Experimental blasting studies on structures
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EXPERIMENTAL BLASTING STUDIES ON STRUCTURES

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ABSTRACT.

The results are presented of controlled blasting tests on a total of six buildings on two different soils. Damage was correlated, with size of charge and distance, and with displacement, velocity, acceleration, settlement, and strain measurements in the buildings. Peak velocity appears to provide the best correlation with damage for all soil conditions.

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EXPERIMENTAL BLASTING STUDIES ON STRUCTURES

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SUMMARY

Controlled blasting operations were carried out in close proximity to six buildings to correlate damage with size and distance of blast. Three of the buildings were situated in wet silty clay soil, the others in well consolidated glacial till. In each case a series of blasts, increasing in intensity, were detonated until damage occurred. Measurements were made of displacement, velocity, acceleration and strain in the structures and at various points in the surrounding terrain. These were also correlated with size of charge, distance and type of soil.

In the silty clay it was found difficult to produce horizontal vibration components although very large vertical vibratory motion occurred. Damage in this area was always associated with large settlement. In the till both horizontal and vertical components were produced. Damage was recognizably associated with the ground wave rather than with incidental effects such as settlement.

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The results are compared with damage criteria currently in use. The observations suggest that of the quantities measured, peak velocity gives the best general correlation with damage.

INTRODUCTION

One of the more vexing problems associated with blasting operations is the danger of damage to nearby buildings. Many operations are handicapped by the necessity of holding blasting charges below a rather indefinitely established "safe limit". In many other cases damage claims arise out of building defects noticed by building owners after blasting has occurred, and it is necessary to try to assess the validity of the claims from a post-mortem consideration of the blasting operations.

A variety of damage criteria have been proposed, of which the best known are those of Thoenen and Windes(1), Crandell(2), and Morris(3). Unfortunately their applicability to the problem in hand has been difficult to judge since there have been very few cases where accurate observations have been made of blasting operations and of the resulting damage to buildings. Clearly the only way to obtain such information is to conduct controlled blasting operations near buildings with the objective of producing damage.

An opportunity to conduct such an experiment arose at the St. Lawrence Power Project during January and February, 1958. Many houses in the area that now form the head-pond were slated for demolition, and it was possible to select a few of these for blasting studies. The selected buildings were old but in good condition. All had basements constructed with heavy stone masonry walls, but both frame and brick superstructures were included. Two types of soil were available: a rather soft sand-clay material and a well consolidated glacial till. Unfortunately there were no buildings founded on rock. Since this experimental work was done a paper has been published describing similar work in Sweden by Langefors, Westerberg and Kihlstrom[4], who carried out experiments on buildings founded on rock. The two studies together thus provide evidence for a wide range of soil conditions.

In addition to investigating damage criteria a secondary aim of this study was to evaluate methods of monitoring blasting operations. To allow for variations in terrain a criterion based simply on explosive charges and distances must be rather conservative. Moreover, there are many special situations, involving multiple charges or an unusually complicated structure in which it is impossible to predict with any precision. If actual measurements of vibration can be made it may be possible to operate with larger charges and still be well below the damage threshold for the particular region. Hence it is desirable to find a reasonably simple vibration measurement that will provide a dependable indication of damage risk. The uncertain state of present knowledge is illustrated by the fact that the three criteria referred to above are based on maximum acceleration, velocity and displacement, respectively. Which of these is the most useful quantity, and how do they differ? All three quantities were measured, in an attempt to answer these questions. In addition a few observations were made with the traditional falling-pin monitoring system, which certainly has the virtue of simplicity.

Occasionally damage occurs not from ground vibrations but from associated air blast (usually broken windows). In the St. Lawrence studies air blast pressures were measured to ensure that this extraneous effect did not affect the results. Other special instrumentations were occasionally used including a few measurements of strain in building walls.

PROCEDURE

Description of Structures and Soil Conditions

Altogether six structures on two different types of soil were used in the tests. Three of the structures were on a loose wet sand about 20 feet deep under which was soft marine clay. The density was of the order of 110 lb per cubic foot. The water table at the time of the tests was about 7 feet below grade. This soil condition will be referred to as sand-clay for the purpose of this paper. The other structures were on glacial till - referred to hereafter as till. This is a high shear strength material consisting of a highly compacted mixture of sand, clay, gravel and boulders. Density of this material was about 145 lb. per cubic foot, and the water content about 10 per cent. Both soils were frozen to a depth of about 1 foot at the time of the tests. Typical profiles and other information relating to the soils are given in Appendix I.

The structures are briefly described and designated in Table /1/. Photographs of buildings R, E, C and F are given in Figs. /1/ to /4/. All were in good condition except for quite localized areas in one or two of them. Building R and part of Building F were of frame construction above the masonry basement walls. In the other buildings structures above ground level were mainly of 12 inch solid brick which was in good condition except that the bond between the bricks and the mortar was weak in two of the buildings. House E had a 45 degree crack across the front wall which had been patched up and which was said to have been caused by the Cornwall earthquake of 1945. The crack can be seen in Fig. 3. Rock was encountered about 15 feet down at House F.

Instrumentation

The instruments used for the investigation will be discussed in four groups, namely: Shock measuring instruments, recording equipment, structural damage indicators and ancillary measuring devices.

(1) Shock Measuring Instruments

(a) Displacement:- One Sprengnether and three modified Leet three-component seismographs were used to measure displacement. The Leet instruments, of early vintage, originally showed extraneous frequency components due to lack of lateral restraint at the pivots. Suitable modifications had been made to eliminate this defect.

All three components of the Leet instruments have a nominal magnification of 50 and will provide useful records for displacements from about 0.001 to 0.025 inches. The Sprengnether has a magnification of 320 in the vertical direction and 180 in the horizontal direction. It records displacements from about 0.0001 to 0.005 inches. The moving elements of both instruments have a fundamental natural frequency of about 1 cycle per second and are critically damped. The Leet and Sprengnether instruments weigh 70 and 40 lb. respectively.

(b) Velocity:- Two Willmore-Watt seismometers were used to measure the velocity of the movement of the structure or of the ground. The Willmore seismometer is a seismic instrument, with a natural frequency also of the order of 1 cycle per second, in which a coil is arranged to cut a moving magnetic field. Thus its output is proportional to the rate of change of cutting flux and therefore to the derivative of displacement which is directly proportional to the velocity. The element is critically damped by loading the coil with a suitably proportioned loading resistance. It is a single component instrument but by a simple adjustment it can measure velocity in the vertical or horizontal directions. In this work it was used to measure motion only in the longitudinal direction i.e. horizontally in line with blast. The instrument was connected via a suitable resistance network directly to a galvanometer element in a multi-channel recorder. It weighs approximately 20 lb.

(c) Acceleration:- An accelerometer is also a seismic system but with a natural frequency above the range of interest. The response to acceleration of a properly damped instrument is essentially flat for frequencies up to about 50 per cent of its natural frequency. The unbonded strain gauge type made by Statham Laboratories were used for these tests. Three of the transducers had natural frequencies of the order of 400 cycles per second, two were at 250 cycles and one at 110 cycles per second. The recording system had a frequency response which was flat well above that of the accelerometers. Viscous oil damping is incorporated in the transducers. Thus it was essential, for maintaining good frequency response, that they be kept at room temperature during the very cold weather in which the tests were carried out. This was achieved by providing heat lamps over the transducers. The weight of the accelerometers was of the order of 6 oz.

For all types of instruments suitable precautions must be taken to ensure that they truly indicate the vibration. The first requirement is that they be fastened securely to the medium or structure. Otherwise vertical accelerations greater than g or somewhat smaller horizontal components will cause relative movement between the medium or structure and the transducers. A second requirement is that the added mass of the instrument should not load the medium unduly. Because of this second requirement the rather heavy displacement seismographs were always mounted on an extended rigid surface such as a basement floor or paved road and anchored down with chains and turnbuckles.

(2) Recording Equipment

Apart from displacement records obtained from the seismographs the other data were recorded on a photographic type of multichannel oscillograph (Consolidated) or on a direct writing oscillograph (Brush). The amplifiers and galvanometers associated with the Consolidated recorder were flat from 0 to 600 cycles, and the galvanometer fed directly by the Willmore seismometer was flat to 1000 cycles per second. The Brush equipment is approximately flat from 0 to 100 cycles per second.

(3) Structural Damage Indicators

(a) Tell-tales:- In order to obtain a positive indication of movement in existing cracks in plaster or in basement walls, a sheet of paper was pasted across each crack (See Fig. /5/). The adhesive used was a type that is rigid when dry and thus does not yield under load. Consequently a widening or extension of original cracking produced a tear in the paper.

(b) Settlement:- Excessive settlement of a structure, which could be the primary cause of damage rather than the building vibration, was measured with a precise level. Reference points were set up usually in the basement of the structure concerned and where possible a reference datum remote from the test site was also used. Settlement was determined by measuring the changes in the levels of these points with respect to the datum and to each other before and after each blast.

(c) Horizontal Deformation:- Plumb lines were suspended from points near the top of each structure with the plumb bobs just above reference points near ground level. These were used to observe permanent movement of the top of the structure relative to the ground.

(4) Ancillary Measuring Devices

(a) Building Strain:- An attempt was made to measure the dynamic strain in the walls of the structure due to the blasting operations. The walls of the structures and of the basements were of a non-homogeneous nature and the bond between the mortar and the individual bricks or stones was not particularly good in at least two of the structures tested. Thus it was not practical to apply resistance wire strain gauges in the usual manner even if the locations at which the maximum strain would occur could be adequately predicted. To overcome these difficulties a method was devised for measuring the total strain along the whole length of a wall. Resistance wire strain gauges were used to measure the strain in thin steel strapping secured at diagonally opposite corners of a wall. By pre-tensioning the strapping to about half its yield strength it was possible to measure both positive and negative strain up to the limit of the available strain in the strapping.

(b) Air-blast pressure:- Throughout the tests air blast was controlled so that it would not contribute to damage. To this end it was necessary to set up a suitable monitoring system. For this purpose a simple crystal microphone was used in conjunction with a cathode ray oscillograph - the resulting signal, representing the air-blast pressure, being photographed. The frequency response of the over-all system was approximately flat from 20 to 7500 cycles per second.

(c) Falling Pin Gauge:- The opportunity was taken of correlating the response of the falling-pin gauge with damage and with ground vibration. The gauges used comprised eight 1/4 inch diameter rods ranging from 6 inches to 15 inches in length. These are stood on end on a carefully levelled flat plate. Each pin is provided with a restraint so that in falling it will not disturb the remaining pins in the gauge. The performance of the pin gauge is supported by a very elementary analysis which ignores the frequency response of the system[(5)]. From this it is deduced that the threshold value of vibration required to upset a pin varies inversely as its length. It is stated that damaging levels of vibration will upset pins longer than about 8 inches. A more detailed analysis (see Appendix II) indicates that the response of the pins depends more on the waveform of the disturbance than on the lengths of the pins. For the complex vibration that typically occurs there is about equal likelihood of upsetting any of the pins in the set.

Typical Operating Procedure

The typical operation on any one structure was as follows:

1. The building was carefully examined and all portions of the structure that were in poor condition were appropriately marked and noted. Tell-tales were then pasted over the cracks. Photographs were made of areas where damage was expected and again after damage occurred.
2. Plumb bobs were installed, and reference points were set up for settlement measurements.
3. Accelerometers, seismometers, seismographs, falling pin gauges and the strain measuring equipment were installed and connected as necessary to the various recording devices. The air blast measuring equipment was set up outside the structure. Seismographs were also disposed at two or three distant locations suitable for monitoring the larger blasts.
4. The procedure then was to detonate charges of increasing intensity until the structure was damaged. The resulting ground vibration and movements of the structure were observed for each charge and the structure was carefully examined for signs of visible damage.

Instrumentation for acceleration measurements was straightforward. Accelerometers were usually screwed solidly into the foundation walls nearest to the source, with additional units at other points of interest in the building. The only change during measurements on a given building was to adjust the gains of the associated amplifiers to obtain a record of suitable amplitude.

The displacement seismographs presented a problem since the available instruments were too sensitive to record damaging levels of vibration. Consequently, the usual procedure was to use a small preliminary blast for comparing displacements at the building with those at one or two distant monitoring points. Subsequent blasts were observed at the distant points only, and the displacements at the building were calculated from the ratios observed during the calibration blast.

Velocity measurements were similar to the accelerometers except that, as previously noted, only two instruments were available. Moreover they were rather difficult to mount so that they were both secure and accurately levelled, with the result that at high vibration intensities there was evidence that the moving elements were striking the limiting stops. Consequently the number of reliable velocity records was greatly reduced. The main body of direct observations therefore are displacements and acceleration. As the analysis proceeded it was evident that velocity was an important quantity, and calculations were made to obtain velocity from the other records.

The objective with respect to the charges was to place them sufficiently far away from the structure that proximity effects in the soil immediately surrounding the charge i.e. the area in which fracture of the soil occurs, would not contribute to the damage of the structure. In practice it was difficult to carry out this plan since extremely large charges were required to damage a structure when it was 100 feet or more away. This would have involved keeping larger quantities of explosive on hand than was practical. The procedure was therefore to place small calibrating charges at about 150 feet and succeeding larger charges progressively closer, the minimum distance in most cases being not less than 50 feet from the structure. Thus the soil between the individual charges and the structure was undisturbed. The holes varied from 15 to 30 feet in depth depending on the total charge planned and the collar required to control flying debris and air blast. The larger charges were placed in groups of holes between 15 and 25 feet apart arranged to produce approximately a plane wave disturbance representative of a distant blast source. The explosives used were 75% Forcite (Canadian Industries Ltd.) and 60% Special Gel (Dupont), 4 to 5 inches in diameter.

Twenty-two blasts were set off in the vicinity of the six buildings. These will be referred to by a consecutive series of numbers, with a letter prefix referring to the building under test (e.g. C4 is the fourth blast, which occurred near Building C). Two of the buildings (E and S) were fairly close together and observations were made simultaneously in both.

BUILDING DAMAGE

Definition of Damage

One can visualize a variety of vibration processes resulting in stresses on various parts of a structure, and these considerations each lead to a different estimate of what will cause damage. Such a detailed examination, though it may provide useful understanding of some special cases, will not provide a practical basis for controlling blasting operations.

An alternative approach is simply to look for an empirical relation between some measure of vibration energy and building damage. Most buildings are complex structures from the viewpoint of ground vibrations, and as will be shown, a typical blasting vibration is a complex disturbance. When the vibration energy reaching a building exceeds a certain threshold value it is reasonable to expect that some portions of the building or the supporting soil will be stressed beyond their yield points. The question is whether this damage threshold is sufficiently well defined to lead to a general criterion of safe blasting practice. What is most desirable is a threshold of damage that is relatively insensitive to peculiarities of soil or structure.

For purposes of relating vibration energy to damage three categories are defined as follows:

(1) Threshold of damage:- opening of old cracks and formation of new plaster cracks, dislodging of loose objects (e.g. loose bricks off chimneys).

(2) Minor damage;- superficial, not affecting the strength of

the structure (e.g. broken windows, loosened or fallen plaster), hairline cracks in masonry.

(3) Major damage:- resulting in serious weakening of the structure (e.g. large cracks or shifting of foundations or bearing walls, major settlement resulting in distortion or weakening of the superstructure, walls out of plumb).

The damage observed in the six structures investigated is detailed below.

Observations of Damage

(1) Buildings in sand-clay

Building C:- There was no noticeable damage from Test C4. Damage from C5 was mainly in the form of vertical cracks, from hariline to 1/2 inch in width, in the two walls closest to the blast. One of these (1/4 inch width) extended down through the basement wall. An original crack in the rear basement wall opened up and some pieces of stone forming the wall were dislodged (see Fig. 5).

Building E:- Damage first occurred with Test E10 when some vertical and diagonal cracks developed in the basement and upper walls. An original diagonal crack in the front wall was opened up to 1/2 inch in width. Test E11 demolished large sections of the two rear walls and caused the rear upper floor to collapse. The diagonal crack in the front wall opened up to about 1 inch in width.

Building S:- Cracks in the brickwork were mainly vertical and varied from hairline to about 3/8 inch wide.

These were all in a room, about 25 feet in length, at the rear of the building - this being the closest to the blast. The immediately adjoining rooms at the front, a total of some 50 feet in length, were completely undamaged. The rear basement door frame, which originally was in poor condition, settled about an inch.

(2) <u>Buildings in till</u>

<u>Building R</u>:- No damage occurred until Test R3 when some horizontal cracks, up to about 1/32 inch wide, developed in the basement walls. These extended into the two walls, longitudinal with the blast, about 12 feet from the near wall. Nearly all the tell-tales across original cracks broke although none were opened up. Somewhat fewer tell-tales broke in the walls normal to the blast. Most of the windows in the ground floor longitudinal walls broke while those in the walls normal to the blast remained intact. The top section of the chimney was sheared through.

<u>Building T</u>:- Test T17 caused a few of the tell-tales across original cracks to break although none opened up. No new plaster cracks were noticed possibly because there were several layers of paper on the walls. A few of the bricks in the chimney became dislodged. Somewhat fewer tell-tales were broken by Test T18. This caused some minor horizontal and vertical cracks in the basement walls - these being generally between courses or associated with windows, etc. Additional bricks were dislodged from the chimney.

<u>Building F</u>:- Test F20 caused some minor horizontal cracking in the basement wall, generally between courses, and some stone to be dislodged from around one basement window. One partition wall, which was originally in poor shape, was cracked and a few tell-tales were broken. Major damage was inflicted on the building by Test F22. Considerable sections of the rear basement wall fell away and the masonry walls above ground level were bulged out approximately 3 inches. some of the upstairs partition walls became separated from the outside walls and there were a number of cracks 1/16" to 3/8" wide, both vertical and diagonal, associated with doorways and windows.

In general the type of damage was found to be related to the soil condition. In the sand-clay vertical cracks occurred which were associated with large settlement. Examples of this type of damage are shown in Figs. /6/ and /7/. In the till, damage was more often associated with horizontal cracking and shattering of the basement walls as exmplified by Figs. /8/ and /9/. It was interesting that chimneys are sometimes the first part of a building to show signs of weakness. Figure /10/ shows an example that occurred at Building R.

It may be concluded that the damage in the buildings in sand-clay was caused by failure of the soil, i.e. settlement, under the buildings rather than by wave energy within the building proper.

VIBRATION RECORDS

Interpretation of Records

Vibration measurements are commonly made with instruments that record either displacement or acceleration. Some authorities (e.g. Crandell) suggest that it does not matter which quantity is measured since one can use the amplitude and frequency of the disturbance (assumed to be sinusoidal) to calculate the corresponding value of whatever quantity (displacement, velocity, acceleration) is needed.

An examination of actual records , as exemplified by Figures 11a, b and c, however will indicate that this is a much over-simplified picture. Figure /11d/ shows a typical set of records of acceleration and velocity obtained at the same observation point for the same blast. The records are quite different in character, and an attempt to determine the most characteristic frequency involves a rather arbitrary decision. Hence it is not possible to use the frequency with confidence as a means of calculating, for example, velocity from acceleration. A numerical differentiation of the velocity record shows that the two records do correspond reasonably well. However the numerical integration and differentiation of such records is a tedious process, and it is obviously better to measure directly the quantity whose amplitude correlates best with damage. Then the precise wave form is of no concern and need not even be recorded.

In the present study it was found possible to estimate the maximum velocity by measuring the steepest slope on the corresponding displacement record. Initially the values determined in this way were systematically lower than observed velocities, but vibration table experiments indicated that the magnification of the displacement instrument at the frequencies involved was about 20 to 40 instead of the rated value of 50. The results have been corrected accordingly. No procedure simpler than a complete integration was found for estimating velocity amplitudes from the acceleration records - a fact that is unfortunate since acceleration records were almost always available for positions a few feet from the point of maximum damage.

Variation of Amplitude with Weight of Explosive and Distance

As noted previously it was not always possible, especially with the displacement instruments, to observe directly the vibration of the portion of the building nearest to the blast. Hence a preliminary analysis was made to determine a satisfactory means of making corrections to the actual observations to give the vibration levels at the most-stressed portions of the structures. To this end the results were examined to find the variation of amplitude with charge (weight of explosive) and with distance from the source. This was done for the acceleration and displacement records.

The observations were found to be very complicated. It was deduced that the observed amplitude at any point depended not only on charge and distance but also on source variations (variations in explosive and in its coupling with the soil immediately around it), structural peculiarities in the medium between source and observation point, and instrument point variations (the coupling between the instrument, the structural element it was attached to, and the medium). However, by a selection of observations that minimized or eliminated the extraneous variables, it was possible to obtain relationships between vibration amplitude and charge and distance. These relationships are average values from which individual results may depart considerably.

The variation of amplitude with charge was investigated by considering pairs of amplitude readings taken at the same observation point with the same instrument for two different charges. Only pairs involving small variations in distance were used, and residual distance effects were corrected for on the assumption of an inverse distance law. Assuming that the amplitude is proportional to some power of the charge, each pair was used to calculate a value of n in the relation $A_1/A_2 = (E_1/E_2)^n$, where A_1 and A_2 are the amplitudes, E_1 and E_2 are the weights of the corresponding charges. This procedure eliminated all extraneous variables except the source factor and possibly some local peculiarities of the medium. Fifty-two such pairs of observations gave an average value of n = 0.70 with a probable error of .04. This value is in agreement with the value n = 2/3 given by Thoenen and Windes, rather than the values given by Morris (n = 1/2) and Crandell (n = 1).

A detailed examination of the results showed no distinction between longitudinal and vertical components or between sand-clay soil and till. There was no variation in n with size of charge from 15 to 750 lb. The average value for the acceleration results was slightly higher than for displacement ($n = .75$ vs. $n = .62$) but this is considered hardly significant.

The variation with distance was determined by considering pairs of observation points at different distances from the same charge. This eliminated source variations but included variations associated with the medium and with the observation points. The results were used to obtain values of m in the expression $A_1/A_2 = (d_2/d_1)^m$ where d_1 and d_2 are distances corresponding to amplitudes A_1 and A_2. An average value of $m = 1.8$, with a probable error of 0.2 was obtained, but the distribution of the observations was unsymmetrical, beginning with a large number of values very close to $m = 1.0$ and with few exceptions extending to higher values only. Variations in instrument coupling to the medium might be expected to produce a symmetric distribution, with low values as common as high ones. Hence it appears that the principal variation is due to imperfections in the medium. It is surmised that in a perfect medium the inverse distance law would hold.

The largest deviations from the inverse distance law were always associated with a marked change in terrain or in the nature of the vibration record. In the sand-clay area there was a large low frequency vibration (2.5 cycles/sec.)

within a few hundred feet of the source which did not occur at all at 1500 feet and beyond (see Fig. /11a/). Amplitude ratios taken inside or outside this area followed the inverse distance law fairly well, but those involving both near and far measurements gave large deviations. These are the points labelled NF in Fig. /12/ which is a scatter diagram of the distance-amplitude results. In general these studies indicated that a prediction for a distant point based on observations at a near point would be quite conservative, whereas it would be unwise to attempt the reverse prediction from distant to near points.

In passing it might be remarked that there is no evidence of a Rayleigh Wave or other type of surface wave, for which m would be about 0.7. Only four observations gave values of m less than 0.9.

BUILDING DAMAGE VS. VIBRATION AMPLITUDE

Observations of Damage vs. Vibration

The quantities of concern in relating building damage to ground vibration are listed in Table II. The acceleration results are almost all direct observations of movement in foundation walls nearest to the blast. Velocity observations usually required a small distance correction. The displacement records almost all required the calculation described earlier, utilizing a calibration test in the building combined with results for a distance monitoring point. The limited velocity observations were augmented by calculations based on the maximum slopes of displacement records. This procedure was not entirely satisfactory since no displacement records were obtainable in the buildings for the damaging blasts. Hence these calculations also involve the same extrapolation procedure used for displacement. Nevertheless there is fair agreement between calculations and observations where both are available. Better agreement was found for a few cases for which acceleration records were integrated to obtain maximum velocity.

Figures 13 and 14 are scatter diagrams showing the relations between longitudinal and vertical displacements, frequency and damage. The results show considerable variation in damage threshold depending on the principal frequency. In fact the trend suggests that the threshold corresponds to a constant velocity. (The dashed lines in the figures represent a velocity of 4.5 inches/sec., a criterion that will be discussed later.) When the results are examined in detail it is seen that a low frequency group were all obtained in the sand-clay soils, whereas most of the higher-frequency values were obtained in the till soils.

Thus there appears to be some correlation between the nature of the soil and the frequencies predominating on displacement records. It will be seen that it is not possible to assign a damage threshold in terms of displacement without some qualification regarding frequency.

The velocity results are plotted in Fig. 15. Since many of the velocity values were derived indirectly a correlation with frequency was not attempted. In any case, despite the extra steps in the derivation, the velocity damage threshold was remarkably constant for all six buildings. The damage threshold for either longitudinal or vertical velocity is about 4 inches per second.

The acceleration results are shown in Figures 16 and 17. Although the results are plotted against frequency it should be remembered that there are usually an assortment of frequencies on an acceleration record. In some instances it was difficult to decide which of two or three widely differing "principal frequencies", all of about the same amplitude, should be plotted. The vertical acceleration component shows a well defined damage threshold of about 4 g. The longitudinal results included one exceptionally low value, but otherwise suggest a damage threshold of between 2 g. and 3 g.

Comparison with other Damage Criteria

Various criteria of damage and recommended safe limits have been proposed, based on a limiting value of displacement, velocity or acceleration. It will be of interest to compare the foregoing results with these criteria.

Thoenen and Windes[1] made exhaustive studies of blasting vibrations and of building damage, but unfortunately the two phases of their work are not too well connected. Measurements were made of the damage produced by a mechanical vibration of ceiling panels in six buildings and these indicated a damage threshold (in terms of our definition) of about 0.7 g. Only one case was reported of damage due to a blasting operation.

This produced threshold damage at a displacement amplitude of about 0.1 inches, which corresponds to about the middle of the threshold vs. frequency curves obtained for the St. Lawrence Studies. The threshold acceleration values obtained in the St. Lawrence study were substantially higher than the vibrator result obtained by Thoenen and Windes. (2 g. to 4 g. as compared to 0.7 g.).

This was true not only for accelerations at ground level but for those measured, in a few cases, in the upper parts of the buildings. Hence it is concluded that a steady state vibration of the type they studied introduces higher maximum stresses than are produced by the transient disturbance due to blasting. It might also be noted that the primary damage mechanism observed in the St. Lawrence work was never similar to the case they studied, of simple transverse motion in a free panel.

Morris(3), on the basis of strength calculations for a series of brick piers, recommended as a safe limit a displacement of 8×10^{-3} in. More recently Morris and Westwater(6), on the basis of a few observations of damage to buildings, estimated that the actual damage threshold is about 40×10^{-3} in. This latter figure is in agreement with the high-frequency portion of the St. Lawrence results (for buildings in till) but is much lower than the low-frequency values obtained for some of the buildings founded in sand-clay. To include the results of Langefors et al a much lower displacement threshold would be required for buildings founded on rock (about 1.6×10^{-3}).

Thus it would appear that Morris' recommended limit is conservative except for buildings in rock, where it is greater than the actual damage threshold.

Crandell[2] used a criterion based on peak energy in the disturbance, which leads to a velocity criterion. He specified a velocity of 3.2 inches/sec. as the beginning of a "caution zone". A velocity of 4.5 inches/sec., is defined as the beginning of the "danger" zone, and it is assumed that this corresponds to the damage threshold, although no substantiating evidence is given. The more recent papers by Langefors, Westerberg and Kihlstrom[4] include a large number of experimental observations of damage to houses by blasting. These show a damage threshold of about 4.5 in./sec. The St. Lawrence results for both longitudinal and vertical components of velocity agree very well with these results. It is worth noting that this is so for both sand-clay and till foundation materials, whereas the similar results of Langefors et al were obtained for houses based on rock. Thus for a variety of foundation conditions, and a corresponding variety of damage mechanism, involving predominant frequencies ranging from 2.5 to 400 cycles/sec., a velocity of 4.5 in./sec. appears to be the threshold of damage.

Observations with Falling Pin Gauge

The pin gauges were set up during blasts at buildings S, T and F, the first of these being in sand-clay terrain and the others in till. At each building the pins fell over before damaging levels were reached. The relevant data are listed in Table III. It is difficult from the rather limited evidence to set a precise threshold vibration level, but it appears that at least some of the pins may be expected to fall when vibration levels are slightly below the damage threshold.

Building Strain

The results of the building strain measurements are shown in Table IV. These appear to be reasonably consistent in that the strain indicated increased with charge. Also large settlement was associated with large dynamic strain and with permanent strain remaining in the wall after the blast. Where settlement was small the strain records indicated that the wall returned to its original condition and there was no remaining permanent strain. The dynamic strain imposed in the wall of House T was insufficient to cause even minor cracking of the wall even though shear cracks occurred in the walls of the basement below. The records showed that the total strain available in the strapping was insufficient to follow the total dynamic strain in the wall in all cases. This caused some flattening of the strain records at the peaks. The measuring system indicates the strains averaged over a very long length of wall and thus may not indicate maximum local strain. The records show very slow variations as compared to the time scale of the disturbance. Thus it appears that the strapping does not follow the sharp peaks in strain.

It is concluded that this method of measuring strain is not wholly satisfactory and requires further study.

Air Blast

Table /1/ shows the measured values of air blast pressure associated with each test. The U.S. Bureau of Mines[6] have found by test that pressures of the order of 100 lb./sq.ft. and greater are necessary to produce window breakage. It will be seen that the measured pressures are well below the level that would cause damage and this is in accordance with the results of the tests. None of the damage that occurred in any of the six structures could be attributed to air blast. An interesting effect occurred during the final blast at House R. Most of the windows in the walls longitudinal to the blast were broken whereas those in the walls transverse to the direction of the blast remained intack, even in the near wall which was only 29 feet away from the blast. The broken windows were attributed to a rocking motion of the frame structure arising from the longitudinal component of the ground vibration.

BUILDING DAMAGE VS. CHARGE AND DISTANCE

The relation between building damage and ground vibration is of interest since it permits a detailed examination of existing criteria. For control of blasting operations, however, it would be simpler if safe limits based directly on explosive charge and distance could be set up. The St. Lawrence results have been examined for a correlation between damage and the parameter $E^{2/3}/d$, and the results are plotted in Fig. /18/. It will be seen that the damage threshold is fairly well defined although it is not quite as good as the correlation between damage and velocity. Fig. /18/ also permits a comparison of the results with the recommendations of Crandell, Morris, and Langefors et al.

Allowing a safety factor it appears that $E^{2/3}/d = 0.1$ might be recommended as a safe limit. It would be of considerable interest to extend the range of the measurements in both directions. For large charges and distances it will be necessary to await occasional large blasting operations. But the interesting case of small charges, at distances less than say 30 ft. can readily be examined. This range has already been considered by Langefors et al, and their recommended safe limit, which extends down to a distance of 3 ft., is shown on Fig. /18/.

CONCLUSIONS

1. The results indicate that there is a well-defined threshold level of vibration above which building damage may be expected. The St. Lawrence work indicates that either acceleration or velocity may be used as an index of damage for the two soil types studied. Considering also the Swedish work in rock it appears that velocity is the quantity more generally applicable, to all soils. Damage is likely to occur with a velocity of 4 to 5

inches per second. A safe limit of 2 inches per second is recommended.

2. In general the vibration records are very complex, and there is no simple and reliable way of inferring the maximum velocity amplitude from displacement or acceleration records. Hence for monitoring purposes a direct measurement of velocity is desirable. This might be done by means of a velocity-sensitive transducer or by using an accelerometer combined with a suitable integrating network. The instrumentation problem is now being studied.

3. For single charges the St. Lawrence studies indicate that the damage threshold is given approximately by $E^{2/3}/d = 0.3$ (where E is weight of explosive in pounds, d is distance in feet). Allowing a factor of safety of three the value of $E^{2/3}/d = 0.1$ is recommended on a safe limit for normal blasting operations. This agrees approximately with a Swedish recommendation, applicable to very small charges and distances. Thus it is believed that the above formula has quite general application for most soils and for a wide range of charges and distances.

   No observations were made for multiple charges using delay systems. It appears from other information, however, that delays of the order of a few milliseconds may produce a cumulative effect somewhat greater than the amplitude due to an individual charge. An additional factor of two should therefore be used for calculating the maximum charge per delay.

4. When it is necessary to operate close to the damage threshold, instrument monitoring is desirable. The safest procedure is to begin with one or more test shots with reduced loading, to determine the energy propagation from source to the structures concerned. The test shots should, however, be placed in the same area as the final large shots, since the vibration amplitude may vary unpredictably with location.

5. The traditional falling-pin gauge was unexpectedly successful as an indicator of the damage threshold. It appears that if any array of 1/4" diameter pins varying in length from 6" to about 18" is used, at least some pins will fall before the damage threshold is reached. A further study of the pin gauge and similar devices is planned.

ACKNOWLEDGEMENTS

Although this has been a joint study it should be noted that the test operations were supported largely by the Hydro-Electric Power Commission of Ontario which, through its Research Division and St. Lawrence Power Project, provided most of the staff and instrumentation and many special facilities and services. The authors are especially appreciative of the interest and cooperation of Mr. Gordon Mitchell, Director of the St. Lawrence Power Project, and his staff, which greatly facilitated the operation.

This report is published with the approval of the Directors of the Research Division, the Hydro-Electric Power Commission of Ontario and the Division of Building Research, National Research Council.

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TABLE I

Characteristics of Buildings Selected For Test

Designation	Building Use	Soil	Superstructure*	Approximate Size (ft.)			Floors (excl. basement)	Estimated Age (years)
				Length	Width	Height To Roof		
C	Church	Sand-Clay	12" brick - plaster inside	51	31	13	1	50
E	House	"	"	43	37 (front) 16 (back)	16	2	50
S	School	"	"	74	28	25	2	50
R	House	Till	Frame and wood siding, wood fibre board inside	23	29	15	2	18
T	House	"	12" brock - plaster inside	51	30	20	2	50
F	House	"	Main part: 12" brick - plaster inside Annex: frame and wood siding - plaster inside	55	24	17	2	50

* All buildings were on basements with walls of stone and mortar 18" to 24" thick.

TABLE II (Part 1)

SUMMARY OF RESULTS

Vertical Component - Sand Clay

Test	Charge	Distance	Acceleration		Displacement		Velocity			Total Settlement	Horizontal Deformation	Damage
			Ampl.	Freq.	Ampl. $\times 10^{-3}$	Freq.	Observed		Calc.			
							Ampl.	Freq.	Ampl.			
	lb	ft	x g	c/s	inches	c/s	in/sec	c/s	in/sec	in.	in.	
C4	120	100	3.6	50	(22)	25			(3.0)	0.06	0.1	none
	120	145	2.5	50	-	-			-	-		none
C5	142	50	6.1	40	(50)	25			(6.4)	0.95	0.35	minor
	142	95	5.6	40	(25)	25			(3.1)			thresh.
E6	92	120	2	85, 150	(57)	2.5			(2.9)	0.02	-	none
	92	155	1.6	85, 150	(43)	2.5			(2.1)			none
E8	280	88	4	130, 70	(130)	2.5			(5.7)	0.11	0.1	none
	280	125	(2.8)	130, 70	-	-			-	-	-	-
E10	140	50	(4.3)	70	(140)	2.5			(6.4)	1.2	0.55	minor
E11	140	25	(7)	40	(240)	2.5			(10.7)	5.1	3	major
S8	260	160	1.6	130, 70	(64)	4.6			-	-	-	none
S10	140	80	2.4	70	(85)	3.2			(3.9)	-	-	none
S12	550	75	15.5	250, 85	(240)	3.2			(10.7)	0.79	0.7	minor
	550	125	(9.2)	250, 85	(140)	3.2			(6.4)	-	-	none

NOTE: Values in parentheses are estimated from data from other related records.

..... (cont'd)

TABLE II (Part 2)

SUMMARY OF RESULTS

Vertical Component - Till

Test	Charge	Distance	Acceleration		Displacement		Velocity			Total Settlement	Horizontal Deformation	Damage
							Observed		Calc.			
			Ampl.	Freq.	Ampl. x 10^{-3}	Freq.	Ampl.	Freq.	Ampl.			
	lb	ft	x g	c/s	inches	c/s	in/sec	c/s	in/sec	in.	in.	
R1	47	200	(0.36)	170, 17	6.7	11			(0.63)	-	0	none
R2	75	75	(1.3)	170, 17	(19)	11			(2.3)	-	0	none
R3	120	29	(47)	170, 17	(84)	11			(8.3)	0.06	0.12	minor
T15	250	120	1.02	57	(10.7)	16			(0.61)	0	0	none
T17	350	80	3.6	50	(36)	16			(4.6)	0.02	0.07	threshold
T18	650	70	5.2	36	(46)	16			(4.3)	0.07	0.15	minor
F19	50	140	0.65	50	17	11			(2.0)	0	0	none
F20	400	90	5.7	130	(86)	11			(10.0)	0.1	0.5	major
	400	95	6.6	130								major
	400	115	45	130	(67)	11			-	-	-	none
F22	750	75	10.5	85	(170)	11			(17)	0.27	1.5	major
	750	70	9.5	64	-	-			-	-	-	major

... cont'd

TABLE II (Part 3)

SUMMARY OF RESULTS

Longitudinal Component - Sand Clay

Test	Charge	Distance	Acceleration Ampl.	Acceleration Freq.	Displacement Ampl. $\times 10^{-3}$	Displacement Freq.	Velocity Observed Ampl.	Velocity Observed Freq.	Velocity Calc. Ampl.	Total Settlement	Horizontal Deformation	Damage
	lb	ft	x g	c/s	inches	c/s	in/sec	c/s	in/sec	in.	in.	
C4	120	100	0.36	43	(60)	25	1.2+	8	(4.8)	0.06	0.1	none
	120	145	0.3	43	-		0.76	-	-	-	-	none
C5	142	50	0.7	64	(140)	25	4.8+	8	(10.6)	0.95	0.35	minor
			0.5	64	(80)	25	34	-	-	-	-	threshold
E6	92	120	0.55	250	(94)	2.5	2.2	8	(1.5)	0.02	-	none
	92	155	0.66	-	(72)	2.5	1.7	-	(1.1)	-	-	
E8	280	88	2.7	125	(180)	2.5	-	-	(3.3)	0.11	0.1	none
E10	140	50	2.6+	70	(200)	2.5	-	-	(7.7)	1.2	0.55	minor
E11	140	25	(8.5)	50	360	2.5	-	-	(16)	5.1	3	major
S8	260	160	0.8	250	-	-	1.3	50	-	-	-	none
S10	140	80	0.8	250	(120)	3.2	2	150	-	-	-	none
S12	550	75	2.8	250	(350)	3.2	-	-	(10)	0.79	0.7	threshold
	550	125	1.7	250	(200)	3.2	-	-	(6.0)	-	-	none

... cont'd

SUMMARY OF RESULTS

Longitudinal Component - Till

Test	Charge lb.	Distance ft.	Acceleration Ampl. x g	Acceleration Freq. c/s	Displacement Ampl. x 10^{-3} inches	Displacement Freq. c/s	Velocity Observed Ampl. in/sec	Velocity Observed Freq. c/s	Velocity Calc. Ampl. in/sec	Total Settlement in.	Horizontal Deformation in.	Damage
R1	47	200	(0.26)	460, 13	10.4	7.7	0.46	-	(0.55)	-	0	none
R2	75	75	(1.0)	460, 13	(38)	7.7	(1.9)	-	-	-	0	none
R3	120	29	(3.5)	460, 13	(130)	7.7	6.8	10	-	0.06	0.12	minor
T15	250	120	1.05	15, 43	(40)	7	(2.9)	-	-	0	0	none
	250	145	0.7	15, 43	-	-	2.4	13	-	-	-	
T17	350	80	2.5	50	(72)	10	10	10,7.3	(7.0)	0.02	0.07	threshold
T18	650	70	4.8	50	(90)	9.5	(10+)	6.5	(4.3)	0.07	0.15	threshold
	650	100	(3.4)	50	(63)	9.5	7+	-	-	-	-	
F19	50	140	0.75	42	25	10	1.4	100,8.5	(1.7)	0	0	none
F20	400	90	5.3 +	170	(75)	9.5	8+	-	(6.7)	0.1	0.5	major
	400	95	4.0	170	-	-	8+	-				major
	400	115	4.1	-	(39)	5	-	-	-	-	-	none
F22	750	75	6.0	85	(128)	9.5	-	-	(12)	-	-	major
		70	8.0	85	-	-	-	-	-	-	-	major

TABLE III

Observations With Falling Pin Gauges

Test	Pin Location	Longitudinal Acceleration x g	Longitudinal Displacement x 10^{-3} inches	Damage	Pins Upset	Longitudinal Velocity inches/sec.
Clay						
S10	Basement	0.8	120	None	8	2.0
S11	"	0.2	220	"	8	1.5
S12	"	1.7	200	Minor at 75'	8	10.0
Till						
T17	"	(1.9)	55	Threshold	3[a]	7.5
T17	Road	(1.0)[b]	8	-	0	2.8
T18	Basement	3.4	(63)	Minor	8	7.0
T18	Road	(0.8)	(22)	-	0	2.5
F19	Basement	0.6	15	None	0	1.4
F20	Basement	4.0	75	Major	8	8.0 +
	2nd floor	3.6		"	8	8.0 +

a - The shortest and the two longest pins fell

b - Values in parentheses are estimated from data from other related records.

TABLE IV

Building Strain Measurements

Test	Charge	Distance	Maximum Strain				Remarks
			Longitudinal Wall		Near Transverse Wall		
			Dynamic	Permanent	Dynamic	Permanent	
	lb	ft	μins./in.	μins./in.	μins./in.	μins./in.	
C4	120	100	150	0	150	0	No damage.
C5	142	50	375	640	1000	150	Settlement - cracked wall and foundation.
S10	140	80	155	0	300	0	No damage.
S11	140	105	80	0	100	0	No damage.
S12	550	75	450	530	650	900 (100 ft. away south wall of room)	Settlement - minor cracking..
T13	15	150					
T14	31	150	13	0	2	0	
T15	250	120	325	0	45	0	No damage.
T16	50	122	60	0	- record missed	0	No damage.
T17	350	80	500	0	250	0	No damage.
T18	650	70	860 +	0	150	0	Minor damage in basement only - some plaster cracks opened slightly.

TABLE V

Air Blast Observations

Building Designation	Charge	Distance	Air Blast Pressure lb/sq ft
R	47	215	1.25
R	120	44'	2.5
Church	No records	-	-
E	260	80	1
E	140	55'	12
School	750	100	2
T 13	15	-	-
14	31	-	-
15	250	-	-
		120+	
16	50	150	5
17	350	100	12
18	600	80	25

Fig. 1. Building R (after Test R3)

Fig. 2. Building E (after Test E11)

Fig. 3. Church.

Fig. 4. Building F.

Fig. 5. Showing tell-tale across an original crack.

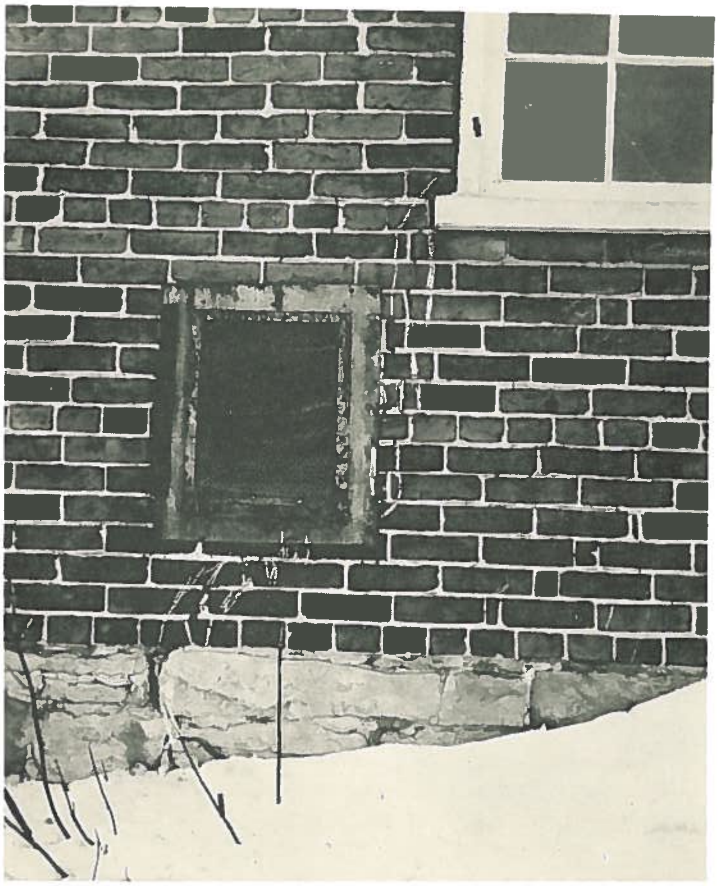


Fig. 6. Vertical crack due to settlement (School)

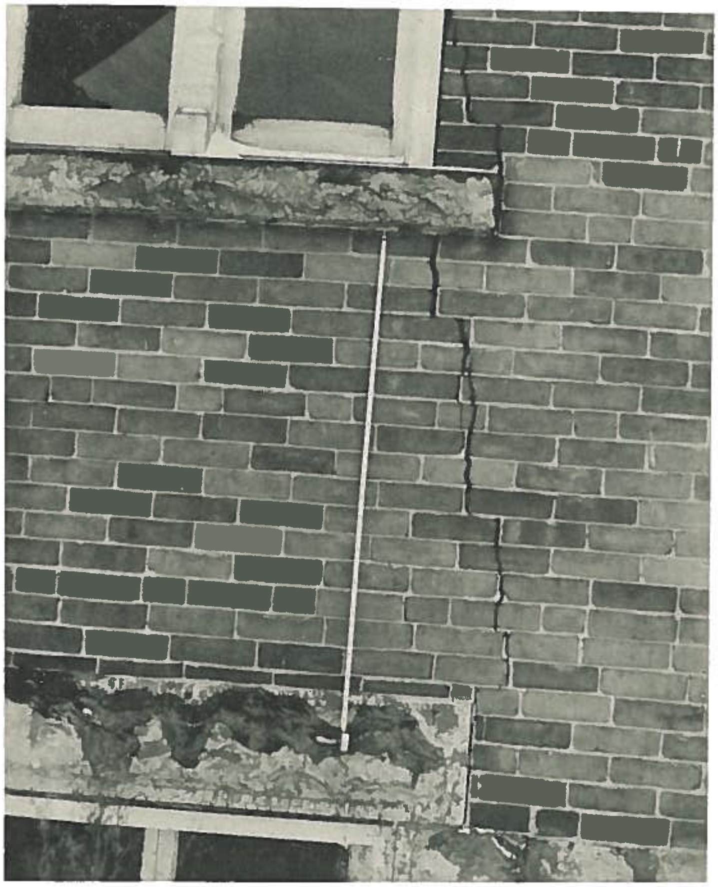


Fig. 7. Vertical crack due to settlement (Church)

Fig. 8. Horizontal crack - Building T.

Fig. 9. Horizontal crack - Building F.

Fig. 10. Chimney Damage - Building R.

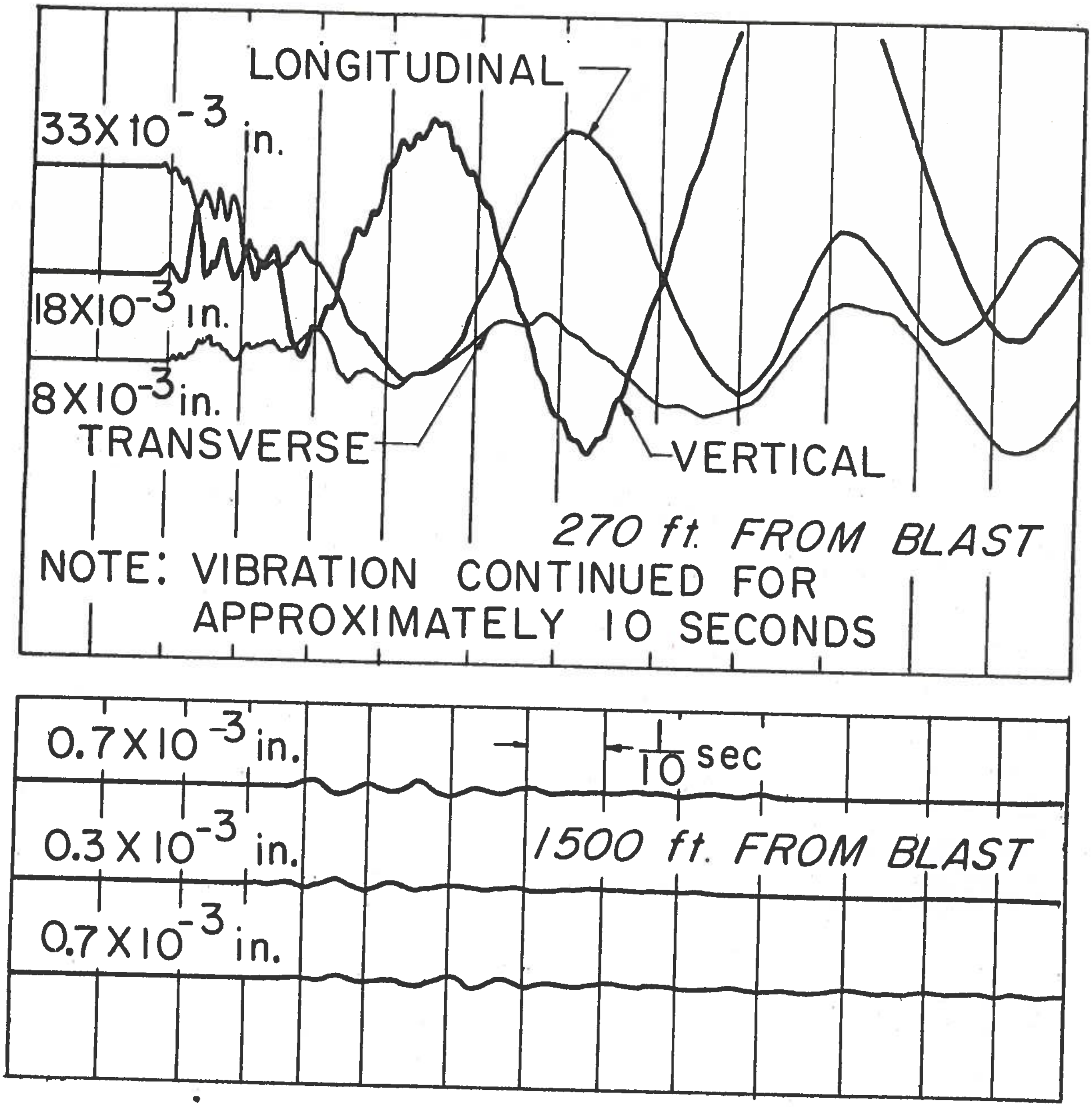


FIGURE 11 a
TYPICAL SAND CLAY RECORD
(DISPLACEMENT)

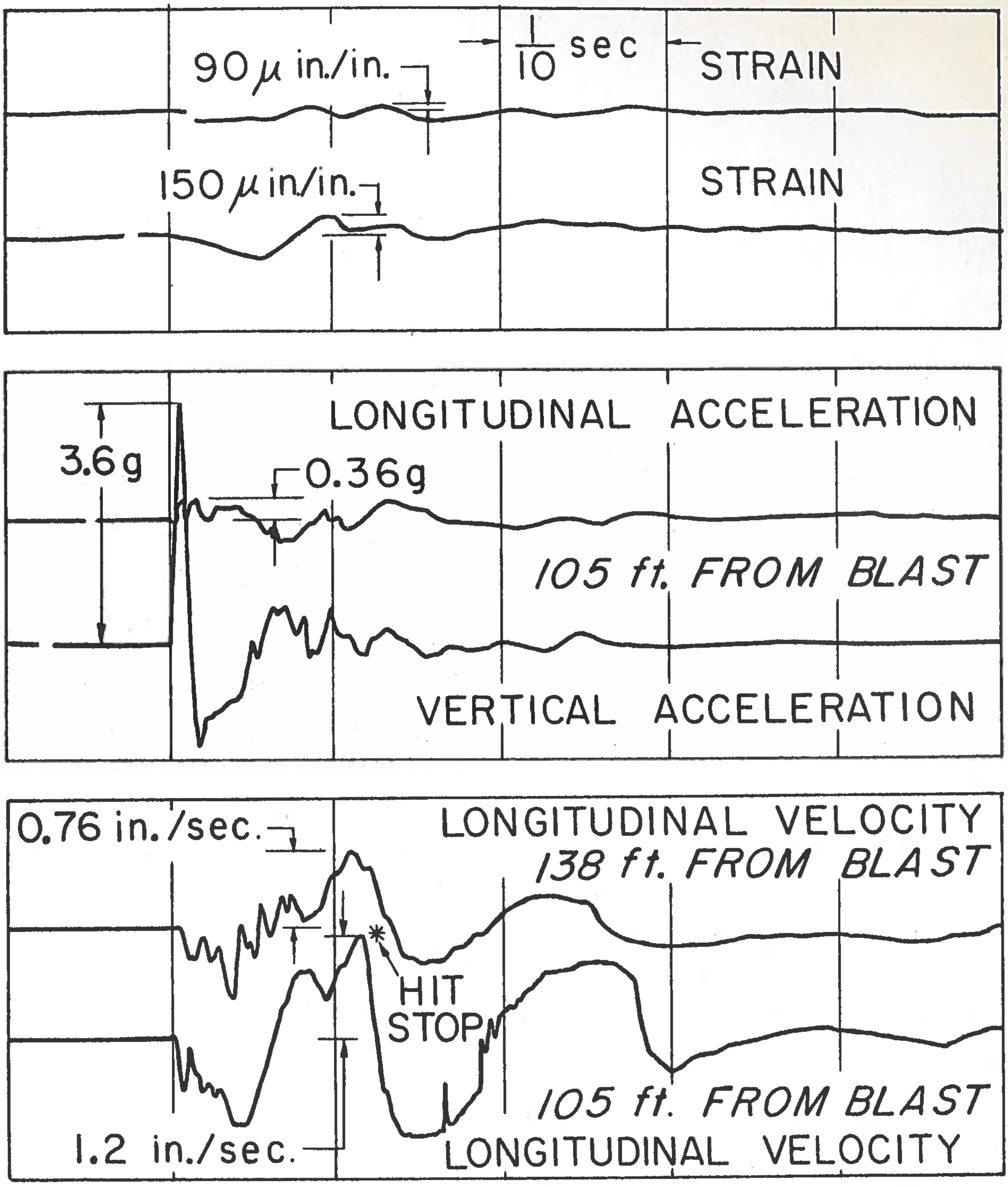


FIGURE 11b
TYPICAL RECORD FOR SAND CLAY
(STRAIN ACCELERATION VELOCITY)

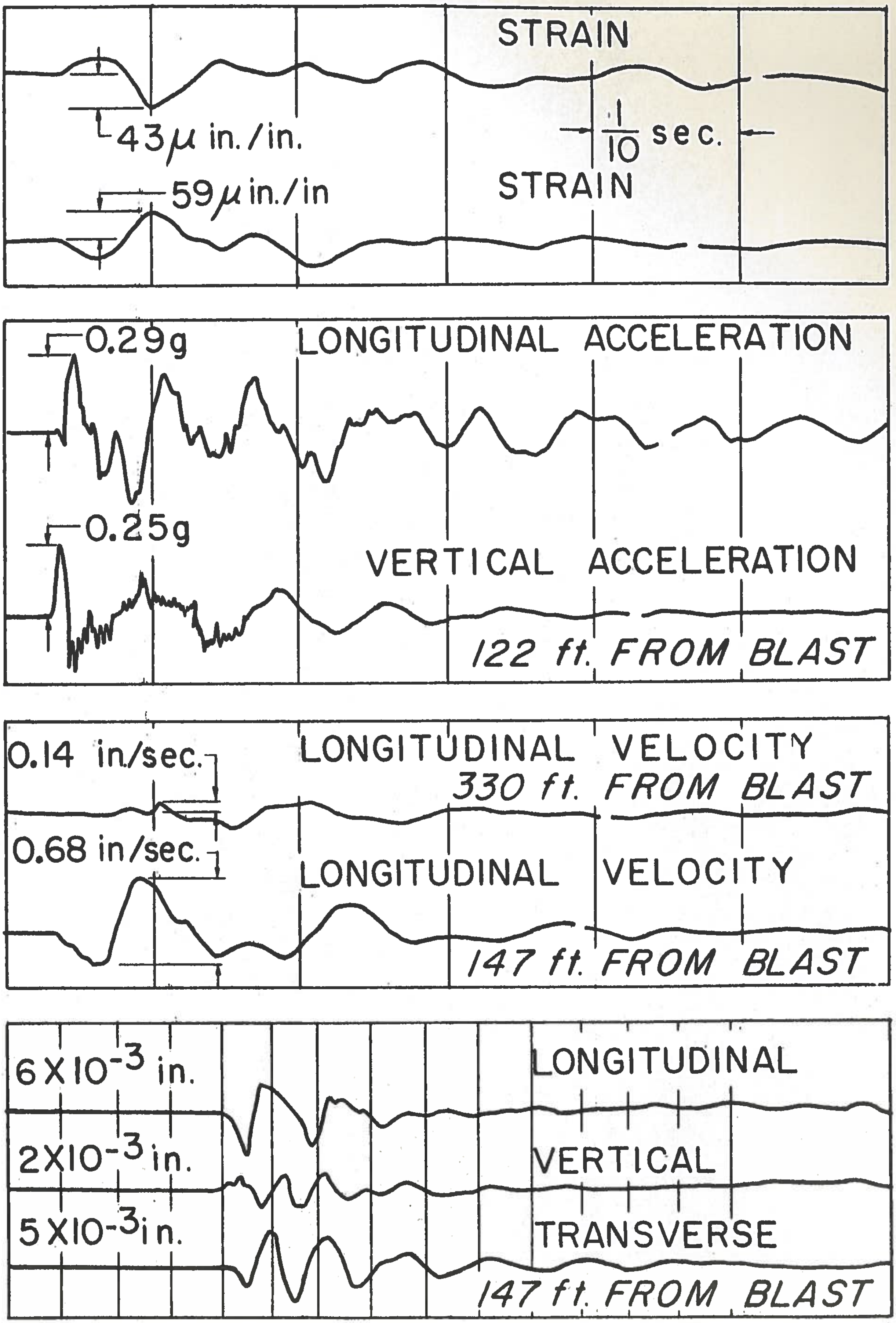


FIGURE 11 C
TYPICAL TILL RECORD

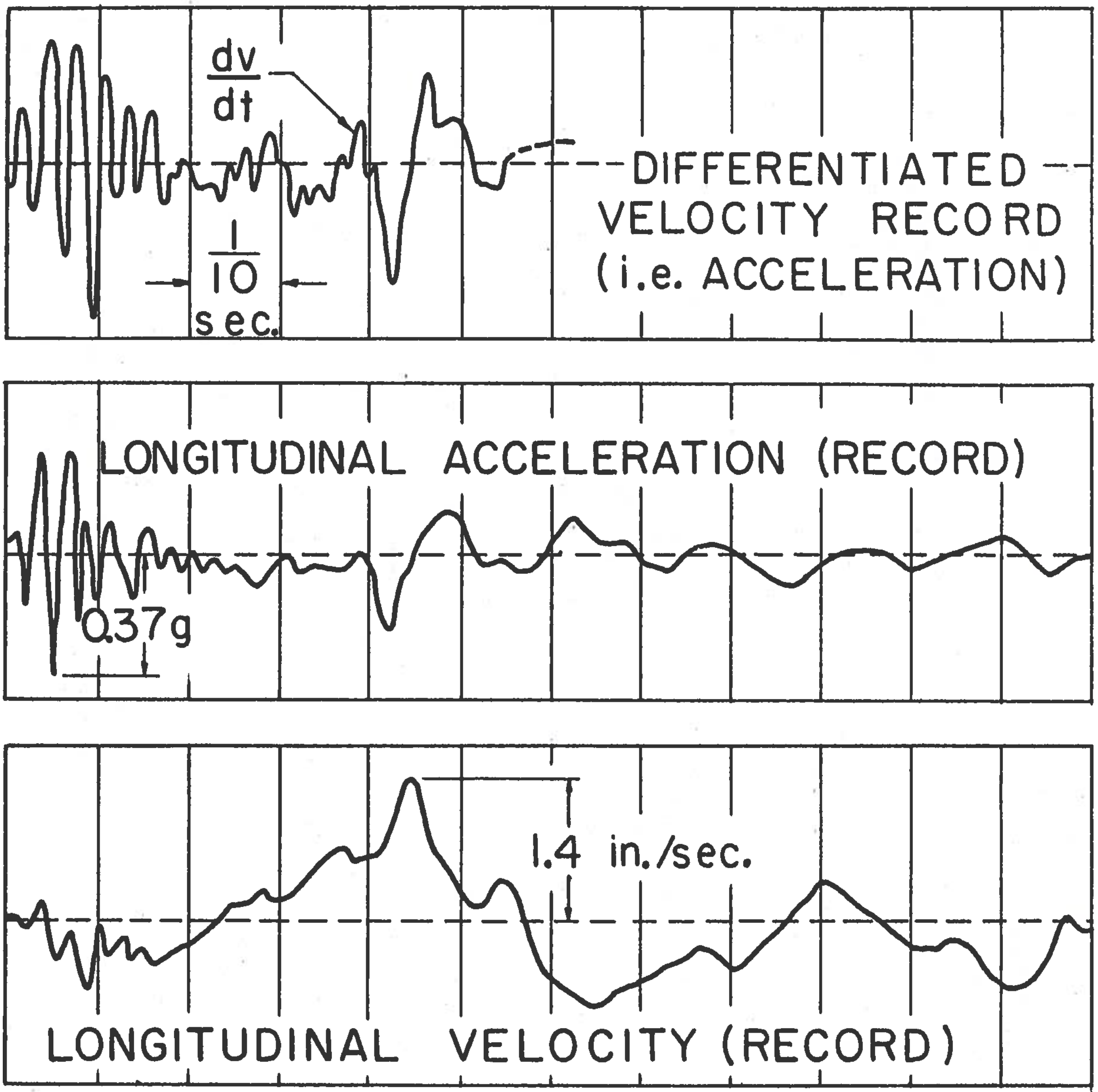
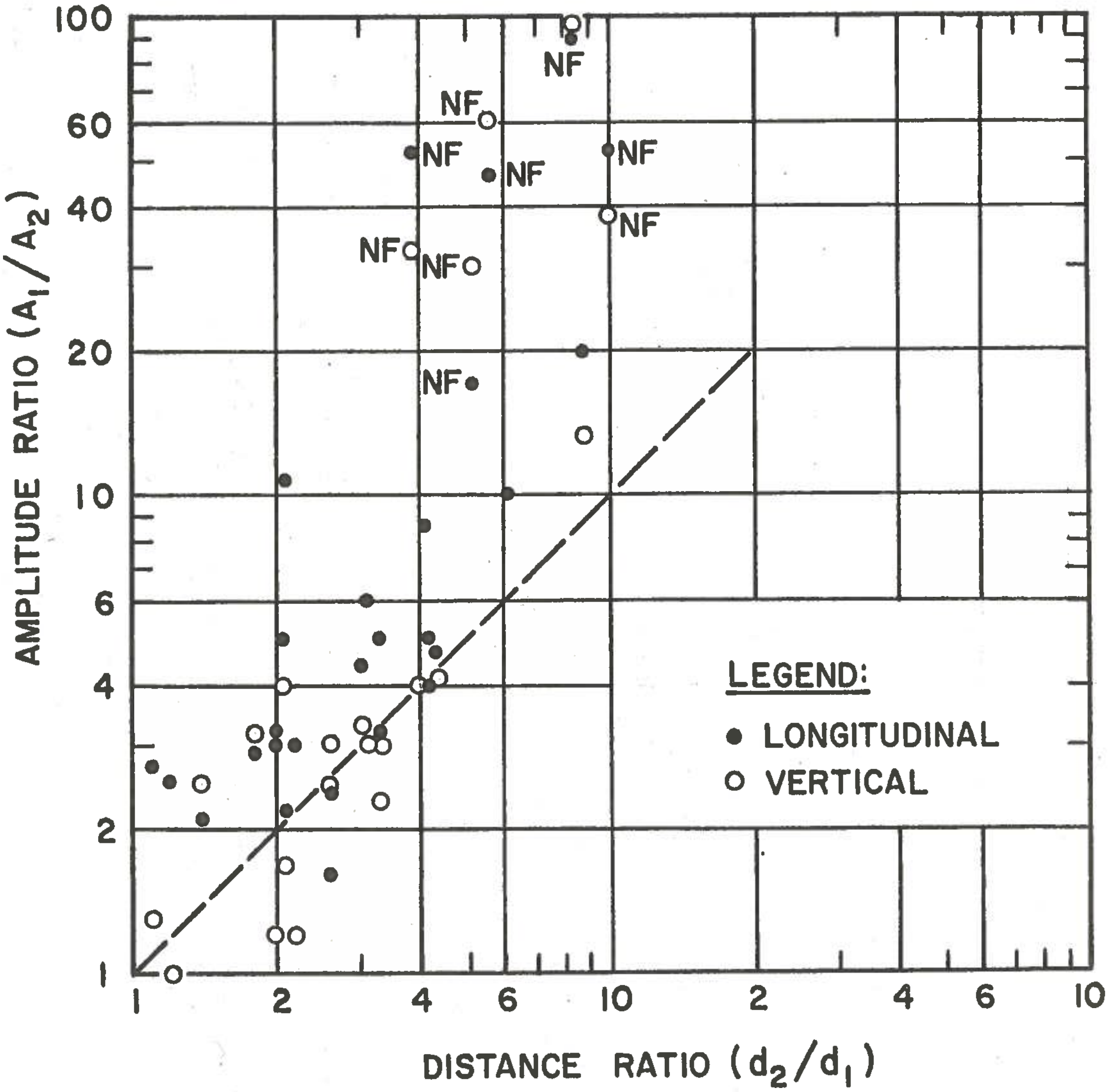
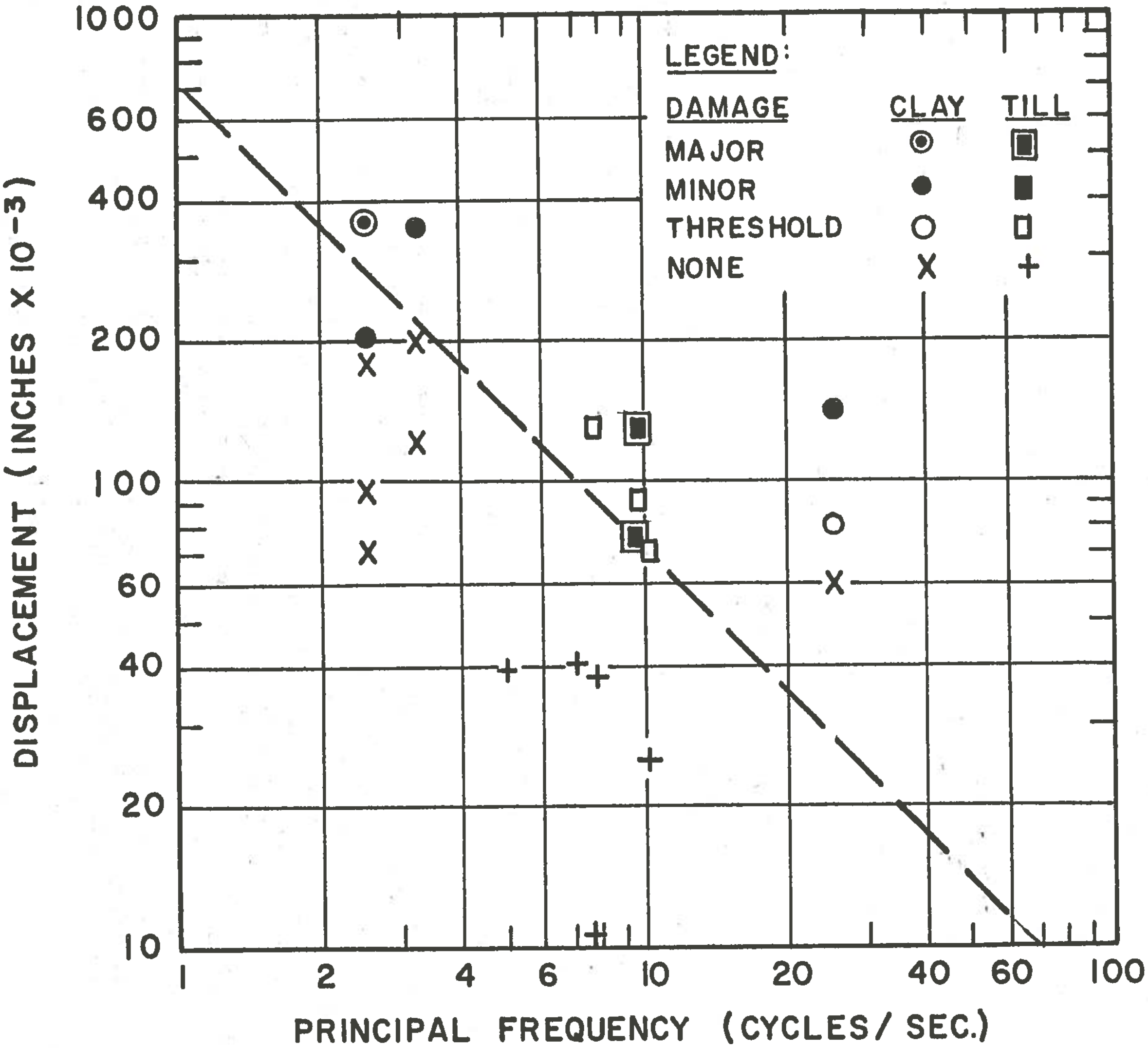


FIGURE 11 d
COMPARISION OF ACCELERATION VELOCITY RECORDS
(15$\frac{1}{2}$ LB. AT 88 FEET IN SAND CLAY)



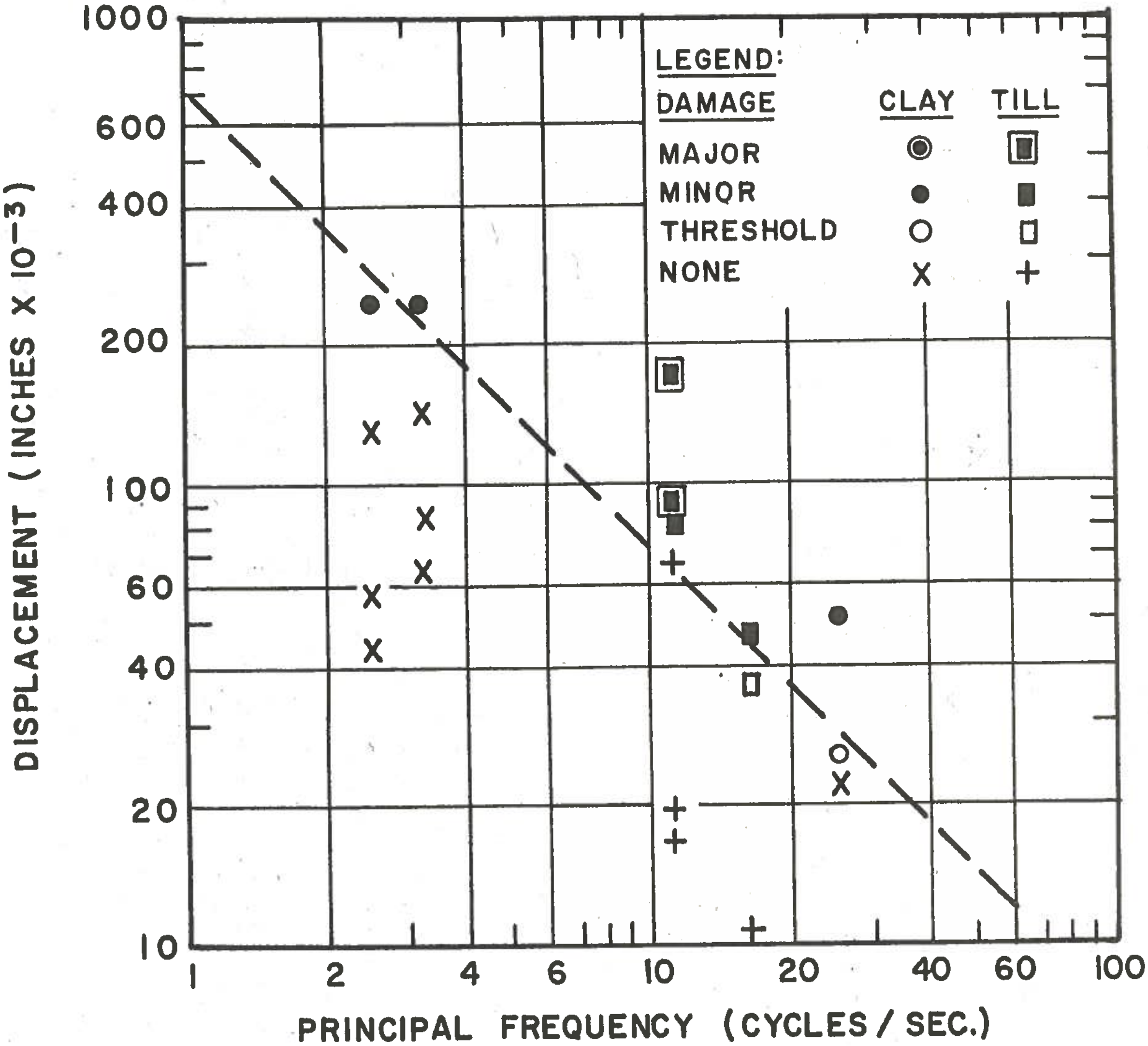
SCATTER DIAGRAM OF AMPLITUDE RATIOS VS DISTANCE RATIOS

FIGURE 12



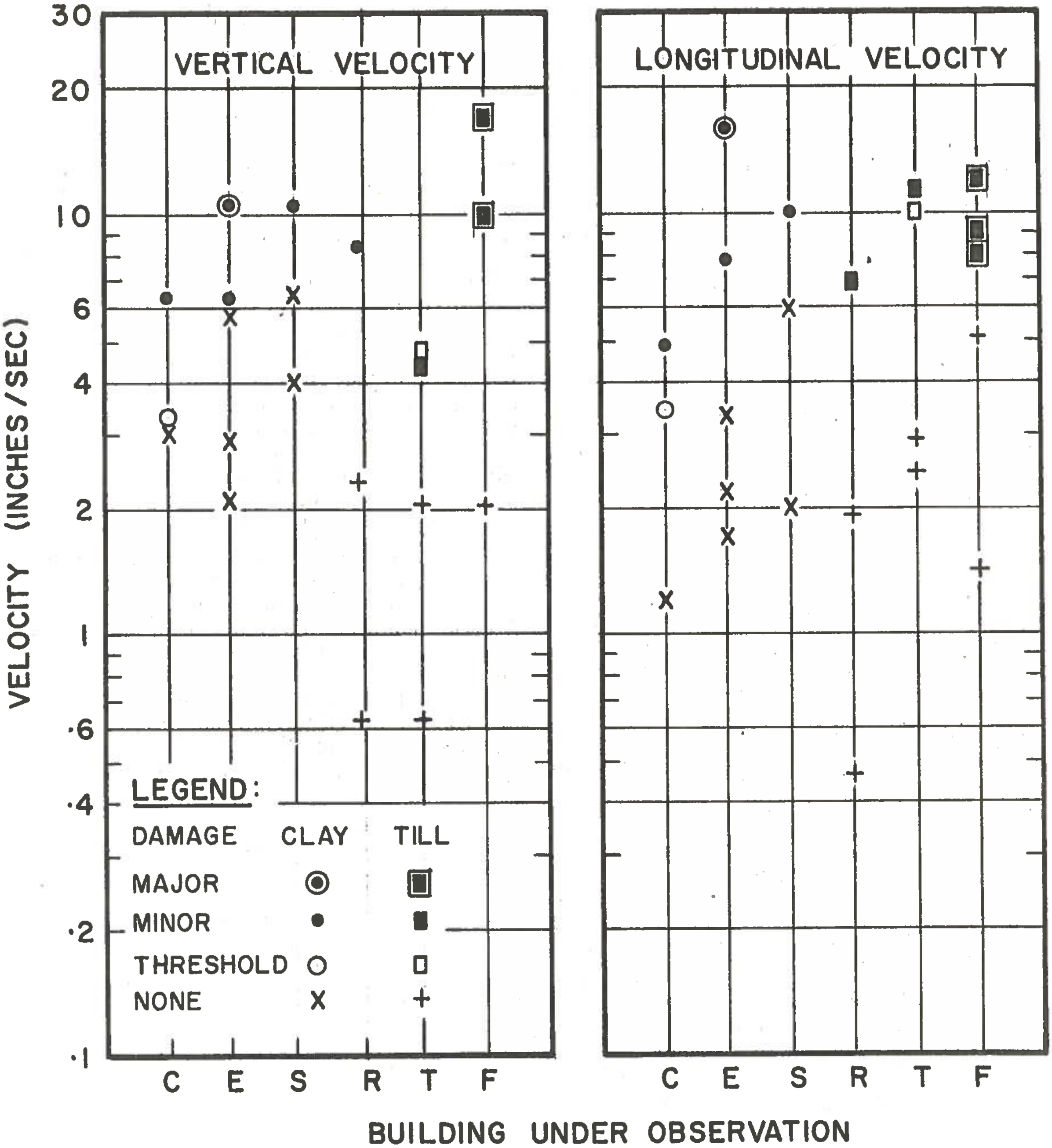
LONGITUDINAL DISPLACEMENT VS. FREQUENCY & DAMAGE

FIGURE 13



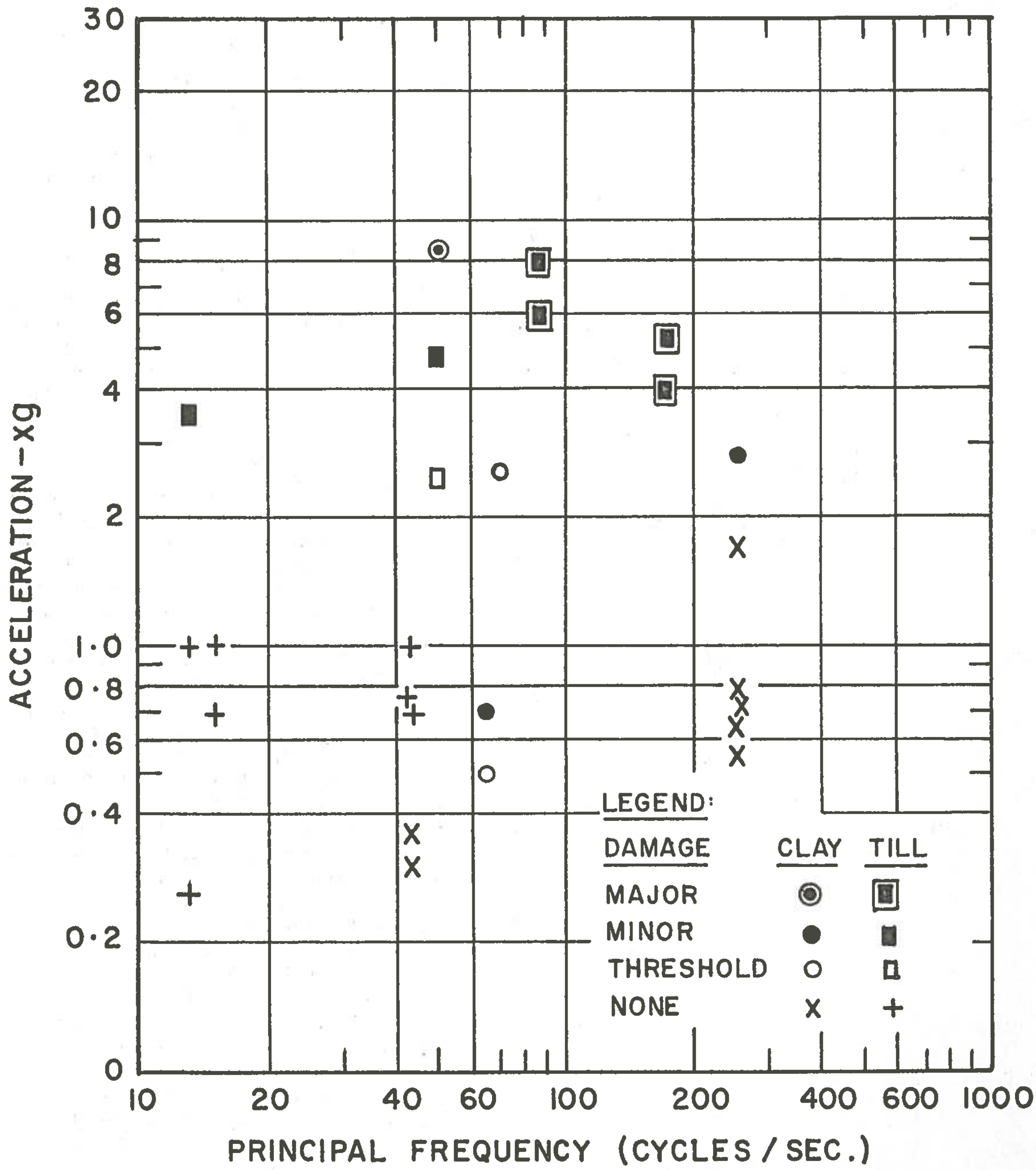
VERTICAL DISPLACEMENT VS. FREQUENCY & DAMAGE

FIGURE 14



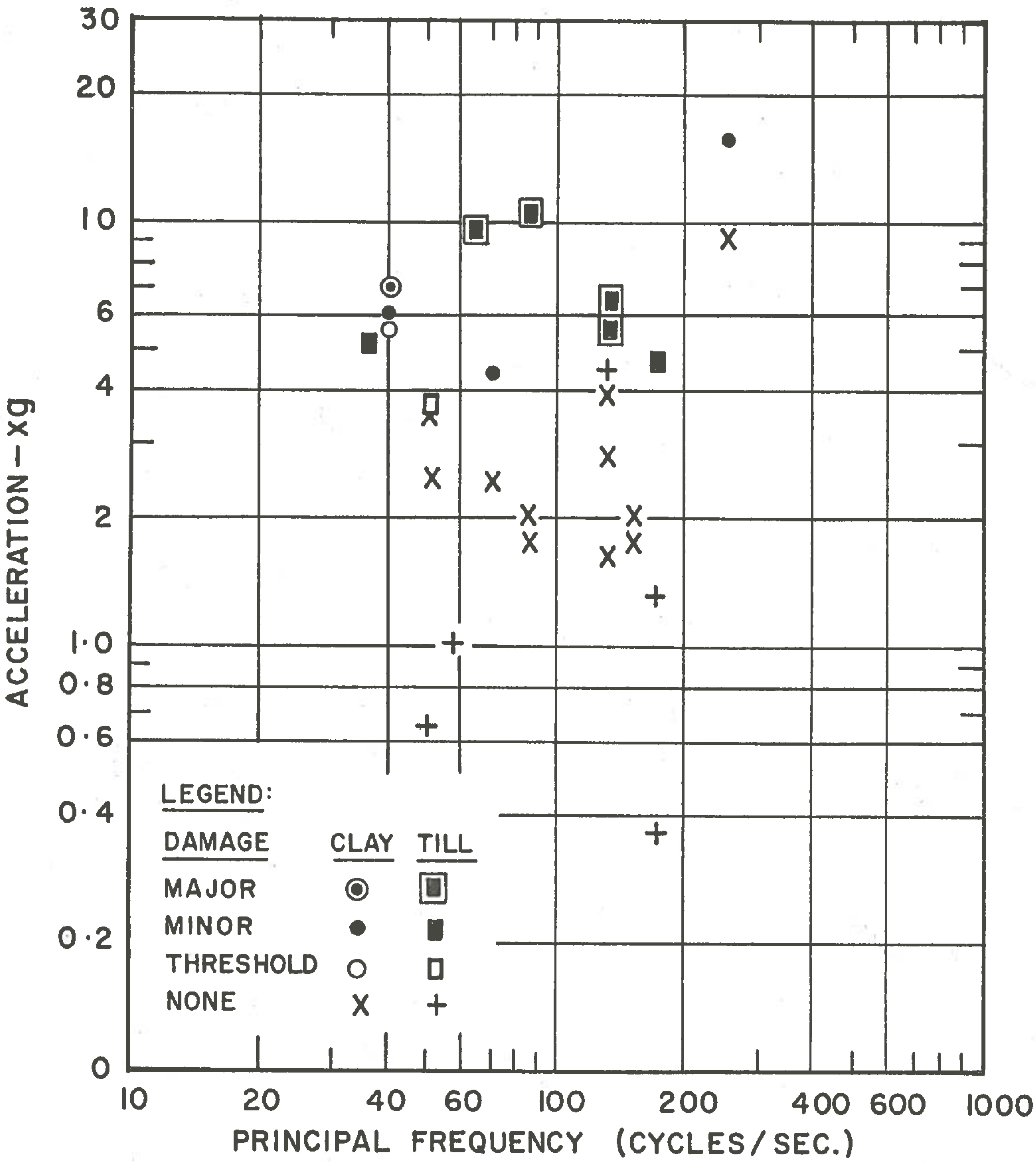
LONGITUDINAL AND VERTICAL VELOCITY VS DAMAGE

FIGURE 15



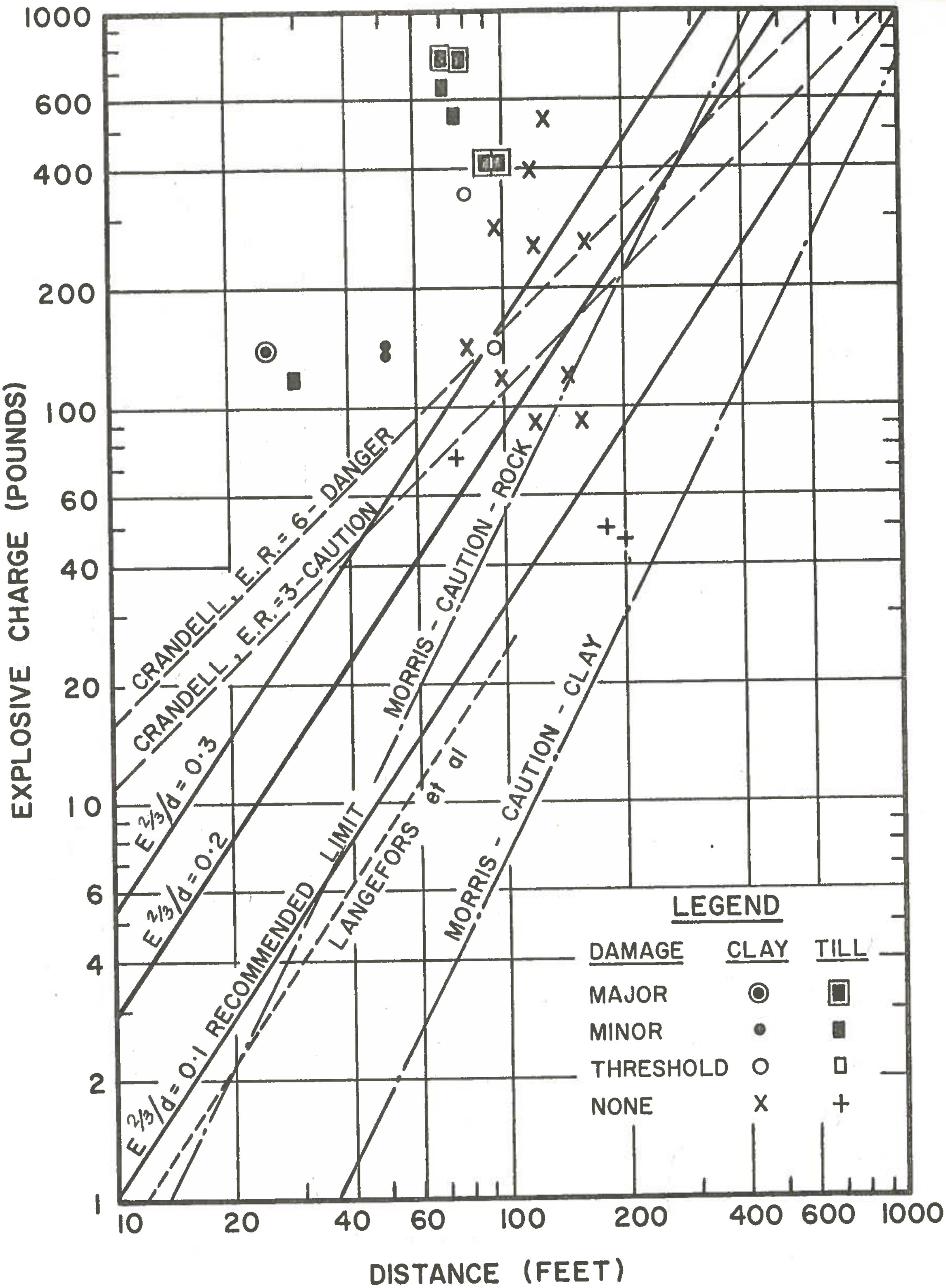
LONGITUDINAL ACCELERATION VS. FREQUENCY & DAMAGE

FIGURE 16



VERTICAL ACCELERATION VS. FREQUENCY & DAMAGE

FIGURE 17



DAMAGE VS CHARGE AND DISTANCE - COMPARSION OF VARIOUS CRITERIA

FIGURE 18

APPENDIX I

Typical Soil Profile

Sand - Clay

Sample No.	Depth in Ft From	To	Soil Classification	Natural Density pcf	Natural Moisture % Dry Wt	Unconf. Shear psf
	0.0	0.4	Datum to ground.			
	0.4	3.0	Loose yellow-brown fine sand.			
1[c]	3.0	4.5	Sand. Loose yellow-brown fine sand.	100.9	12.2	
2[d]	6.0	7.5	Sand. Loose brown fine to medium sand.	115.4	23.0	
3[c]	11.0	12.5	Sand. Loose grey fine to medium sand to 12.2' then alternating 1/8-inch layers of very fine grey silty sand and silt. Containing very small pockets of medium to coarse grey sand.		23.6 19.3	
4[c]	16.0	17.5	Sample lost. Indications of continuing sand.			
5[c]	19.0	20.5	Marine clay. Firm blue-grey silty clay with a trace of very fine sand. Occasional thin sulphide layers.	110.4	44.0	345
6[c]	22.00	23.0	Marine clay. Firm blue-grey silty clay with a trace of fine sand. Occasional sulphide mottling and two 1/4-inch sulphide layers.	114.2	40.5	360
7[c]	25.00	27.0	Marine clay. Firm blue-grey silty clay with a trace of fine sand. Occasional sulphide mottling and two 1/4-inch sulphide layers.	112.2	46.8	380

LEGEND

c - 2 in. Shelby tube.
d - 2 in. Split tube with insert.

Grain Size Distribution

Sand

Millimeters Diameter	Per Cent Passing (by weight)
0.84	100
0.42	96
0.25	55
0.105	2
0.074	1

Silt

Millimeters Diameter	Per Cent Passing (by weight)
0.034	98
0.024	95
0.015	87
0.0093	66
0.0065	57
0.0048	48
0.0021	34
0.0011	23

Typical Soil Profile

Till

Sample No.	Depth in Ft From	Depth in Ft To	Soil Classification	Blow/ft 140 No. Hammer 30 in.	Natural Density pcf	Natural Moisture % Dry Wt	Unconf. Shear psf
1	2.7	4.2	Brown till.	55	138.9	9.6	1110
2	7.7	9.0	Brown till.	57		7.0	
2	9.0	9.2	Grey till.				
3	12.6	14.1	Grey till.	120	155.0	7.7	7111
4	17.5	19.0	Grey till.	31	144.3	8.4	2130
5	22.5	24.0	Grey till.	68		8.0	1690
6	27.4	28.4	Sand layer. Very dense grey silty sand, gravel.	119		7.2	
7	31.8	32.8	Sand layer. Very dense grey fine to medium sand, little silt, gravel.	70			
8			Sand layer. Wash sample. Grey coarse sand, fine gravel.				
	32.8	34.2	Sand layer. Grey fine sand changing to				
9	34.4	35.7	Gray till. Very dense grey silty sand, gravel. Bedrock at 54'.	151	151.0	5.2	4940

Grain Size Distribution

Millimeters Diameter	Per Cent Passing (by weight)	Millimeters Diameter	Per Cent Passing (by weight)
0.001	8	0.84	78
0.005	16	2.00	82
0.01	21	4.7	83
0.05	41	9.4	89
0.105	54	18.8	92
0.25	68	38.1	100

Shear strength of till - remoulded at standard Proctor density
- effective angle of internal friction 39°
- effective cohesion 400 psf.

APPENDIX II

Response of Falling-Pin Gauge to Blasting Vibrations

In its common form a pin gauge consists of a series of rods 1/4 inch diameter by 6 to 18 inches long. These stand upright on a flat level surface in good contact with the ground. It will be assumed that the ends of the rods do not slide on the surface. Only horizontal disturbances will be considered.

Consider the behaviour of one such rod when its base is displaced an amount x_p, resulting in a horizontal force F on the base of the rod sufficiently large to produce a rotation about the point p (see Fig. 19). Taking the rest position of p as the origin, the coordinates of the mass centre are

$$x_G = a \cos (\alpha + \theta) + x_p$$

$$y_G = a \sin (\alpha + \theta)$$

The dynamic equation is

$$Fa \sin (\alpha + \theta) - Ra \cos (\alpha + \theta) = I\ddot{\theta} \qquad (1)$$

where $F = m \ddot{x}_G$ is the horizontal exciting force,

$R = m \ddot{y}_G + m g$ is the vertical reaction at P and

$I = \frac{m h^2}{12}$ is the moment of inertia of a long thin rod about its mid-point. It will be useful to replace $\ddot{x}_p$ by a parameter G such that $\ddot{x}_p = g d G/h$. Noting that $\alpha \doteq \frac{\pi}{2}$ and θ is small in the critical range, the angular functions may be approximated as follows:-

$$\cos (\alpha + \theta) = \frac{d}{2a} (1 - \frac{1}{2} \theta^2) - \frac{h}{2a} \theta$$

$$\sin(\alpha + \theta) = \frac{h}{2a}(1 - 1/2\ \theta^2) + \frac{d}{2a}\theta$$

Substituting in Eqn. 1 with the additional approximation that $h = 2a$

$$\ddot{\theta} + \frac{3g}{2h}\left[\frac{(G-1)\,d}{2h}\theta - \frac{Gd^2}{h^2} - 1\right]\theta - \frac{3gd}{2h^2}(G-1) = 0 \qquad (2)$$

For small values of θ, say up to $\theta = 4d/h$, and for G less than about h/d, the small terms in the coefficient of θ may be neglected. Then a simple approximate equation is obtained:

$$\ddot{\theta} - \frac{3g}{2h}\theta = \frac{3gd}{2h^2}(G-1) \qquad (3)$$

This equation may readily be solved for various types of excitation. But actual blasting disturbances are so random and irregular in form that they do not lend themselves to a direct solution. In lieu of this three very simple types of excitation will be considered.

Some preliminary observations may be made. The equation does not begin to apply until $\ddot{x}_p$ exceeds gd/h (i.e. G exceeds 1), and tilting first occurs. Thereafter it applies as long as θ is positive and small enough that the approximations hold. For typical excitations the rod will be committed to toppling before the approximations cease to hold. The restoring force has a maximum value for θ = o and decreases to zero for $\theta = d/h$; beyond this point it becomes a displacing force.

Case 1 Consider first the effect of a constant acceleration G_1 acting for a time t_1 and then becoming zero. For constant G Eqn. 3 has a solution of the form

$$\theta = ((G_1-1)d/h+\theta_o)\cosh kt_1+(\theta_o/k)\sinh kt_1-(G_1-1)d/h$$

and

$$\dot{\theta}/k = ((G_1-1)d/h+\theta_o)\sinh kt_1+(\dot{\theta}_o/k)\cosh kt_1$$

where $k = (3g/2h)^{\frac{1}{2}}$ and θ_o are initial angular displacement and velocity.

In the present case $\theta_o = \dot{\theta}_o = o$, and the condition of the rod at $t = t_1$ is given by:

$$\theta_1 = ((G_1 - 1) d/h) (\cosh kt_1 - 1) \tag{4}$$

$$\dot{\theta}_1/k = ((G_1 - 1) d/h) \sinh kt_1.$$

If θ_1 and $\dot{\theta}_1$ are large enough the rod may have sufficient energy to carry it beyond the toppling point $\theta = d/h$. This will be so if the subsequent velocity never becomes negative. The velocity after G becomes zero is given by

$$\dot{\theta}/k = (\theta_1 - d/h) \sinh kt + (\dot{\theta}_1/k) \cosh kt \tag{5}$$

and this must be positive for any value of t. The negative term will be most important for large t, when sinh kt approaches cosh kt. Hence the condition for toppling will be:

$$\theta_1 + \dot{\theta}_1/k - d/h \geqslant o. \tag{6}$$

Substituting from (4) and (5), the condition for Case 1 becomes

$$\frac{1}{G} < 1 - e^{-kt_1}.$$

Values of G_1 ($\ddot{x}_{p1}$) and t_1 that will just cause the pin eventually to topple were calculated for pins 6, 12 and 18 inches long, and these result in the solid curves of Fig. 20. Points above and to the right of the curves denote values of $\ddot{x}_{p1}$ and t_1 that will cause the pins to fall.

On the right-hand ordinate are plotted the minimum values of acceleration (applied for infinite time) that will topple the pins. This is the value $\ddot{x}_p = d/h$, i.e. the acceleration just sufficient to tilt the rod from its upright position. Thus, for accelerations lasting for very long times the acceleration required to topple a pin is inversely proportional to its length. This occurs, however, only when the acceleration lasts longer than is usual in blasting disturbances. The slanted portion of the curves, for times less than about 0.2 seconds, is the range of interest. In this region the sensitivity varies as the square root of the pin length.

<u>Case 2</u> Assume that a constant acceleration G_1 acts for a time t_1 and is followed by an equal and opposite acceleration acting for the same length of time. The reversed excitation will of course tend to respore the pin to its upright position, and will succeed unless the positive portion of the excitation was great enough to carry the pin far enough into the toppling region that the negative phase of the excitation cannot bring it back. When the pin moves beyond the toppling point the displacing force due to gravity rapidly becomes large. At the end of the excitation period ($t=2t_1$) velocity of the pin are given by

$$\theta_2 = (d/h)\left[(G_1-1)(\cosh^2 kt_1+\sin^2 kt_1)-2G_1\cosh kt_1+G_1+1\right]$$

$$\dot{\theta}_2/k = (d/h)\left[2(h_1-1)\cosh kt_1 \sinh kt_1 - 2G_1 \sin kt_1.\right]$$

Substituting these in (6) yields as the condition for toppling

$$\frac{1}{G} < 1 - 2e^{-kt_1} + e^{-2kt_1}.$$

This leads to the dashed curves in Fig. 20, which conincide with the solid curves for very long times but are much steeper in the region of interest. It will be noted also that the three curves approach each other in this region. The variation of sensitivity with pin length found for one excursion is largely cancelled out when a second, reverse excursion follows.

Case 3 Consider the effect of a third similar excursion, positive again. Proceeding as before the condition for toppling is found to be

$$\frac{1}{G}<1 - 2e^{-kt_1} + 2e^{-2kt_1} - e^{-3kt_1}.$$

A set of curves is obtained which coincides with the solid curves at both extremes but deviates somewhat at the elbow portion of the curves. One of these (for the 18 inch pin) is shown dotted in Fig. 20.

Observations Regarding the Effect of a Blasting Disturbance.

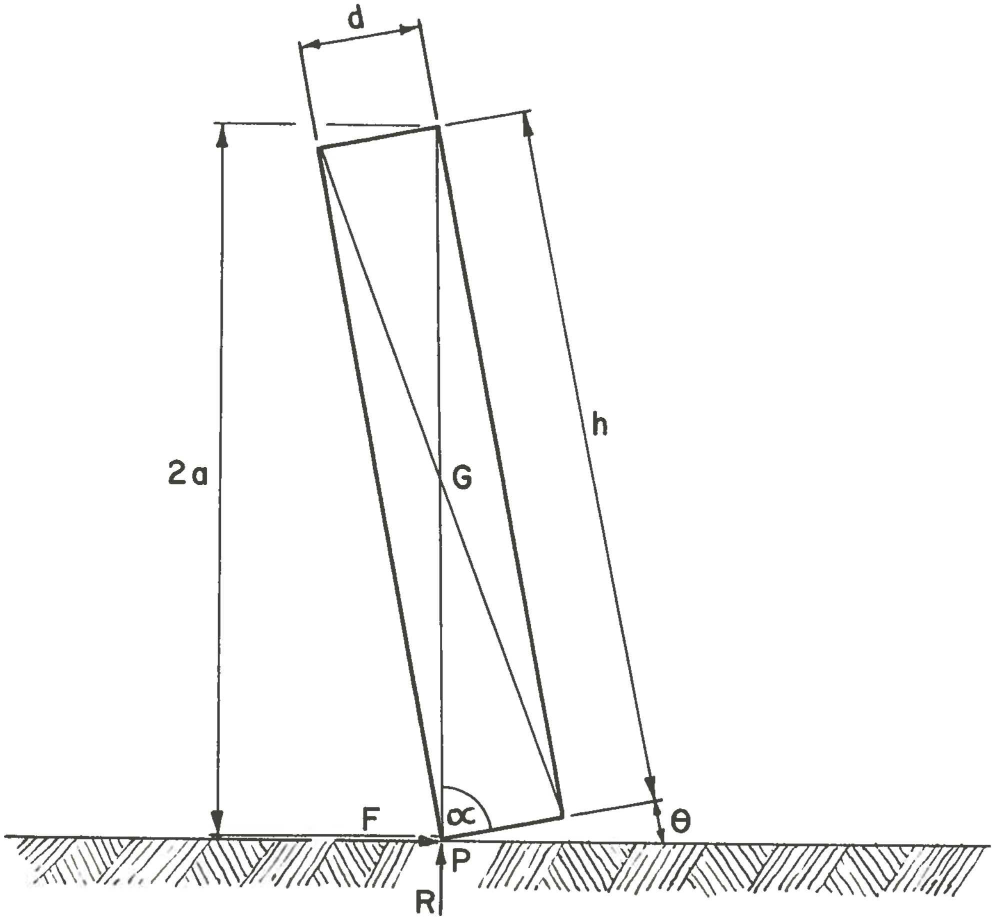
These are three particular, simplified, types of excitation, but they should serve to illustrate the general case. It is apparent that still more extended oscillatory disturbances would cause the threshold values to oscillate back and forth between the solid and dashed curves depending on whether the last half-cycle is positive or negative.

Although blasting vibrations are much more complex than the simple cases considered, one can concentrate on regions of maximum amplitude and assume that the remainder of the disturbance is of minor importance. Usually it will be necessary to consider one maximum excursion followed by two or three reversals of diminishing value. In such cases it is believed that the threshold values for toppling will be bracketted by the two sets of curves of Fig. 2.

Typical maximum excursions on an acceleration record may last from 0.002 to 0.2 seconds. Durations greater than 0.2 for one excursion are uncommon, and hence the sloping portions of the curves are the important part. In this range the acceleration sensitivity varies with duration of the impulse (i.e. with the half-period if the disturbance is oscillatory).

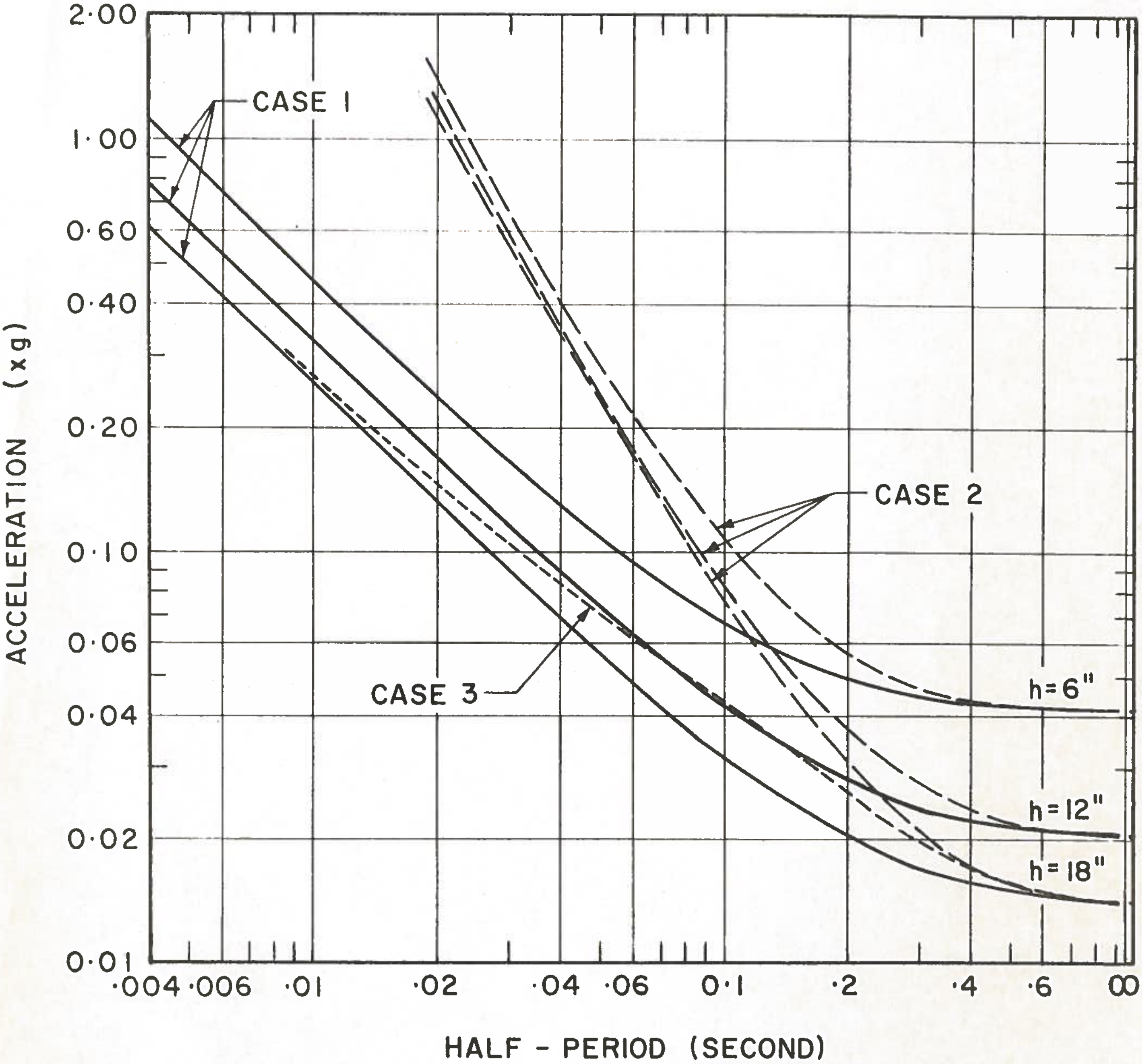
Since the maximum velocity of point p is given by $v_1 = \ddot{x}_{p1}t_1$, a 45 degree line across Fig. 20 denotes a particular value of maximum velocity. The sloping portion of the solid curves have exactly this slope, and indicate that for the single positive excursion of Case 1 maximum belocities from 1.1 to 1.7 inches per second will topple pins from 6 to 18 inches long. This applies for disturbances with a wide range of periods. For the second case, of an additional negative excursion, a similar result holds but for a limited range of half-periods which depends somewhat on the pin length. The threshold values of velocity range from 3.1 to 3.8 inches per second. Case 3 gives threshold values between those for Case 1 and Case 2.

It is concluded that in actual blasting disturbances the toppling threshold will be somewhere between the results for Case 1 and Case 2, depending on the detailed nature of the disturbance.



GEOMETRICAL FORMULATION OF FALLING-PIN PROBLEM

FIGURE 19



AMPLITUDE AND HALF-PERIOD OF ACCELERATIONS RESULTING IN TOPPLING OF PINS OF LENGTH h

FIGURE 20