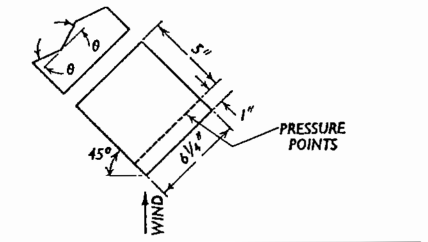
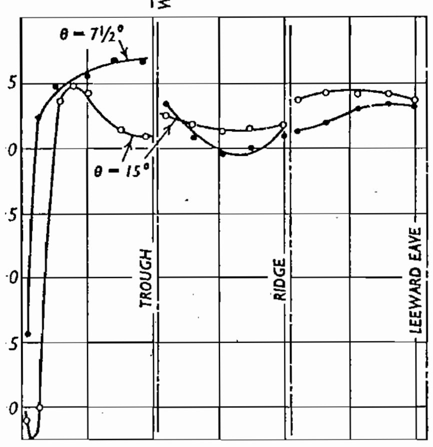
be seen the pressure changes from 3 to +0·7 over one slope.

### 15 Deg. Slope Roofs

ıder normal airflow a 15 deg. triple-span has a suction on the windward slope ar to that of the single-span roof. To eward of this slope the roof is subjected reduced suction, averaging about −0·3. the gable end of the first span high ons occur with diagonal airflow, in one reaching a pressure of −2·0 and ıging −1·4 across the span. Negative ures of over −3·0 were encountered




—Wind pressure over special roofs with small angle of slope

the windward ridge of 15 deg. slope when the ridge was situated at the ward end and the airflow was at 45 deg. occurred with both multi- and single . Fig. 6 illustrates the pressure dision over such a roof, for both 15 deg. ½ deg. slopes.

### Saw-Tooth Roof

ere is no mention of saw-tooth roofs itish Codes, but a Swiss Code of Pracives values of average pressures over roofs for normal airflow and 45 deg. w.

. 7 shows the pressure distribution normal airflow over the centre line of and multispan roofs with the 30 deg. to windward, and again with 60 deg. to windward. It will be seen that the significant positive pressure has an ge value of 0·47. It is on the end . slope when this is to windward. Note rge suctions which occur on the first g. and second 60 deg. slopes when the is from the left and the first 60 deg. econd 30 deg. slopes when the wind m the right.

greater positive pressures were found liagonal winds, though point pressures 35 occur on intermediate 30 deg. and g. slopes. Very high suctions were, er, encountered; the worst occurring the wind direction at 15 deg. to the ends and the 30 deg. slopes to wind-

Under this condition, on the first . slope, the average negative pressure 2·0 near the gable end decreasing to 5 on the centre line. With the wind in verse direction, but at 25 deg. to the ends, an average suction of 1·4 was

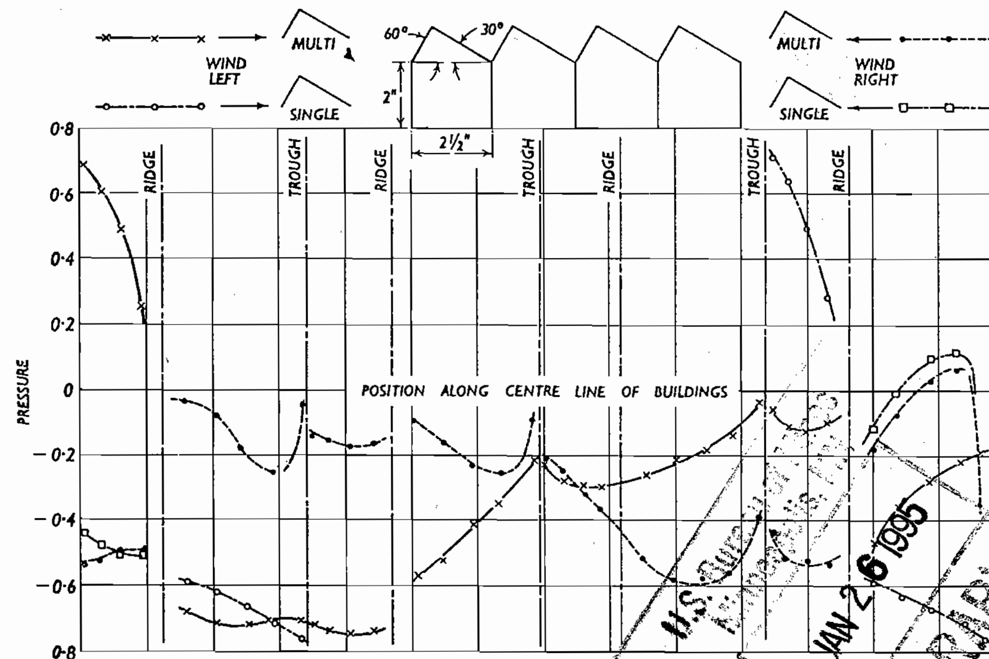



Fig. 7—External pressure over saw-tooth roofs subjected to normal airflow



found on the first 30 deg. slope. The maximum recorded negative pressure was −2·2 and indications suggested that this would have risen to about −2·5 had a pressure point been situated at the appropriate spot.

### Conclusions

With multispan roofs there are very definite pressures, normally negative, on the intermediate spans which should be taken into account in design. On all roofs, but especially with multispan ones, very high positive and negative pressures may arise with a diagonal wind. These far exceed the values normally considered which generally take into account, only normal airflow. On full-scale buildings, even without the possible effect of internal pressure, the upward forces are, in many cases, considerably greater than many roof weights at winds of, say, 70 m.p.h. There is little doubt that some of the lightweight buildings being designed to-day would, if not attached to foundations, be completely blown away in a hurricane. Newspaper reports show that this has already happened in Australia.

*The photograph on Page 536 is reproduced by permission of Fairey Air Surveys, Ltd.*

# Experimental Studies of the Effects of Blasting on Structures

By A. T. EDWARDS* and T. D. NORTHWOOD†

*The results are presented of controlled blasting tests on 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.*

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 very little has been known about actual building damage due to actual blasting operations. The number of buildings accidentally damaged by blasting is very small, and the number for which there is reliable information about both the damage and the blasting operations is still smaller. 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

* Research Engineer, the Hydro-Electric Power Commission of Ontario, Toronto, Canada.
† Research Physicist, Division of Building Research, National Research Council, Ottawa, Canada.

Fig. 1—Building R (after Test R3)

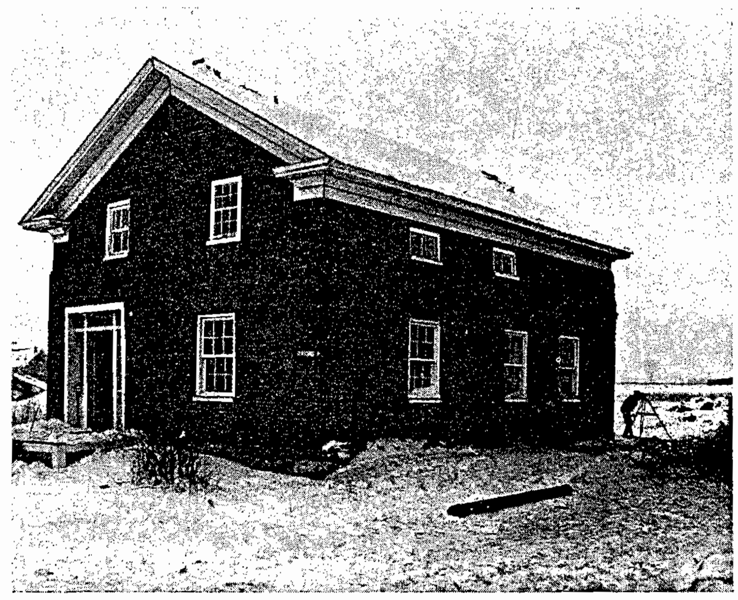


Fig. 2—Building E (after Test E11)

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 found on rock. The two studies together thus provide evidence for a wide range of soil conditions.

In addition to an investigation of 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 make predictions 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 the St. Lawrence tests 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 instrumentation was occasionally used, including that for a few measurements of strain in building walls.

### Description of Structures and Soil Conditions

Six structures on two different types of soil were used in the tests. Three of the structures were on a loose wet sand, about 20ft deep, under which was soft marine clay. The water table at the time of the tests was about 7ft below grade. This soil condition will be referred to as sand-clay for the purpose of this article. 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 the 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 1ft at the time of the tests. The structures are briefly described and designated in Table I. Photographs of buildings R, E, C, and F are given in Figs. 1 to 4.

All structures were in good condition except for quite localised areas in one or two of them. Building R and part of building F were of frame construction above masonry basement walls. In the other buildings, structures above ground level were mainly of 12in 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 deg. 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 may be seen in Fig. 2. Rock was encountered about 15ft down at house F.

### Instrumentation

The instruments used for the investigation will be discussed in four groups, namely shock measuring instruments, recording

Fig. 3—Church

Fig. 4—Building F

TABLE I—*Characteristics of Buildings Selected for Test*

| Designation | Building use | Soil | Superstructure* | Approximate size (ft) Length | Width | Height to roof | Floors (excluding basement) | Estimated age (years) |
|---|---|---|---|---|---|---|---|---|
| C | Church | Sand-clay | 12in brick—plaster inside | 51 | 31 | 13 | 1 | 50 |
| E | House | Sand-clay | 12in brick—plaster inside | 43 | 37 (front) 16 (back) | 16 | 2 | 50 |
| S | School | Sand-clay | 12in brick—plaster inside | 74 | 28 | 25 | 2 | 50 |
| R | House | Till | Frame and wood siding, wood fibre board inside | 23 | 29 | 15 | 2 | 18 |
| T | House | Till | 12in brick; plaster inside | 51 | 30 | 20 | 2 | 50 |
| F | House | Till | Main part: 12in 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 18in to 24in thick.

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 owing 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 record displacements from about 0·001in to 0·02in. The Sprengnether instrument has a magnification of 320 in the vertical direction and 180 in the horizontal direction. It records displacements from about 0·0001in to 0·005in. The moving elements of both instruments have a fundamental natural frequency of about 1 c/s and are critically damped. The Leet and Sprengnether instruments weigh 70 lb 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 c/s, 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 suitably proportioned loading resistance. It is a single component instrument, but by a simple adjustment it can measure velocity in either vertical or horizontal direction. In this work it was used to measure motion only in the longitudinal direction. The instrument was connected via a suitable resistance network directly to a galvanometer element in a multi-channel recorder.

(*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 was used for these tests. Three of the transducers had natural frequencies of the order of 400 c/s, two were at 250 c/s and one at 110 c/s. The recording system had a frequency response that 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 ground 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 a shifting of 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. When measuring large amplitudes the Leet seismographs were anchored with chains and turnbuckles fastened to the supporting slab.

(2) RECORDING EQUIPMENT

Apart from displacement records from the seismographs, records were obtained on a photographic type of multi-channel 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 1,000 c/s. The Brush equipment is approximately flat from 0 to 100 c/s.

(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

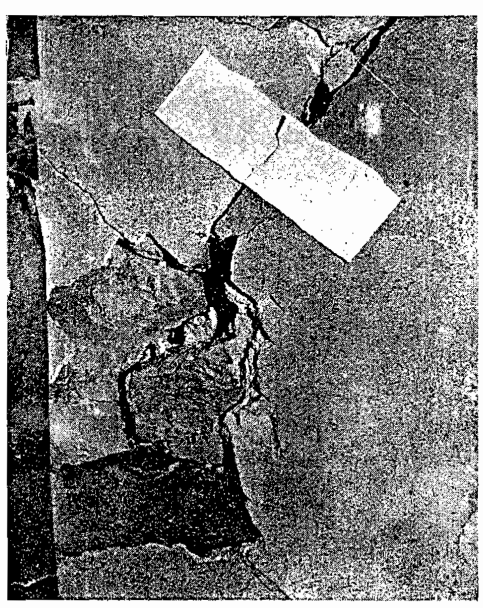


Fig. 5—Showing tell-tale across an original crack

was pasted across each crack (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. They 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 caused by 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 pretensioning 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 7,500 c/s.

(*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 ¼in diameter rods ranging from 6in to 15in in length. These are placed on end on a carefully levelled flat plate. Each pin is provided with a rigidly supported cylindrical casing so that one pin falling 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 8in. A more detailed analysis (in preparation) indicates that the response of the pin depends more on the waveform of the disturbance than on the length of the pin. For the complex vibration that typically occurs there is about equal probability 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 was then 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 analysis of the results 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 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 100ft 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 150ft and succeedingly larger charges progressively closer, the minimum distance in most cases being not less than 50ft from the structure. Thus the soil between the individual charges and the structure was undisturbed. The holes varied from 15ft to 30ft 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 15ft and 25ft apart, arranged to produce approximately a plane wave disturbance representative of a distant blast source. The explosives used were 75 per cent Forcite (Canadian Industries, Ltd.) and 60 per cent Special Gel (Dupont), 4in to 5in 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.

### Definition of Damage

One can visualise 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).

### (1) Buildings on Sand-Clay

*Building C.*—There was no noticeable damage from Test C4 (120 lb at 100ft). Damage from C5 (142 lb at 50ft) was mainly in the form of vertical cracks, from hairline to $\frac{1}{4}$in in width, in the two walls closest to the blast. One of these ($\frac{1}{4}$in 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.

*Building E.*—Damage first occurred with Test E10 (140 lb at 50ft) 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 $\frac{1}{4}$in in width. Test E11 (140 lb at 25ft) 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 1in in width.

*Building S.*—Following Test S12 (550 lb at 75ft), cracks in the brickwork were mainly vertical and varied from hairline to about $\frac{3}{8}$in wide. These were all in the section, about 25ft in length, closest to the blast. The adjoining section some 50ft in length was completely undamaged. The basement door frame, which originally was in poor condition, settled about an inch.

### (2) Buildings on Till

*Building R.*—No damage occurred until Test R3 (120 lb at 29ft) when some horizontal cracks, up to about $\frac{1}{32}$in wide, developed in the basement walls. These extended out in the two walls, longitudinal with the blast, about 12ft from the rear 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.

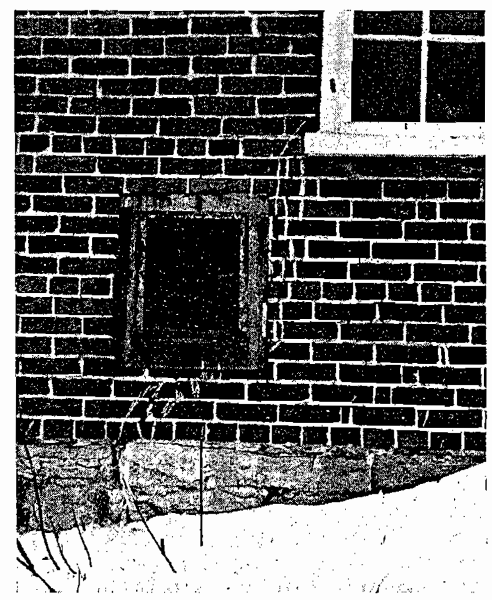


Fig. 6—Vertical crack due to settlement (school)

*Building T.*—Test T17 (350 lb at 80ft) 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 (650 lb at 70ft). This caused some minor horizontal and vertical cracks in the basement walls, these being generally between courses or associated with windows, &c. Additional bricks were dislodged from the chimney.

*Building F.*—Test F20 (400 lb at 90ft) caused some minor horizontal cracking in the

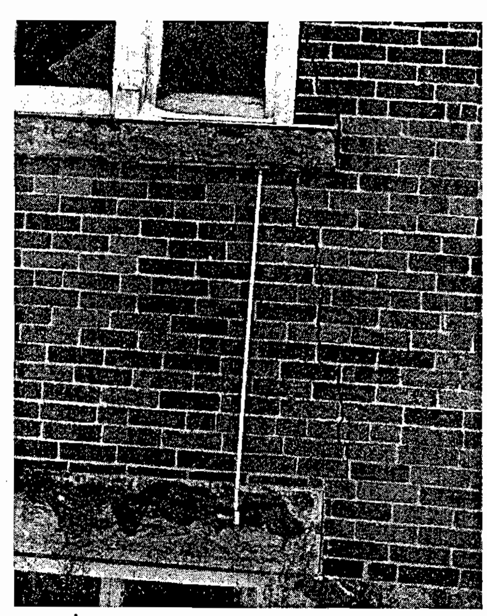


Fig. 7—Vertical crack due to settlement (church)

Fig. 8—Horizontal crack—building T

Fig. 9—Horizontal crack—building F

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 (750 lb at 70ft). Considerable sections of the rear basement wall fell away and the masonry walls above ground level were bulged out approximately 3in. Some of the upstairs partition walls became separated from the outside walls and there were a number of cracks $\frac{1}{16}$in to $\frac{3}{8}$in, 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 exemplified by Figs. 8 and 9. It is interesting that chimneys are sometimes the first part of a building to show signs of weakness. Fig 10 shows an example that occurred at building R.

Fig. 10—Chimney damage—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.

INTERPRETATION OF VIBRATION 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, however, as exemplified by Figs. 11 (*a*), (*b*) and (*c*), will indicate that this is a much over-simplified picture. Fig. 11 (*d*) shows a typical set of

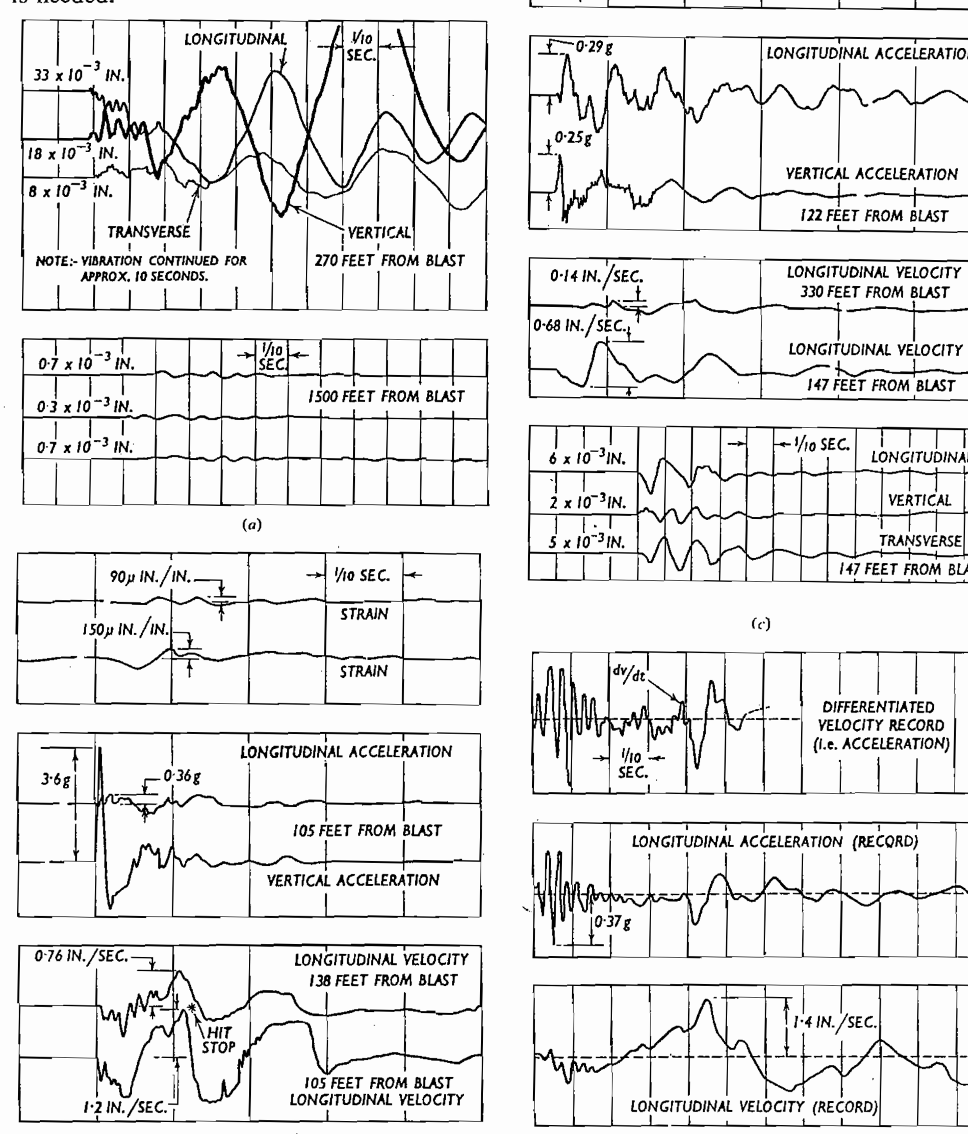



Fig. 11(a), (b), (c) and (d)—Typical records of acceleration and velocity records of acceleration and velocity obtained at the same observation point for the same blast. The records are quite different in character, and an attempt to determine the most characteristic frequency involves a rather arbitrary 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 records shows that the two records do correspond reasonably well. The numerical integration and differentiation of such records is a tedious process, however, 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 maximum 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.*—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 reaction 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). By a selection of observations that minimised or eliminated the extraneous variables, however, 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 because of the extraneous variables.

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 and $E_1$ and $E_2$ 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 were available, including longitudinal and vertical components, of acceleration and displacement, in till and in clay. There was no systematic difference between longitudinal and vertical components. Differences between acceleration and displacement and between till and clay were barely significant, statistically speaking, with slightly higher values for acceleration than for displacement, for till than for clay. Considering the experimental conditions no great reliance is placed on these distinctions. Combining the intermediate results, weighting them according to their precision indices, an overall average value of $n=0 \cdot 67$, with a standard deviation of $0 \cdot 05$, was obtained. This is in agreement with the value $n=\frac{2}{3}$ given by Thoenen and Windes rather than the values used by Morris ($n=\frac{1}{2}$) or Crandell ($n=1$).

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 $m=1 \cdot 8$, with a probable error of $0 \cdot 2$. was obtained, but the distribution of the observations was unsymmetrical, beginning with a large number of values very close to $m=1 \cdot 0$ 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 records. In the sand-clay area, there was a sustained large low-frequency vibration (2·5 c/s) within a few hundred feet of the source which did not occur at all at 1,500ft and beyond (Fig. 11(*a*)). 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

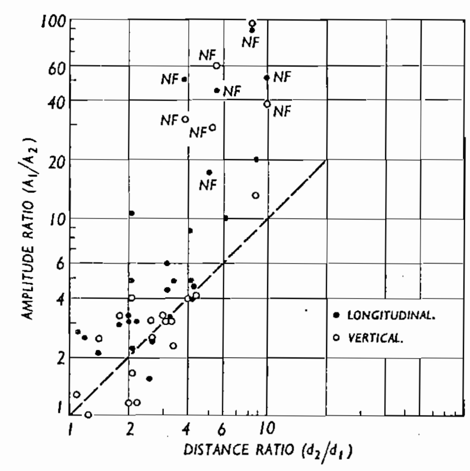



Fig. 12—Scatter diagram of amplitude ratios versus distance ratios

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 the rate of diminution does not suggest a Rayleigh wave or other type of surface wave, for which $m$ would be about $0 \cdot 7$. Only four observations gave values of $m$ less than $0 \cdot 9$.

### Observations of Damage versus 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, utilising a calibration test in the building combined with results for a

Table II—*Summary of Results*

I—*Vertical Component—Sand Clay*

| Test | Charge (lb) | Distance (ft) | Acceleration: Amplitude (× g) | Acceleration: Frequency (c/s) | Displacement: Amplitude (×10⁻³ in) | Displacement: Frequency (c/s) | Velocity: Observed Amplitude (in/sec) | Velocity: Observed Frequency (c/s) | Velocity: Calculated Amplitude (in/sec) | Total settlement (in) | Horizontal deformation (in) | Damage |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| C4 | 120 | 100 | 3·6 | 50 | (22) | 25 | | | (3·0) | 0·06 | 0·1 | None |
| | 120 | 145 | 2·5 | 50 | — | — | | | — | — | — | None |
| | 142 | 50 | 6·1 | 40 | (50) | 25 | | | (6·4) | 0·95 | 0·35 | Minor |
| C5 | 142 | 95 | 5·6 | 40 | (25) | 25 | | | (3·1) | — | — | Threshold |
| 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 |

* Values in parentheses are estimated.

II—*Vertical Component—Till*

| Test | Charge (lb) | Distance (ft) | Acceleration: Amplitude (× g) | Acceleration: Frequency (c/s) | Displacement: Amplitude (×10⁻³ in) | Displacement: Frequency (c/s) | Velocity: Observed Amplitude (in/sec) | Velocity: Observed Frequency (c/s) | Velocity: Calculated Amplitude (in/sec) | Total settlement (in) | Horizontal deformation (in) | Damage |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| R1 | 47 | 200 | (0·36)* | 170, 17 | 6·7 | 11 | | | (·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 |

* Values in parentheses are estimated.

*III—Longitudinal Component—Sand Clay* TABLE II (*continued*)—*Summary of Results*

| Test | Charge (lb) | Distance (ft) | Acceleration: Amplitude (× g) | Acceleration: Frequency (c/s) | Displacement: Amplitude (× 10⁻³ in) | Displacement: Frequency (c/s) | Velocity: Observed Amplitude (in/sec) | Velocity: Observed Frequency (c/s) | Velocity: Calculated Amplitude (in/sec) | Total settlement (in) | Horizontal deformation (in) | Damage |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 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 |

* Values in parentheses are estimated.

*IV—Longitudinal Component—Till*

| Test | Charge (lb) | Distance (ft) | Acceleration: Amplitude (× g) | Acceleration: Frequency (c/s) | Displacement: Amplitude (× 10⁻³ in) | Displacement: Frequency (c/s) | Velocity: Observed Amplitude (in/sec) | Velocity: Observed Frequency (c/s) | Velocity: Calculated Amplitude (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 |

* Values in parentheses are estimated.

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 in a few cases for which acceleration records were integrated to obtain maximum velocity.

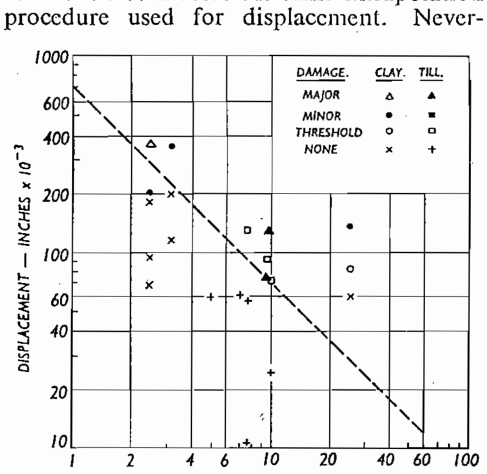


Fig. 13—Longitudinal displacement versus frequency and damage

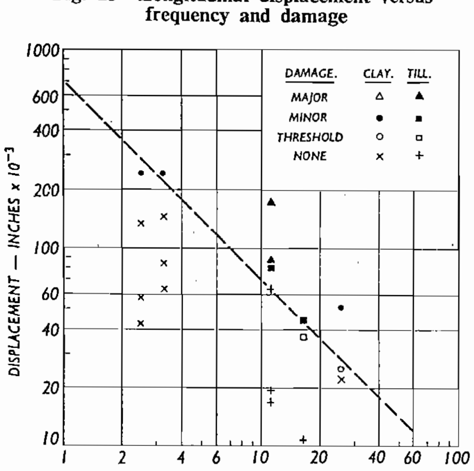


Fig. 14—Vertical displacement versus frequency and damage

Figs. 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 4.5in per second, 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.

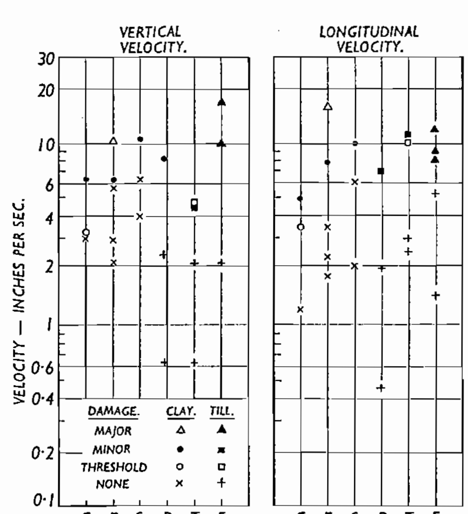


Fig. 15—Longitudinal and vertical velocity versus damage

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 4in per second.

The acceleration results are shown in Figs. 16 and 17. Although the results are plotted

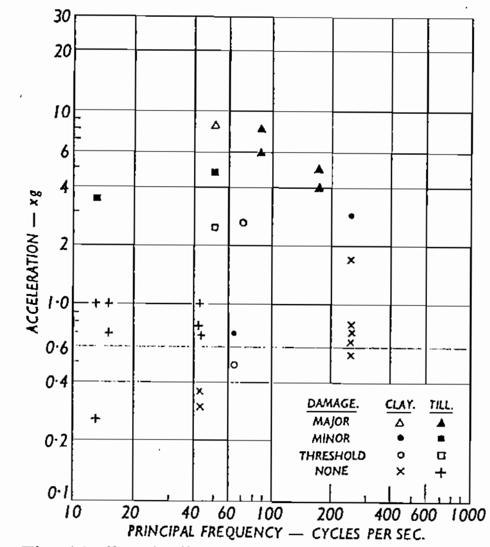


Fig. 16—Longitudinal acceleration versus frequency and damage

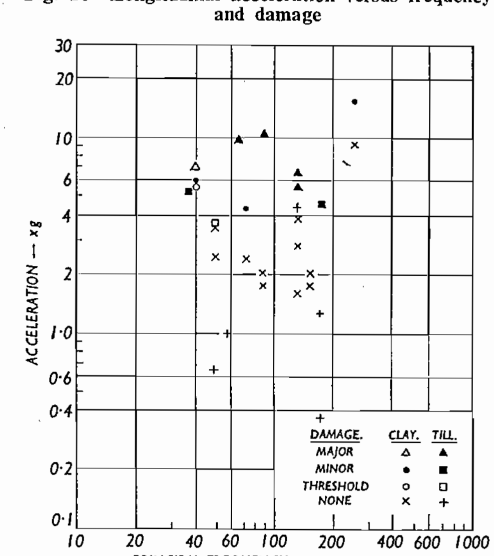


Fig. 17—Vertical acceleration versus frequency and damage

against frequency it should be remembered that there is 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 $0 \cdot 7\ g$. Only one case was reported of damage due to a blasting operation. This produced threshold damage at a displacement amplitude of about 0·1in which corresponds to about the middle of the threshold versus frequency curves obtained for the St. Lawrence studies. The threshold acceleration values obtained in the St. Lawrence study[1] were substantially higher than the vibrator result obtained by Thoenen and Windes ($2\ g$ to $4\ g$ as compared with $0 \cdot 7\ g$). 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 $8 \times 10^{-3}$in. More recently Morris and Westwater,[7] on the basis of a few observations of damage to buildings, estimated that the actual damage threshold is about $40 \times 10^{-3}$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 $1 \cdot 6 \times 10^{-3}$). Thus it would appear that Morris' recommended limit is conservative except for buildings in rock, where it is rather close to 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 3·2in per second as the beginning of a "caution zone." A velocity of 4·5in per second 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 4·5in per second. 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 mechanisms,

TABLE III—*Observations With Falling Pin Gauges*

| Test | Pin location | Longitudinal acceleration ($\times g$) | Longitudinal displacement ($\times 10^{-3}$in) | Longitudinal velocity (in/sec) | Damage | Pins upset |
|---|---|---|---|---|---|---|
| Clay | | | | | | |
| S10 | Basement | 0·8 | 120 | 2·0 | None | 8 |
| S11 | Basement | 0·2 | 220 | 1·5 | None | 8 |
| S12 | Basement | 1·7 | 200 | 10·0 | Minor at 75ft | 8 |
| Till | | | | | | |
| T17 | Basement | (1·9)[b] | 55 | 7·5 | Threshold | 3[a] |
| T17 | Road | (1·0) | 8 | 2·8 | — | 0 |
| T18 | Basement | 3·4 | (63) | 7·0 | Minor | 8 |
| T18 | Road | (0·8) | (22) | 2·5 | — | 0 |
| F19 | Basement | 0·6 | 15 | 1·4 | None | 0 |
| F20 | Basement | 4·0 | 75 | 8·0+ | Major | 8 |
| | 2nd Floor | 3·6 | — | 8·0+ | Major | 8 |

(a) The shortest and the two longest pins fell.
(b) Values in parentheses are estimated.

TABLE IV—*Building Strain Measurements*

| Test | Charge (lb) | Distance (ft) | Maximum Strain: Longitudinal wall, Dynamic ($\mu$ in/in) | Maximum Strain: Longitudinal wall, Permanent ($\mu$ in/in) | Maximum Strain: Near transverse wall, Dynamic ($\mu$ in/in) | Maximum Strain: Near transverse wall, Permanent ($\mu$ in/in) | Remarks |
|---|---|---|---|---|---|---|---|
| C4 | 120 | 100 | 150 | 0 | 150 | 0 | No damage |
| C5 | 142 | 50 | 375 | 640 | 1,000 | 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 (100ft 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 | 8,604 | 0 | 150 | 0 | Minor damage in basement only—some plaster cracks opened slightly |

involving predominant frequencies ranging from 2·5 to 400 c/s, a velocity of 4·5in per second 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 information is 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, and 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. The records showed that the total strain available in the strapping was insufficient to follow the total dynamic strain in the wall. 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 with the time scale of the disturbance, and it is supposed that the strapping does not follow the sharp peaks in strain. It is concluded that this method of measuring strain is not wholly satisfactory.

AIR BLAST

Table V shows the measured values of air-blast pressure associated with each test. The

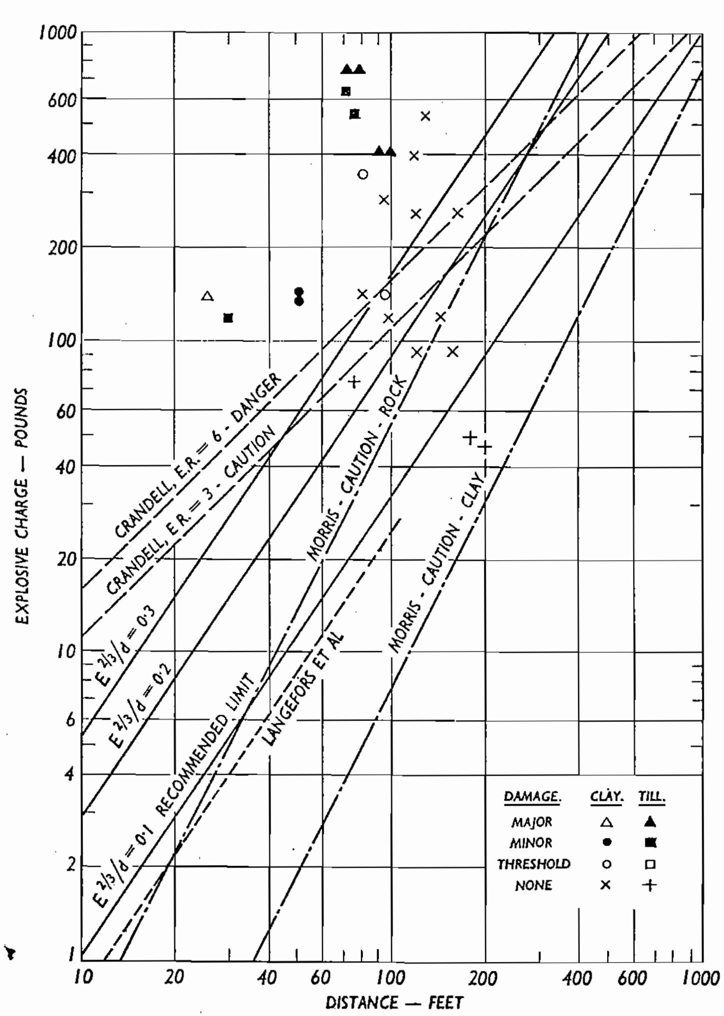



Fig. 18—Damage versus charge and distance—comparison of various criteria

U.S. Bureau of Mines[6] has found by test that pressures of the order of 100 lb per square foot 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.

TABLE V—*Air Blast Observation*

| Building designation | Charge (lb) | Distance (ft) | Air blast pressure (lb per square foot) |
|---|---|---|---|
| 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 |
| T13 | 15 | — | — |
| 14 | 31 | — | — |
| 15 | 250 | — | — |
| | | 120+ | |
| 16 | 50 | 150 | 5 |
| 17 | 350 | 100 | 12 |
| 18 | 600 | 80 | 25 |

TABLE VI—*Typical Soil Profiles for Sand, Clay, and Till*

*Sand and Clay*

| Sample No. | Depth (ft) | Soil classification | Natural density, (per cubic foot) | Natural moisture, (per cent dry weight) | Unconf. shear strength, (per square foot) |
|---|---|---|---|---|---|
| | 0·0 to 0·4 | Datum to ground | | | — |
| | 0·4 to 3·0 | Loose yellow-brown fine sand | | | |
| 1[c] | 3·0 to 4·5 | Sand. Loose yellow-brown fine sand | 100·9 | 12·2 | |
| 2[d] | 6·0 to 7·5 | Sand. Loose brown fine to medium sand | 115·4 | 23·0 | — |
| 3[c] | 11·0 to 12·5 | Sand. Loose grey fine to medium sand to 12.2ft then alternating ¼in layers of very fine grey silty sand and silt. Containing very small pockets of medium to coarse grey sand | | 23·6 | |
| 4[c] | 16·0 to 17·5 | Sample lost. Indications of continuing sand | | 19·3 | |
| 5[c] | 19·0 to 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] | 22·0 to 23·0 | Marine clay. Firm blue-grey silty clay with a trace of fine sand. Occasional sulphide mottling and two ¼in sulphide layers | 114·2 | 40·5 | 360 |
| 7[c] | 25·0 to 27·0 | Marine clay. Firm blue-grey silty clay with a trace of fine sand. Occasional sulphide mottling and two ¼in sulphide layers | 112·2 | 46·8 | 380 |

([c]) 2in Shelby tube. ([d]) 2in split tube with insert.

*Till*

| Sample No. | Depth (ft) | Soil classification | Blow/ft 140 lb 30in drop | Natural density (per cubic foot) | Natural moisture, (per cent dry weight) | Unconf. shear strength, (per square foot) |
|---|---|---|---|---|---|---|
| 1 | 2·7 to 4·2 | Brown till | 55 | 138·9 | 9·6 | 1,110 |
| 2 | 7·7 to 9·0 | Brown till | 57 | — | 7·0 | — |
| 2 | 9·0 to 9·2 | Grey till | — | — | — | — |
| 3 | 12·6 to 14·1 | Grey till | 120 | 155·0 | 7·7 | 7,111 |
| 4 | 17·5 to 19·0 | Grey till | 31 | 144·3 | 8·4 | 2,130 |
| 5 | 22·5 to 24·0 | Grey till | 68 | — | 8·0 | 1,690 |
| 6 | 27·4 to 28·4 | Sand layer. Very dense grey silty sand, gravel | 119 | — | 7·2 | — |
| 7 | 31·8 to 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 to 34·2 | Sand layer. Grey fine sand changing to | — | — | — | — |
| 9 | 34·4 to 35·7 | Grey till. Very dense grey silty sand, gravel. Bedrock at 54ft | 151 | 151·0 | 5.2 | 4,940 |

*Grain Size Distribution*

| Sand | | Silt | |
|---|---|---|---|
| Diameter millimetres | Per cent passing (by weight) | Diameter millimetres | Per cent passing (by weight) |
| 0·84 | 100 | 0·034 | 98 |
| 0·42 | 96 | 0·024 | 95 |
| 0·25 | 55 | 0·015 | 87 |
| 0·105 | 2 | 0·0093 | 66 |
| 0·074 | 1 | 0·0065 | 57 |
| — | — | 0·0048 | 48 |
| — | — | 0·0021 | 34 |
| — | — | 0·0011 | 23 |

*Grain Size Distribution, Till*

| Diameter millimetres | Per cent passing (by weight) |
|---|---|
| 38·1 | 100 |
| 18·8 | 92 |
| 9·4 | 89 |
| 4·7 | 83 |
| 2·00 | 82 |
| 0·84 | 78 |
| 0·25 | 68 |
| 0·105 | 54 |
| 0·05 | 41 |
| 0·01 | 21 |
| 0·005 | 16 |
| 0·001 | 8 |

Shear strength of till—remoulded at standard Proctor density; effective angle of internal friction, 39 deg.; effective cohesion 400 pound per square foot.

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 intact, even in the near wall which was only 25ft 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 versus 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 the correlation is not quite as good as the correlation between damage and velocity or acceleration. 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 $E^{2/3}/d=0\cdot 1$ 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 30ft, can readily be examined. This range has already been considered by Langefors *et al*, and their recommended safe limit, which they extend down to a distance of 3ft, 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 4in to 5in per second. A safe limit of 2in 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, for example, 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 $E^{2/3}/d=0\cdot 3$ (where $E$ is weight of explosive in pounds, and $d$ is distance in feet). Allowing a factor of safety of three the value of $E^{2/3}/d=0\cdot 1$ is recommended as 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 safety factor of perhaps 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 an array of ¼in diameter pins varying in length from 6in to about 18in 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.

## Acknowledgments

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 the Research Division and the 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 co-operation of Mr. Gordon Mitchell, Director of the St. Lawrence Power Project, and his staff..

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

References

[1] Thoenen, J. R., and Windes, S. L. "Seismic Effects of Quarry Blasting," *U.S. Bureau of Mines Bulletin* 442.
[2] Crandell, F. J., "Ground Vibrations Due to Blasting and Its Effect Upon Structures," *Journal, Boston Soc. Civil Engineers*, April 1949, pages 222-245.
[3] Morris, G., "Vibrations Due to Blasting and Their Effects on Building Structures," THE ENGINEER, 190, page 394 and page 414 (October 27, November 3, 1950).
[4] Langefors, U., Westerberg, H., and Kihlstrom, B., "Ground Vibrations in Blasting," *Water Power*, September (pages 335-338), October (pages 390-95), November (pages 421,424), 1958.
[5] Rockwell, E. H., "Vibrations Caused by Blasting and Their Effect on Structures," Publ. Hercules Powder Company, Wilmington, Delaware, 1934.
[6] Windes, S. L., "Damage from Air Blast," U.S. Bureau o Mines, Report R.I. 3622, February, 1942.
[7] Morris, G., and Westwater, R., "Damage to Structures by Ground Vibrations Due to Blasting," *Mine and Quarry Engineering*, April 1953 (pages 116-118).