 1.58$$

For compression wave velocities on the order of 10,000 feet per second, the compression blast would would arrive approximately 0.125 seconds after the blast for a distance of 1250 feet. Based on the above ratio, the corresponding shear wave velocity would be 6300 feet per second such that the shear wave would arrive 0.20 seconds after the blast. Thus there would be a .075 second delay between arrival of the compression wave and arrival of the shear wave. For blast wave durations on the order of one second per interval, the delay of 75 milliseconds would not be distinctly noticeable for multiple intervals and the two waves would tend to superimpose.

Stagg and Engler[4] address the multiaxial aspects of blast waves and show typical blast wave signatures which indicate that vibrations in all three directions, transverse, radial, and vertical, begin simultaneously, although close examination of some of the traces show a peak in the transverse component occurring after the peak in the radial component which is expected based on the above delay between compression wave and shear wave arrival. Stagg and Engler show combined velocities which are the vector sum of the individual components from each of the three directions.

[4] Stagg, M. S. and A. J. Engler, **Measurement of Blast-Induced Ground Vibrations and Seismograph Calibration**, U. S. Bureau of Mines, RI 8506, 1980.

Following the approach used for seismic analysis of structures subject to multiaxial earthquake excitation, the response computed for the transverse excitation would be combined with the response due to radial excitation by the square-root-of-the-sum-of-the-squares method. Since both excitations would produce essentially the sum response, this would result in the strain values shown on Figure A.4 being multiplied by 1.4 or increased 40 percent.

In the case of strain propagation in existing cracks, the combined effect could lower peak ground velocities determined as a vector sum of the horizontal components to less than 0.2 inch per second for cracking of the mortar and less than 1.0 inch per second for movement of blocks. However, the increase of 40 percent appears too conservative since it essentially implies in-phase response in the two bi-axial directions which is unlikely. Therefore, probabilities of damage were based on a single direction excitation with the recognition that the combined directional effects could increase shear stresses and strains over those shown on Figures A.4 and A.5.

3.0 EFFECTS OF EXISTING CONDITIONS ON DAMAGE POTENTIAL OF PLASTER

A second concern relative to the Meason House is the plaster which is original on the first and second floors. In some locations, the plaster has pulled away from the oak lathe and is sagging by an observable amount. It could also be sagging elsewhere where observations cannot be made because failure cracks have not yet developed. The effect of sagging is to introduce stresses in the plaster which reduce the strength available for resisting deformations arising from the structural response to blasting vibrations. This reduced strength would have the effect of decreasing peak velocity levels associated with threshold damage compared with data biased heavily toward newer structures.

3.1 STRESS ANALYSIS

To evaluate stresses existing in plaster which is sagging, an elastic plate model was used. It was assumed that the plate has clamped edges on all four sides representing the areas in which the plaster remains bonded to the lathe. Loading was by means of a uniform load due to the plaster' weight. The thickness of the plate was one inch. The top of Figure A.6 shows a sketch of the plate model used for the stress analysis.

Since sizes of various plates could not be determined, a parametric study was used. The dimension "b" on the sketch of Figure A.6 which represents one side of the plate was varied from one to 10 feet. Stresses were determined for both rectangular and square plates having a ratio of a/b of 1.0 and 2.0 where "a" is the length of the side perpendicular to "b" as shown on Figure A.6. Intermediate values of the ratio from 1.0 to 2.0 in steps of .2 were also evaluated, but the results were omitted from Figure A.6 for clarity. Results for a ratio a/b of infinity are essentially the same as for a/b= 2.0.

The maximum stress which occurs at the center of the plate was evaluated using relations given by Roark[5]. The plot on the bottom of Figure A.6 shows maximum stress levels as a function of the size of the plate based on the values of "b" shown. The maximum stress occurs at the center of the longer edge of the plate (side a) and is compressive. In comparing to allowable stresses it was assumed that plaster fails in accordance with a maximum shear criteria in which case the tensile and compressive strengths would be the same. Stress values at the center of the plate are tensile and are approximately one-half of the maximum stress value. Failure stress values were obtained from Leigh,[6] which is referenced in RI 8507,[7] as 300 psi. The increased stress with the distance "b" shown on Figure A.5 results from the increased loading and increased spans as the size of the sagging plate increases.

Figure A.6(A) shows the maximum deflection or sag of the plate which occurs at the center as a function of the distance "b" for ratios a/b of 1.0 and 2.0. As for the stresses, the increased deflection with the distance "b" results from greater loads and longer spans as increasing amounts of plaster are unsupported.

3.2 EXISTING STRENGTH CAPACITY

Figures A.6 and A.7(A) provide a basis for estimating the stress in a sagging plaster panel as a function of the amount of sag. This permits an approximate determination of the stresses currently existing where sagging can be observed which can serve as an indicator of the general existing strength of the original plaster of the Meason House.

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[5] Roark, R. J., **Formulas for Stress and Strain, Fifth Ed.**,McGraw-Hill Book Co., New York, 1975, Table 26, Case 8.

[6] Leigh, B. R., **Lifetime Concept of Plaster Panels Subjected to Sonic Boom**, University of Toronto, Canada, UTIAS-TN-191, July, 1947

[7] Loc. cit.

Figure A.7(B) shows a plot of maximum stress in the plaster as a function of the sag of the plate for ratios a/b of 1.0 and 2.0. The curve for a/b= 1.0 does not extend as far as the curve for a/b = 2.0 because of the differences in plate deflections and corresponding stress levels. The curves are nonlinear because the increased stress as a result of increased sag comes from a change in the span of the plate, not an increased load for the same size plate. In other words sagging increases as the size of the unsupported plate increases while at the same time the maximum stress is increasing.

The curves on Figure A.7(B) permit an evaluation of the existing strength capacity of the plaster. As shown on Figure A.7(B), for an observed sag on the order of 1/8-inch, the stress is approximately 225 psi. Compared to a strength of 300 psi, this means that 75 percent of the plaster strength is used in resisting the load created by the existing sag. Or, there is only 75 psi of reserve strength available. This means that where the original plaster is sagging, the strength is only one-fourth of what it would be without a sag.

Considering the reduction in strength caused by the sag in the plaster, it can be expected that failure would occur sooner as a result of blasting vibrations than if the plaster were not sagging. Assuming that stress and strain levels in the plaster are a linear function of peak ground velocity, this means that the peak ground velocity which can be expected to cause damage to the plaster in the Meason House would be one-fourth of the velocity expected to cause damage in plaster which is not sagging.

The failure stress in plaster which is not sagging corresponds to a strain of 400 microinches per inch based on the Modulus of Elasticity used in the stress analysis of the plate model of a plaster panel. The reserve strength of the sagging plaster is one-fourth of this or 100 microinches per inch. This is an axial strain which corresponds to a shear strain of 50 microinches per inch.

Wiss measured strains in structures subjected to blasting and found that velocities of 1.0 inches per second corresponded to internal strains of 50 microinches/inch in walls.[8] Diagonal plaster cracking in walls corresponds to a shear strain failure so that the 50 microinches per inch

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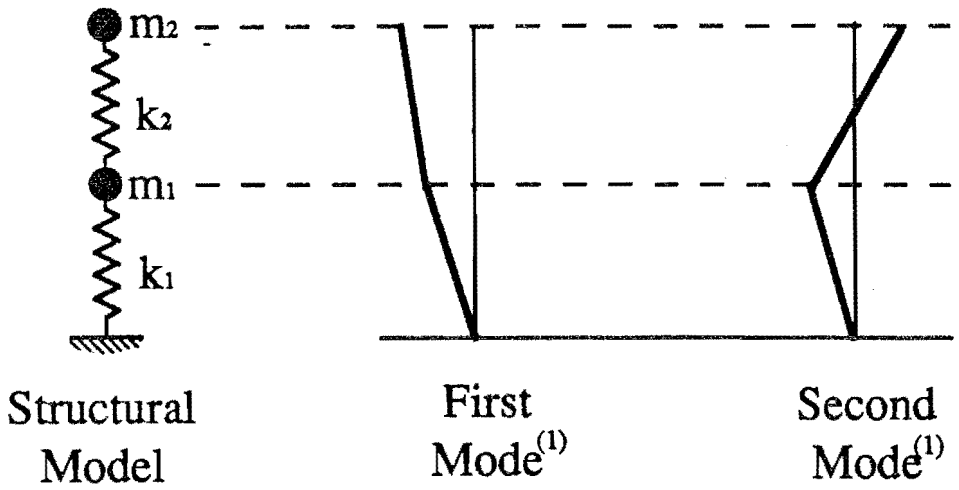
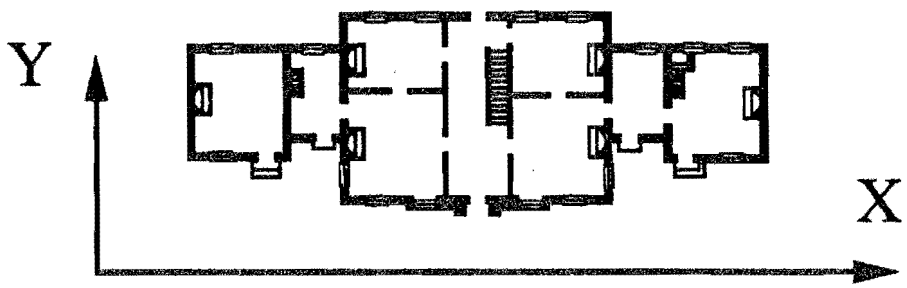
[8] Wiss, J. F. and H. R. Nicholls, **A Study of Damage to a Residential Structure From Blast Vibrations**, Research Council for Peformance of Structures, ASCE, New York, 1974. RI 8507 cites the relationship as 1.0 inch per second peak ground velocity to produce a strain of 50 microinches/inch as opposed to the structural velocity or wall velocity. However, at the locations used for the regression analyses by Wiss, the ground velocity and wall velocity were the same. Thus, the relationship applies to wall velocity as opposed to ground velocity as would be expected.

can be interpreted as shear strain. Thus, to reach a failure strain of 50 microinches/inch a velocity of 1.0 inch per second would be required.





NATURAL FREQUENCIES(2)

MAIN STRUCTURE	FIRST MODE (Hz)	SECOND MODE (Hz)
X - Direction	11.6	29.8
Y - Direction	9.8	25.1
LEFT AND RIGHT WINGS		
X - Direction	22.1	-
Y - Direction	23.6	-

NOTES:
(1) Mode shapes are not drawn to scale and are only intended to illustrate the general shapes.

(2) Natural frequencies are based on analysis.

Figure A.1

MEASON HOUSE NATURAL FREQUENCIES

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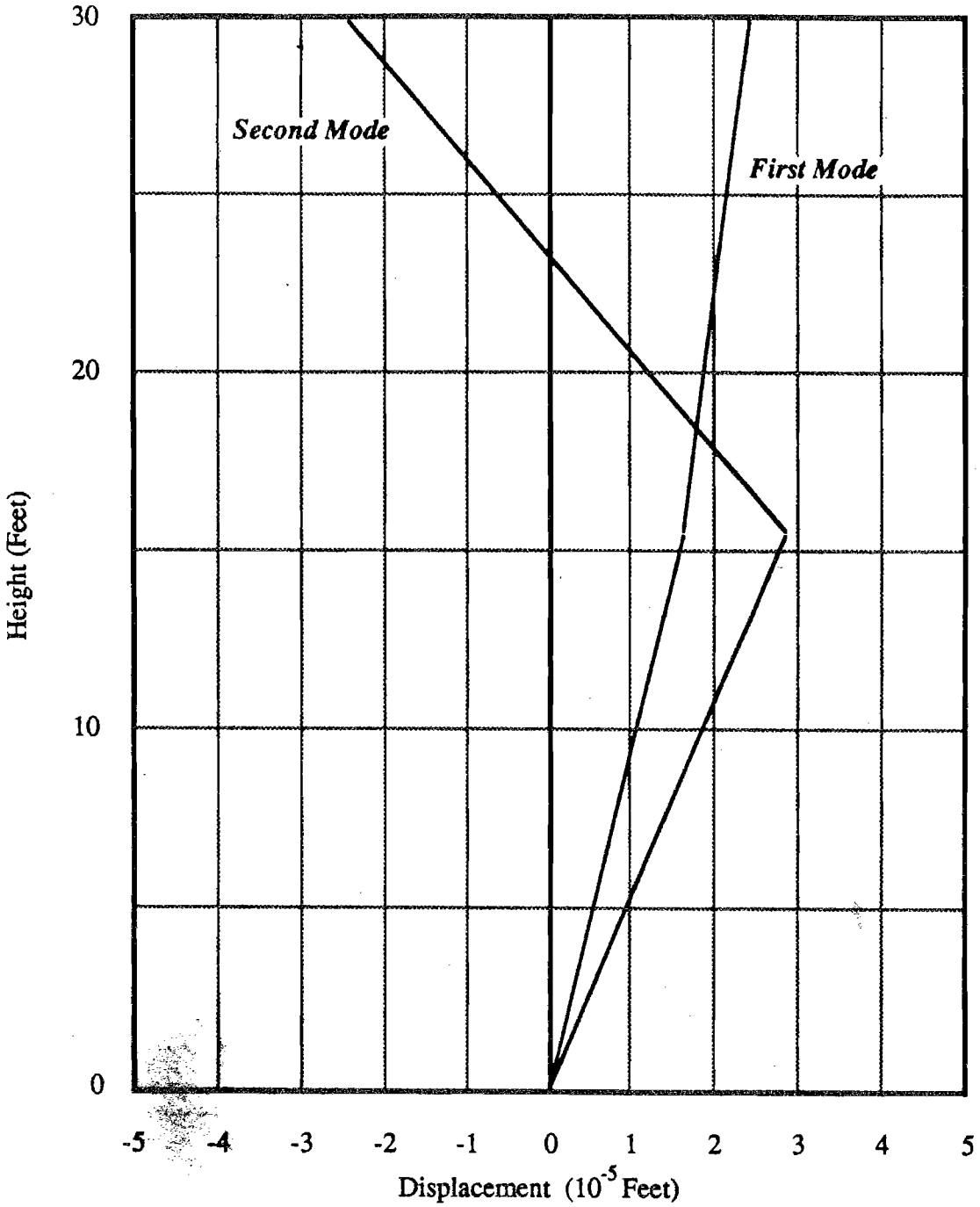

NOTE:
Based on structural dynamic model used to estimate natural frequencies

Figure A.2

APPROXIMATE NORMALIZED MODE SHAPES OF MEASON HOUSE

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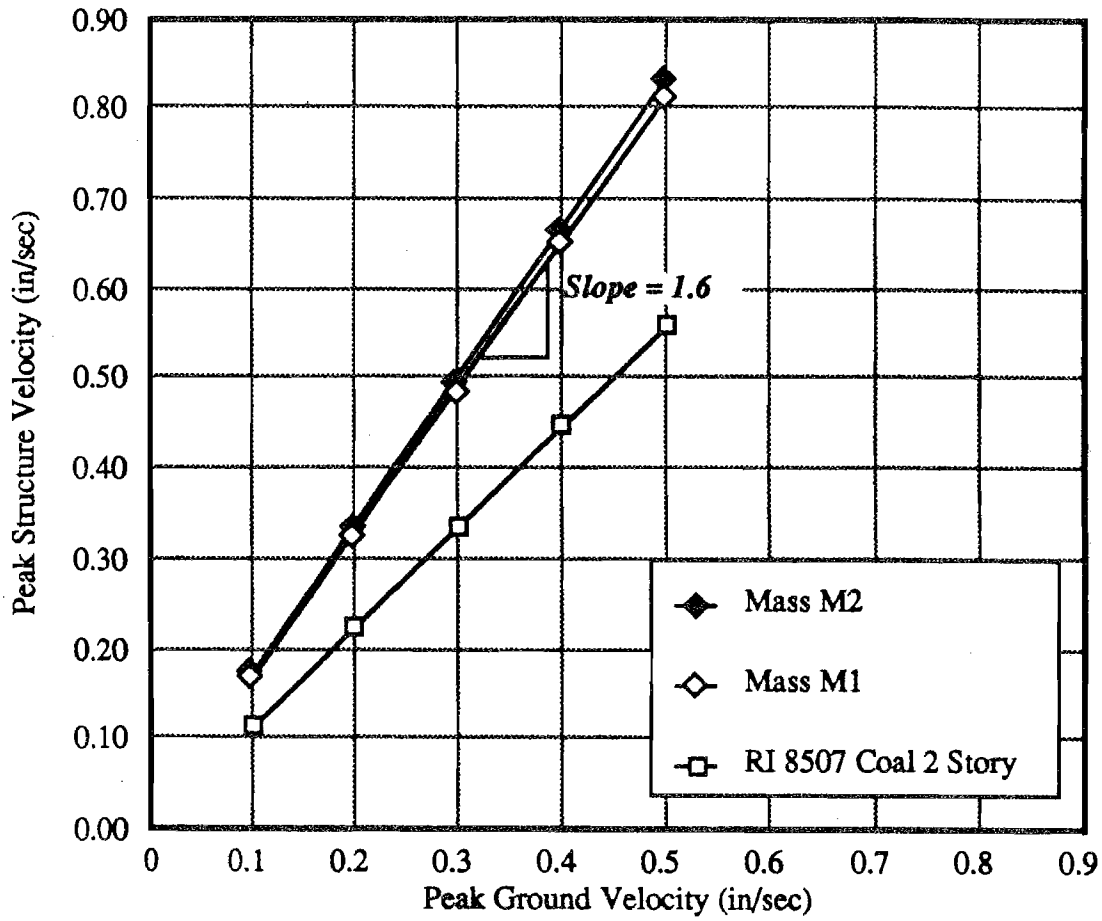

NOTES:

1. Mass 1 and Mass 2 lines determined by dynamic analysis of Meason House using natural frequency structural model.

2. Accelerations required for analysis developed frompeak velocities based on harmonic relationships.

3. Line for RI 8507 reproduced from Fig. 36, Siskind, D.E., et. al., **Structure Response and Damage Produced by Ground Vibration from Surface Mine Blasting**, U. S. Bureau of Mines, RI 8507, 1980. Presented for comparison purposes only.

Figure A.3
STRUCTURAL PEAK VELOCITY COMPARED WITH GROUND PEAK VELOCITY

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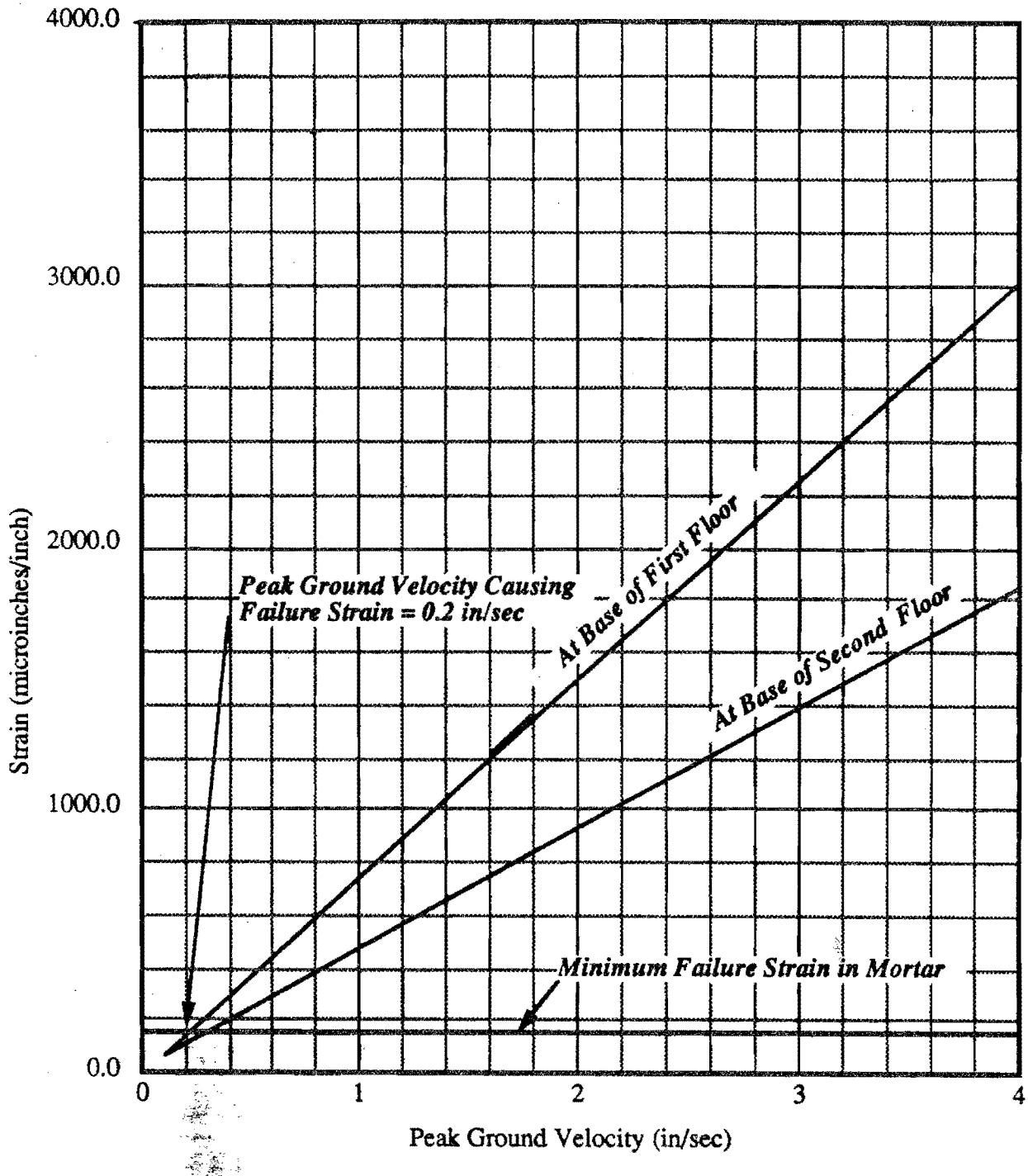

NOTES:

1. Failure strain in mortar taken as minimum value cited in RI 8507 from studies by Edwards, A.T. and T. D. Northwood, "Experimental Studies of the Effects of Blasting on Structures," **The Engineer**, v.210, Sept 30, 1960; and Northwood, T.D, et. al, "Blasting Vibrations and Building Damage," **The Engineer**, v. 215, No. 5601, May 31, 1963

2. Strain based on a Shear Modulus for mortar of 4,000 psi.

Figure A.4

STRAIN IN MORTAR BASED ON ANALYSIS OF MEASON HOUSE RESPONSE

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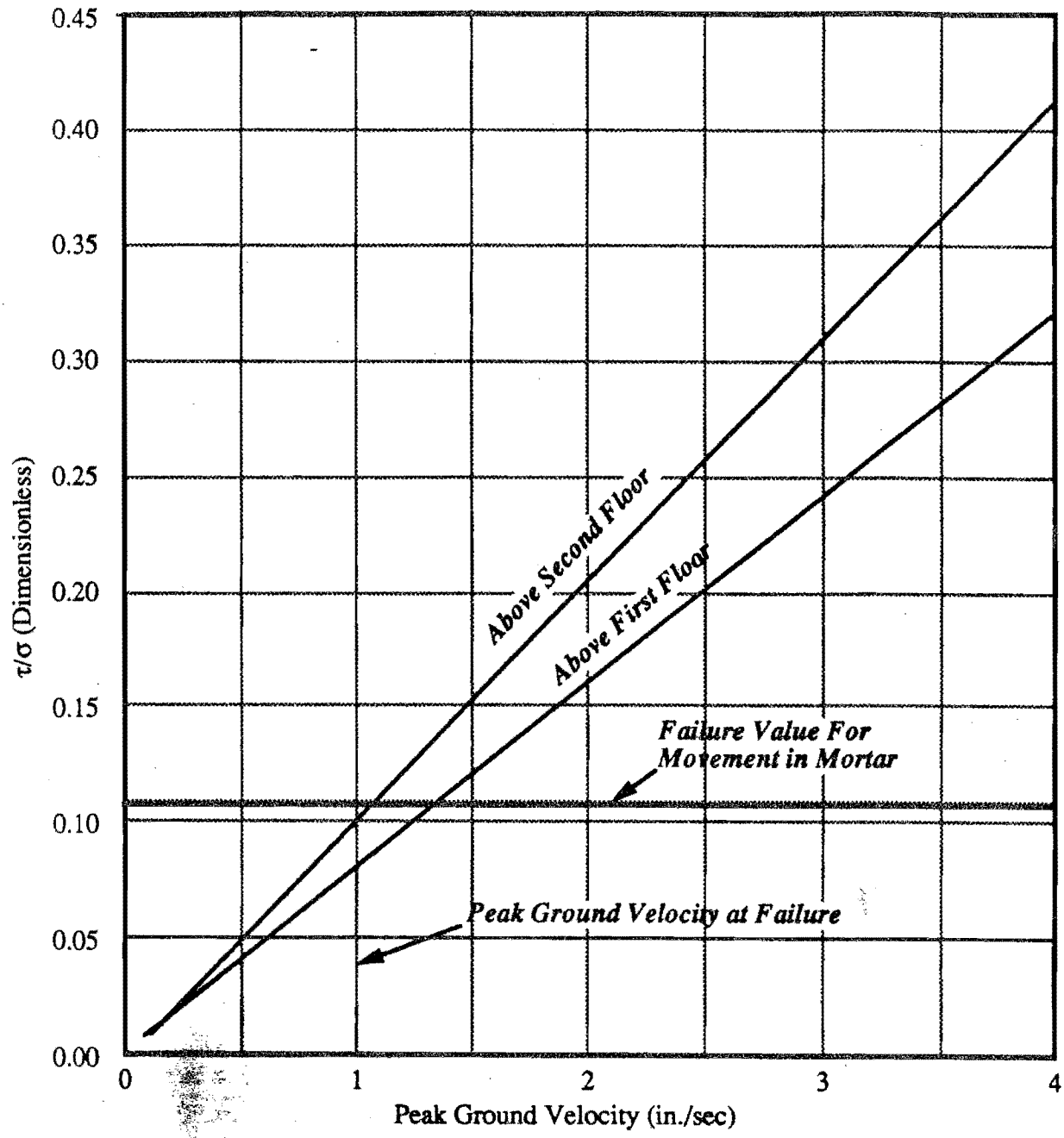

NOTES:
1. Ratio of t/s at failure of 0.11 taken from Page, A. W., "Finite Element Model for Masonry," **Journal of the Structural Division**, American Society of Civil Engineers, v. 104, No. ST8, Aug, 1978, p. 1267

Figure A.5

τ/σ IN MORTAR BASED ON ANALYSIS OF MEASON HOUSE RESPONSE

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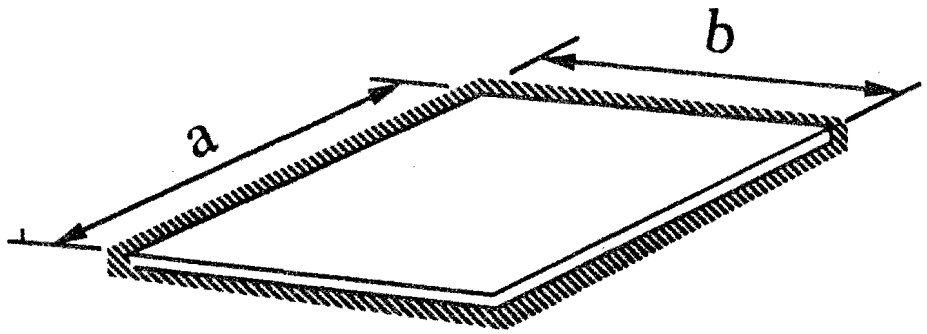

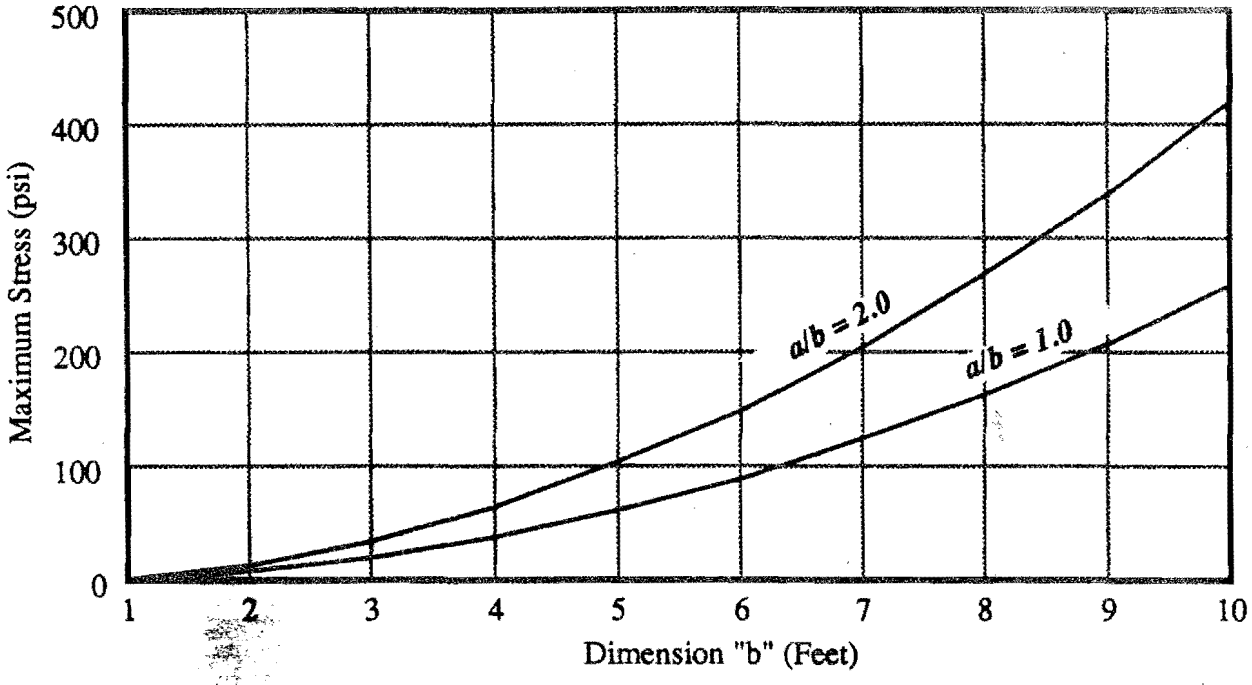

Density = 100 lb/cu.ft.
Modulus = 750,000 psi
Thickness = 1 inch

Ref: Roark, R.J.,**Formulas for Stress and Strain, Fifth Ed.**,McGraw-Hill Book Co., New York, 1975, Table 26, Case 8

Figure A.6

MAXIMUM STRESS IN SAGGING PLASTER PLATE

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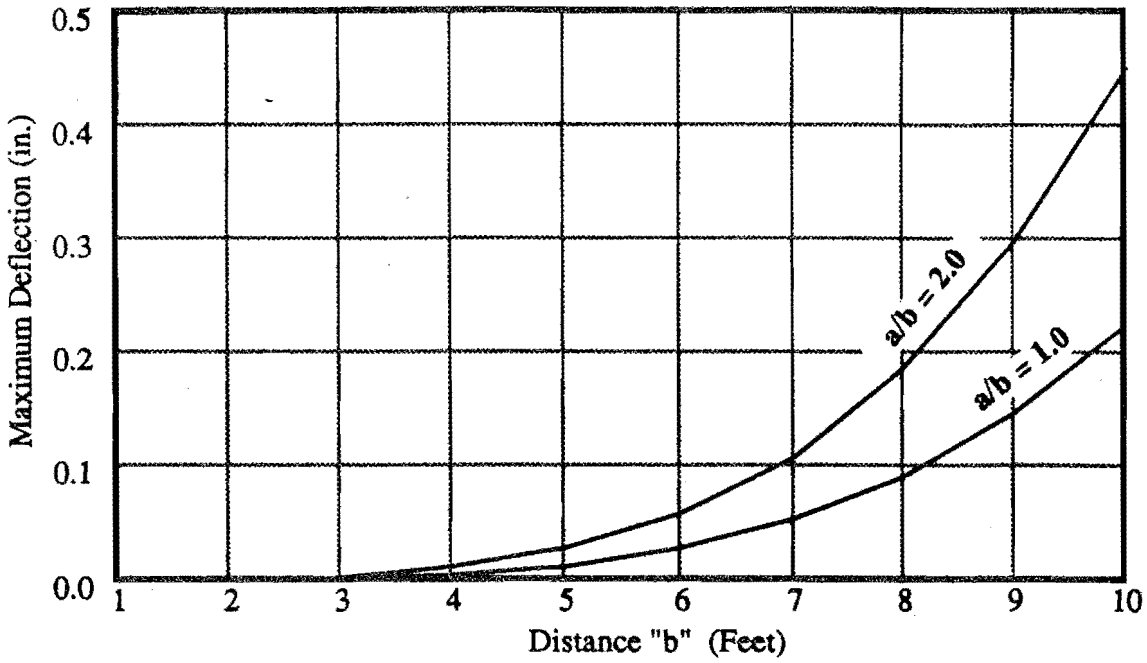
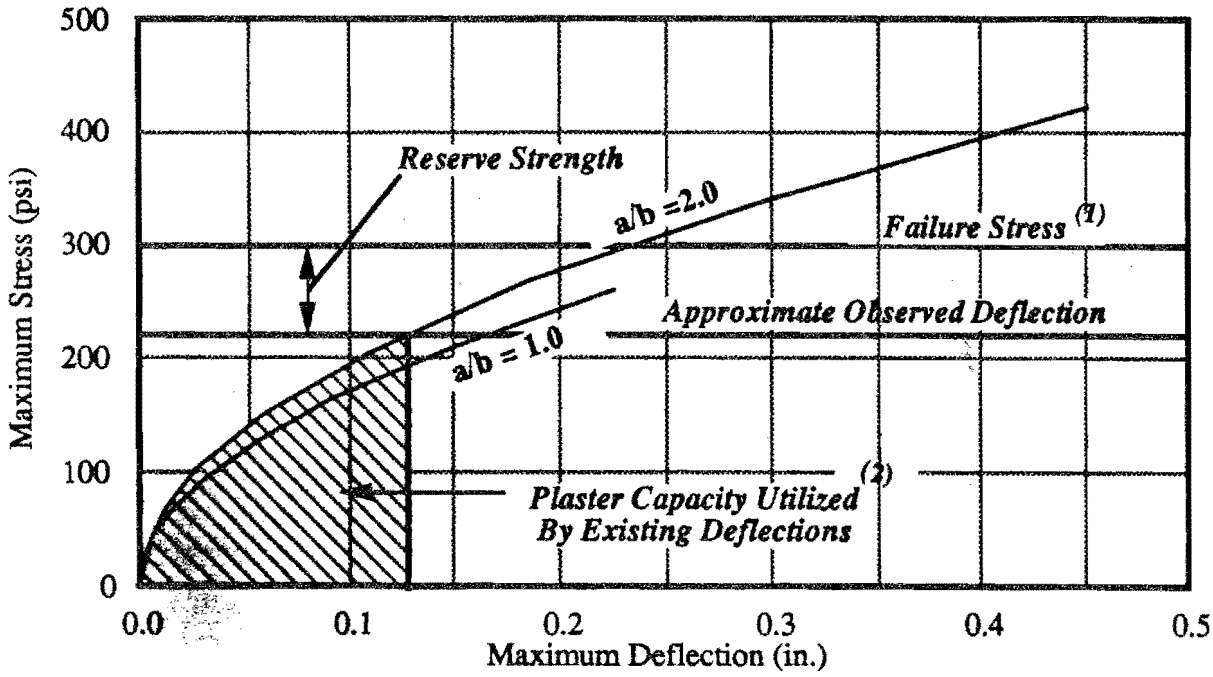


(A) DISPLACEMENT AS A FUNCTION OF PLATE SIZE



(B) RESERVE STRENGTH FOR SAGGING PLASTER

NOTES:

1. Based on Leigh, B. R., **Lifetime Concept of Plaster Panels Subject to Sonic Boom**, University of Toronto, Canada, UTIS-TN-191, July, 1947

2. Based on observation of sagging panels in Meason House. Sag could be greater than 1/8 in such that additional reserve strength would be utilized.

Figure A.7

DISPLACEMENT AND RESERVE STRENGTH IN PLASTER PLATES

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APPENDIX B
REGRESSION ANALYSES
GROUND VIBRATIONS
AND AIRBLAST

APPENDIX B
REGRESSION ANALYSES
GROUND VIBRATIONS
AND AIRBLAST

To provide a basis for estimating damage probabilities the statistics of peak ground vibrations and airblast were required. To avoid confusion in the use of published statistics and permit some selectiveness in the choice of data used, regression analyses were performed for peak ground vibration velocities and airblast using published data as a starting point. Section 1.0 discusses the regression analyses for peak ground vibrations while airblast regression analyses are discussed briefly in Section 2.0. Section 3.0 discusses the regression analysis performed on five-pond shots, and Section 4 presents the statistical analysis peformed for the midwall amplification factors.

The data presented in this appendix forms the basis for estimating damage probabilities for the Meason House relative to estimating probability distributions for peak ground velocities.

1.0 GROUND VIBRATIONS

The data used for the regression analyses of peak ground vibration as a function of scaled distance is that shown in Table 1 of RI 8506.[1] The data presented in Table 1 gives blast parameters of total charge, charge weight per delay, scaled distance, and peak ground velocities in the radial, transverse, and vertical directions as well as the type of blast, coal mine, quarry or construction. To avoid biasing the statistics with quarry and construction blasts which typically result in different frequency content compared with coal mine blasts, only the appropriate data from Table 1 was used.

Figures B.1 through B.3 show the statistical computer output of the three regression analyses for radial, transverse, and vertical peak velocities as a function of scaled distance. Comparison of the plots shown on Figures B.1 through B.3 to the appropriate figures in RI 8507 shows that the results do not differ significantly from those of RI 8507.

[1] Loc. cit.

2.0 AIRBLAST

Airblast data from Table 3 of RI 8485[2] served as the basis for the airblast regression analyses. Table 3 provides the blast parameters, scaled distance according to both the square root of the charge weight per delay and the cube root of the charge weight per delay as well as overpressure data based on several decibel filtering scales. In addition each blast is described as to its type, metal mine highwall, coal highwall, coal parting, etc.

To approximate conditions at the Meason House as well as possible, only coal mine highwall blasts were selected for analysis. The independent variable was chosen to be the cube root scaled distance and the dependent variable was the decibel overpressure reading for peak linear response with a 0.1 Hz high pass filter. Figure B.4 shows the regression analysis results.

3.0 FIVE-POUND SHOTS

On January 28, 1989, a total of 13 five-pound blasts were made. Measurements of peak ground velocity and airblast with made with seismographs in the mine pit, at the Connellsville school, and at the Meason House. The only seismograph which produced measurements was the one in the pit because the blast level was too low to produce vibrations at the school or Meason House. Figures B.5 through B.7 show the regression analyses and statistics for the peak ground velocities in the radial, transverse and vertical directions, respectively. The data points for the regression analysis are given in Table 4 of the report.

4.0 AMPLIFICATION FACTORS

Amplification factors taken from RI 8507 for midwall frequencies were statistically analysed for use in determining the probability distribution for amplified ground motions for the evaluation of damage potential to the plaster. Figure B-8 shows the resulting statistics.

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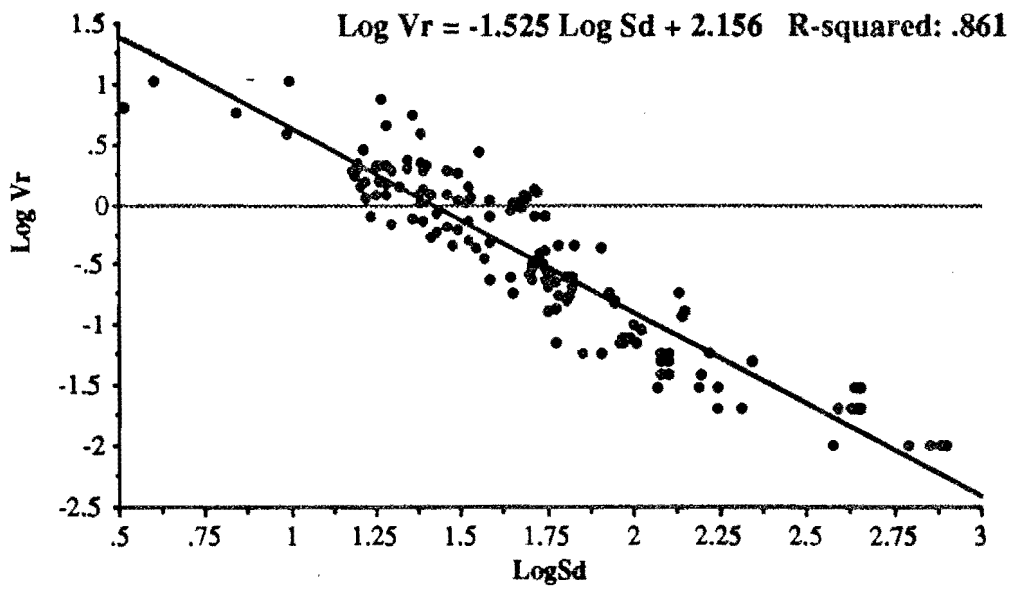
[2] Loc. cit.



REGRESSION ANALYSIS

DF:	R-squared:	Std. Err.:	Coef. Var.:
146	.861	.276	-56.558

Beta Coefficient Table

Parameter:	Value:	Std. Err.:	Variance:	T-Value:
INTERCEPT	2.156	.091	.008	23.654
SLOPE	-1.525	.051	.003	-29.95

Analysis of Variance Table

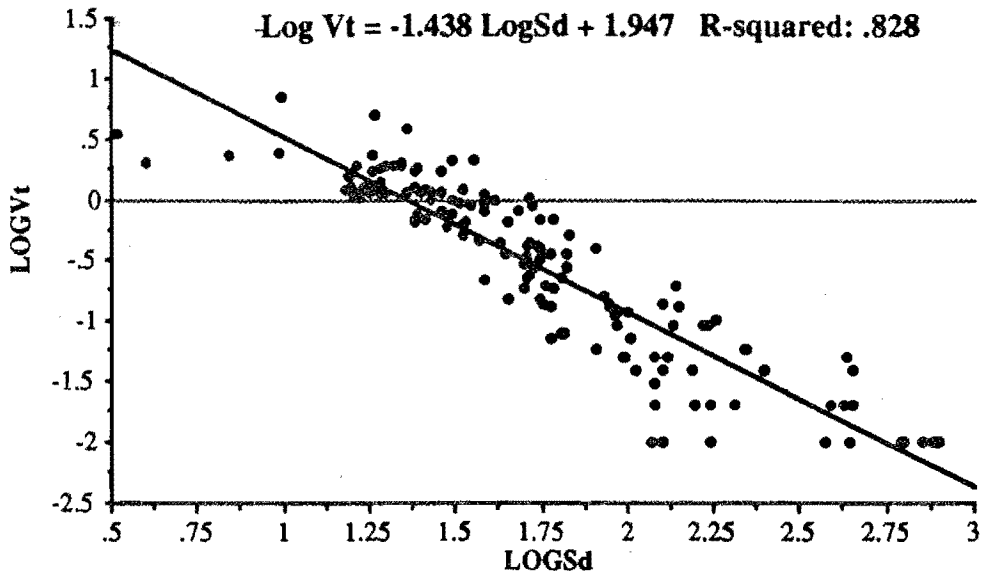
Source	DF:	Sum Squares:	Mean Square:	F-test:
REGRESSION	1	68.241	68.241	897.026
RESIDUAL	145	11.031	.076	$p \leq .0001$
TOTAL	146	79.272		

Residual Information Table

SS[e(i)-e(i-1)]:	$e \geq 0$:	$e < 0$:	DW test:
18.391	66	81	1.667

Figure B.1
RADIAL HORIZONTAL GROUND VIBRATION PEAK VELOCITY REGRESSION ANALYSIS

DONALD E. SHAW, P.E.	1/16/89



REGRESSION ANALYSIS

DF:	R-squared:	Std. Err.:	Coef. Var.:
150	.828	.309	-52.23

Beta Coefficient Table

Parameter:	Value:	Std. Err.:	Variance:	T-Value:
INTERCEPT	1.947	.098	.01	19.839
SLOPE	-1.438	.054	.003	-26.749

Analysis of Variance Table

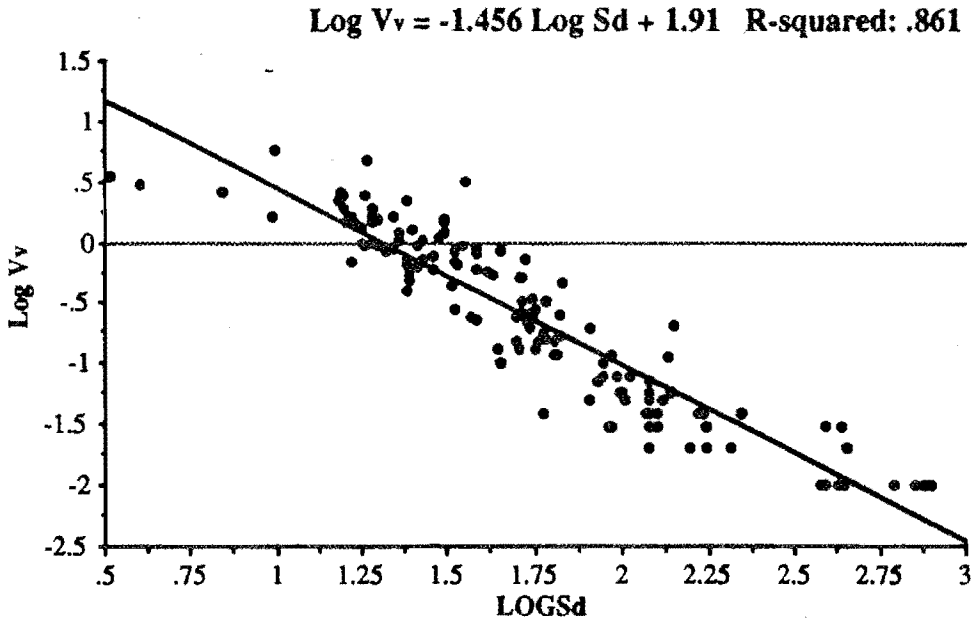
Source	DF:	Sum Squares:	Mean Square:	F-test:
REGRESSION	1	68.1	68.1	715.489
RESIDUAL	149	14.182	.095	$p \leq .0001$
TOTAL	150	82.282		

Residual Information Table

SS[e(i)-e(i-1)]:	$e \geq 0$:	$e < 0$:	DW test:
21.014	81	70	1.482

Figure B.2
TRANSVERSE HORIZONTAL GROUND VIBRATION PEAK VELOCITY REGRESSION ANALYSIS

DONALD E. SHAW, P.E. | **1/16/89**



REGRESSION ANALYSIS

DF:	R-squared:	Std. Err.:	Coef. Var.:
144	.861	.271	-43.063

Beta Coefficient Table

Parameter:	Value:	Std. Err.:	Variance:	T-Value:
INTERCEPT	1.91	.088	.008	21.667
SLOPE	-1.456	.049	.002	-29.8

Analysis of Variance Table

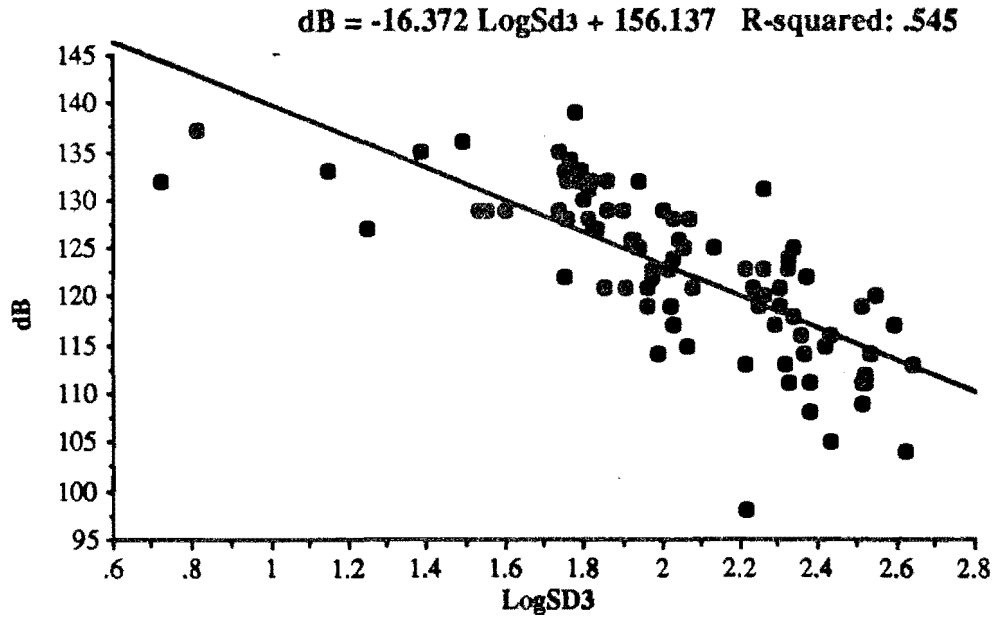
Source	DF:	Sum Squares:	Mean Square:	F-test:
REGRESSION	1	65.343	65.343	888.029
RESIDUAL	143	10.522	.074	$p \leq .0001$
TOTAL	144	75.865		

Residual Information Table

SS[e(i)-e(i-1)]:	$e \geq 0$:	$e < 0$:	DW test:
20.719	70	75	1.969

Figure B.3
VERTICAL GROUND VIBRATION PEAK VELOCITY REGRESSION ANALYSIS

DONALD E. SHAW, P.E.	1/16/89



AIRBLAST REGRESSION ANALYSIS

DF:	R-squared:	Std. Err.:	Coef. Var.:
89	.545	5.577	4.538

Beta Coefficient Table

Parameter:	Value:	Std. Err.:	Variance:	T-Value:
INTERCEPT	156.137	3.294	10.85	47.402
SLOPE	-16.372	1.596	2.547	-10.259

Analysis of Variance Table

Source	DF:	Sum Squares:	Mean Square:	F-test:
REGRESSION	1	3273.568	3273.568	105.239
RESIDUAL	88	2737.321	31.106	$p \leq .0001$
TOTAL	89	6010.889		

Residual Information Table

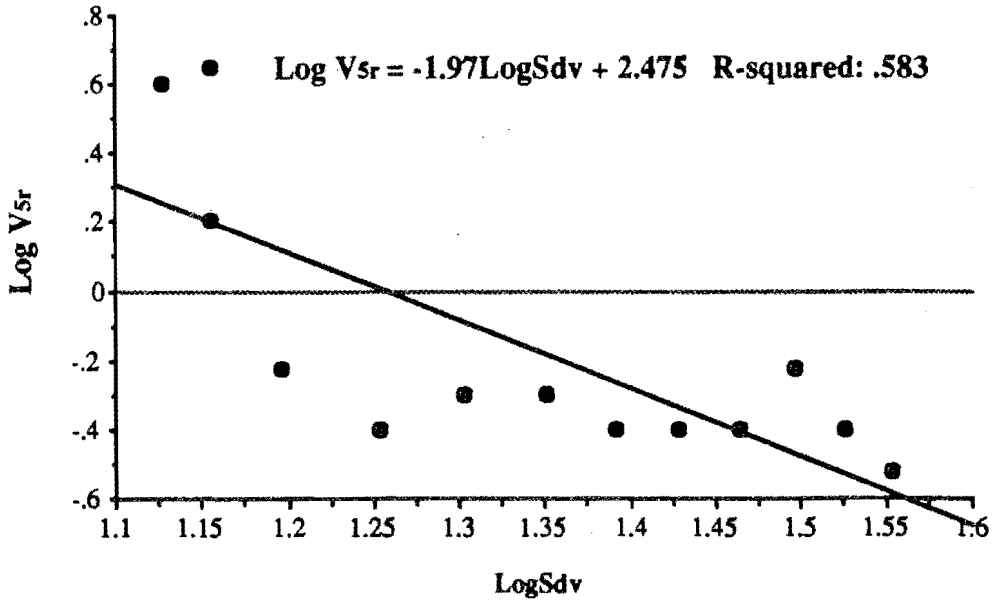
SS[e(i)-e(i-1)]:	$e \geq 0$:	$e < 0$:	DW test:
3322.225	51	39	1.214

NOTE:
Developed from data in Siskind, D.E.,et. al., **Structure Response and Damage Produced by Airblast From Surface Mining**, U. S. Bureau of Mines, RI8485, 1980, for coal mine highwall blasts.

Figure B.4

AIRBLAST OVERPRESSURE REGRESSION ANALYSIS

DONALD E. SHAW, P.E. | **1/16/89**



REGRESSION ANALYSIS

DF:	R-squared:	Std. Err.:	Coef. Var.:
12	.583	.263	-162.066

Beta Coefficient Table

Parameter:	Value:	Std. Err.:	Variance:	T-Value:
INTERCEPT	2.475	.677	.458	3.655
SLOPE	-1.97	.503	.253	-3.918

Analysis of Variance Table

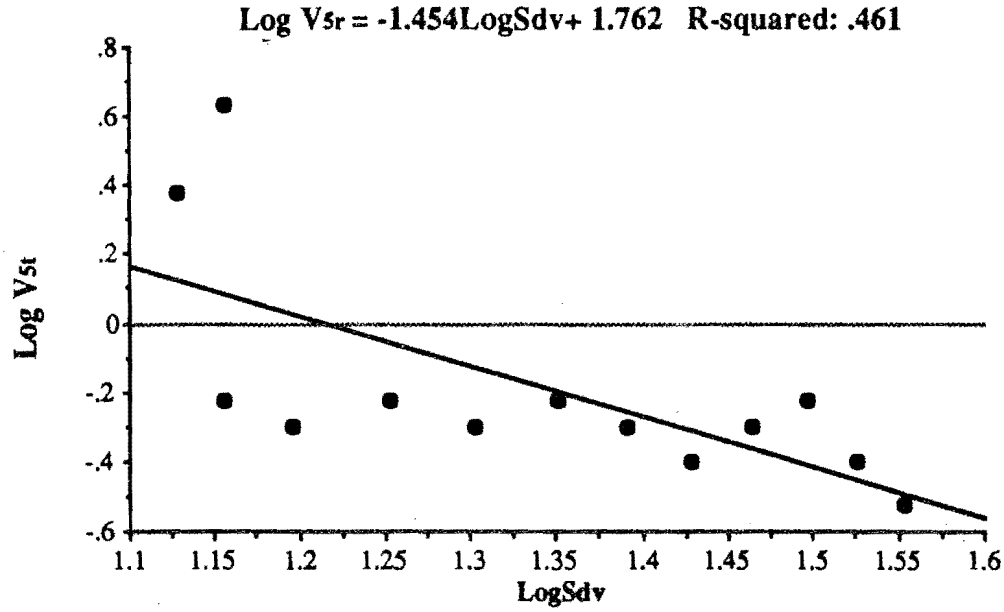
Source	DF:	Sum Squares:	Mean Square:	F-test:
REGRESSION	1	1.061	1.061	15.35
RESIDUAL	11	.76	.069	$.0001 < p \leq .005$
TOTAL	12	1.821		

Residual Information Table

SS[e(i)-e(i-1)]:	$e \geq 0$:	$e < 0$:	DW test:
.384	7	6	.504

Figure B.5
FIVE-POUND SHOTS RADIAL HORIZONTAL GROUND VIBRATION REGRESSION ANALYSIS

DONALD E. SHAW, P.E. | **2/3/89**



REGRESSION ANALYSIS

DF:	R-squared:	Std. Err.:	Coef. Var.:
12	.461	.248	-134.446

Beta Coefficient Table

Parameter:	Value:	Std. Err.:	Variance:	T-Value:
INTERCEPT	1.762	.638	.407	2.76
SLOPE	-1.454	.474	.225	-3.067

Analysis of Variance Table

Source	DF:	Sum Squares:	Mean Square:	F-test:
REGRESSION	1	.578	.578	9.404
RESIDUAL	11	.676	.061	$.01 < p \leq .025$
TOTAL	12	1.253		

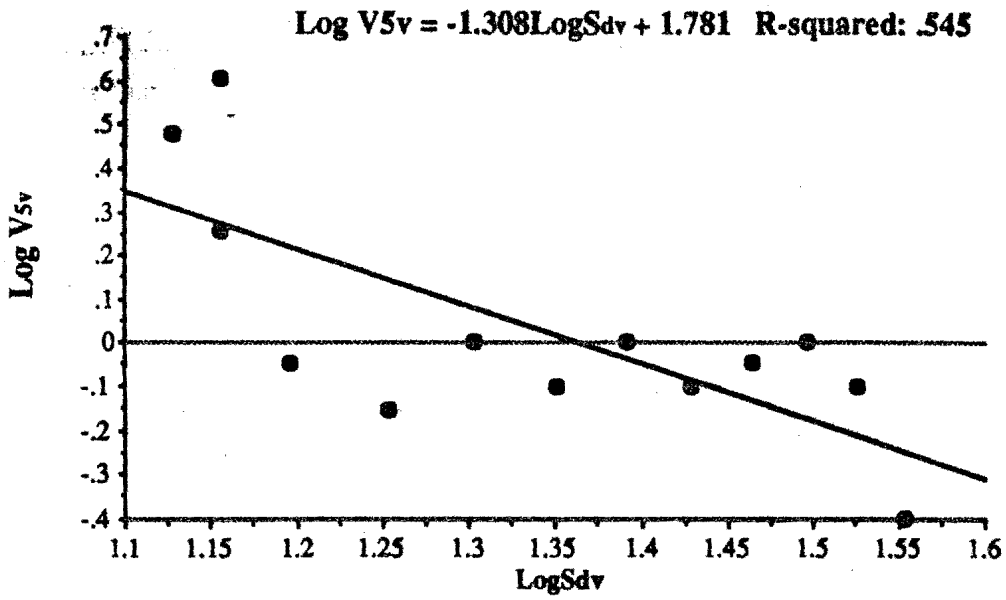
Residual Information Table

SS[e(i)-e(i-1)]:	$e \geq 0$:	$e < 0$:	DW test:
.515	5	8	.761

Figure B.6

FIVE-POUND SHOTS TRANSVERSE HORIZONTAL GROUND VIBRATION REGRESSION ANALYSIS

DONALD E. SHAW, P.E.	2/3/89



REGRESSION ANALYSIS

DF:	R-squared:	Std. Err.:	Coef. Var.:
12	.545	.188	613.153

Beta Coefficient Table

Parameter:	Value:	Std. Err.:	Variance:	T-Value:
INTERCEPT	1.781	.485	.235	3.672
SLOPE	-1.308	.36	.13	-3.63

Analysis of Variance Table

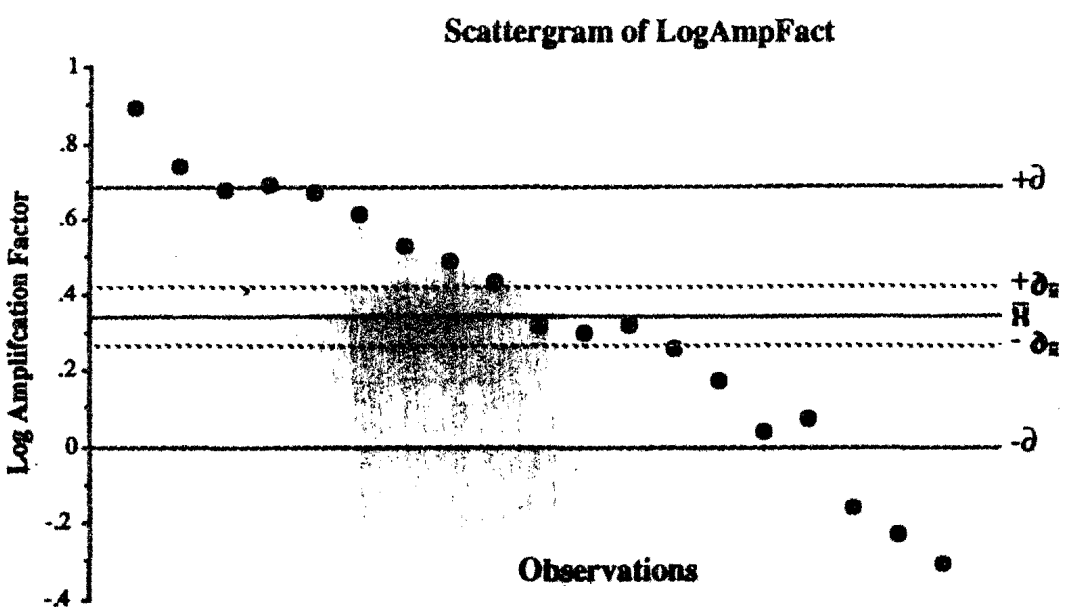
Source	DF:	Sum Squares:	Mean Square:	F-test:
REGRESSION	1	.467	.467	13.174
RESIDUAL	11	.39	.035	$.0001 < p \leq .005$
TOTAL	12	.858		

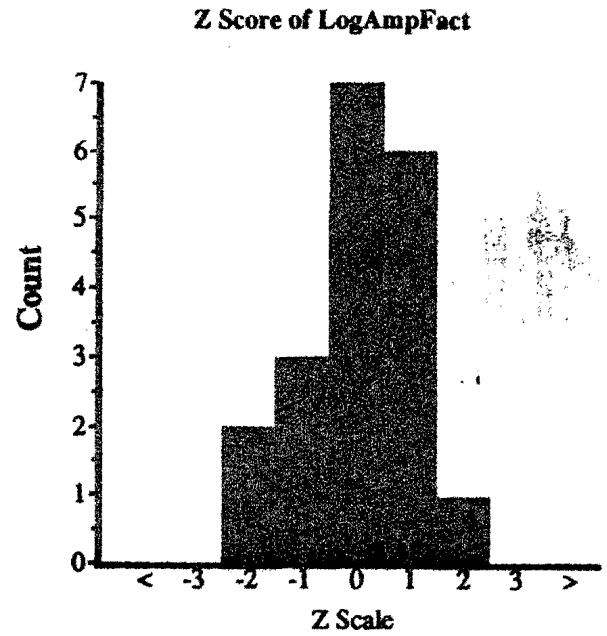
Residual Information Table

SS(e(i)-e(i-1)):	e ≥ 0:	e < 0:	DW test:
[illegible]	6	7	.738

Figure B.7
FIVE-POUND SHOTS VERTICAL GROUND VIBRATION REGRESSION ANALYSIS

DONALD E. SHAW, P.E.	2/3/89





LogAmpFact

Mean:	Std. Dev.:	Std. Error:	Variance:	Coef. Var.:	Count:
.345	.343	.079	.118	99.387	19
Minimum:	**Maximum:**	**Range:**	**Sum:**	**Sum Squared:**	**# Missing:**
-.301	.892	1.193	6.563	4.388	0
Median:	**Mode:**	**Skewness:**			
.322	.322	-.37			

NOTES:
1. Q = Amplification Factor

2. Log (Q) is normally distributed

3. Qmean = 2.4277

Figure B.8

STATISTICS OF DISTRIBUTION OF AMPLIFICATION FACTORS

DONALD E. SHAW, P.E.	2/2/89