United Kingdom Atomic Energy Authority

AWRE, Aldermaston

AWRE REPORT NO. 032/70

# Vibrations in Structures

F.H. Grover

C1

534.15
550.343
534:624.012
624.012:534

## TABLE OF CONTENTS

TABLE OF CONTENTS (CONT.)

## SUMMARY

This report describes three examples of small scale operations undertaken by the Seismic Measurements Group, UKAEA Blacknest.

Versions of equipment and data processing methods developed for the main seismic detection programme of SSD have been used to study the effects in structures close to explosions and other sources of strong vibrations. In particular, a caravan unit originally designed as a mobile seismic array station, has been especially useful as a field laboratory for work of this kind and was used for the operations in Parts 2 and 3.

## PART 1: SEISMIC MEASUREMENTS AT DARESBURY NUCLEAR PHYSICS LABORATORY

## 1. INTRODUCTION

A proposal to increase the length of the electron acceleration path for the synchrotron apparatus at Daresbury involved a tunnelling operation into an adjacent hillside using explosives. It was thought that blasted operations might cause vibrations of sufficient amplitude to upset the alignment of the synchrotron magnets which had been horizontally and vertically adjusted to an accuracy of 0.005 in.

As trial boreholes had to be drilled along the proposed electron path extension in order to establish the nature of the terrain, two of these were to be used for test firings from which the effects of explosive charges in the weight range recommended by the contractors could be determined.

The Seismic Measurements Group were asked to provide an instrument installation suitable for recording these effects and also for monitoring over a period, samples of the seismic background noise produced by regular activities in the Synchrotron-Electron Hall complex. A simple seismic velocity profile of the upper ground layer of the site was also required.

This work was carried out by the Seismic Measurements Group during the summer of 1968 under a contract arranged by UKAEA Works Group, Risley.

## 2. ARRANGEMENT OF INSTRUMENTS

### 2.1 Vibration measurements

Two vertical component Willmore Mk. II seismometers were installed in positions selected by DNPL Engineers. These positions were: (1) the floor of the main electron hall, and (2) the synchrotron magnet mounting.

The seismometer outputs were amplified by local amplifier units and connected by cables to a twin jet ink recorder with calibration facilities. Figure 1 shows the main features of the DNPL site and the positions of the seismometers and boreholes.

### 2.2 Profile measurements

These measurements were made on a flat area of the site covering part of the proposed new electron path. Refraction lines denoted by datum points G1-G9 in figure 2 were surveyed in turn using Dynametric Model 117 profiling equipment comprising striker, geophones (seismic receivers) and timer. Forward and reverse profiles were obtained along each line using datum points at both ends for geophone positions.

Sandstone samples were obtained from the near borehole at 40 ft depth so that the specific gravity of material in the layers surveyed by the profile could be determined.

## 3. RESULTS

Figures 3, 4 and 5 show records of the principal seismic noise measurements. Details of these are summarised in table 1.

## TABLE 1

### Noise Measurements

| Seismometer Position | Cause of Vibration | Dominant Frequency, Hz | Maximum Particle Velocity, cm/s Peak/Peak | Maximum Ground Displacement, cm Peak/Peak |
|---|---|---|---|---|
| Electron Hall Floor | Normal seismic background noise | 4 | $5 \times 10^{-4}$ | $0.2 \times 10^{-4}$ |
| Electron Hall Floor | Overhead crane moving with load | 13 | $49 \times 10^{-4}$ | $0.6 \times 10^{-4}$ |
| Electron Hall Floor | Overhead crane stationary, lowering 17 ton concrete block | 2.7 | $51 \times 10^{-4}$ | $3 \times 10^{-4}$ |
| Synchrotron Magnet | Vacuum pump | 4 | $76 \times 10^{-4}$ | $3 \times 10^{-4}$ |

Figures 6 and 7 show records obtained from the shot firings and table 2 summarises the main features of these records.

### 3.1 Seismic profile measurements

From these measurements a table of travel times corresponding to several distances along each of the lines 1 - 4 was obtained.

The results for line 1 which extended 160 ft in a NW/SE direction are plotted in figure 8.

Lines 2, 3 and 4 running NE/SW were profiled in 80 ft half lengths. Line 3 results, typical of three lines, are plotted in figure 9.

Each of the graphs has a fairly well defined "knee" above which the slope trends to a new steady value. This change in slope indicates the presence of a higher velocity layer of more compact material lying near to the surface. Values for the seismic velocity in the sub-layer and for the depth of the latter below surface can be derived from the plotted data.

The specific gravity of sandstone samples from borehole No. 24 was saturated, 2.19 and dry, 1.95.

## TABLE 2

### Shot Record Measurements

| Seismometer Position | Shot No. | Record Component Measured | Dominant Frequency, Hz | Maximum Particle Velocity, cm/s Peak/Peak | Maximum Ground Displacement, cm Peak/Peak |
|---|---|---|---|---|---|
| Electron Hall Floor | 1 | Largest cycle in high frequency train | 33 | $47 \times 10^{-4}$ | $0.23 \times 10^{-4}$ |
| Electron Hall Floor | 1 | Largest cycle of dominant low frequency | 7 | $94 \times 10^{-4}$ | $2.1 \times 10^{-4}$ |
| Synchrotron Magnet | 1 | Largest cycle at high frequency | 33 | $49 \times 10^{-4}$ | 0.24 |
| Synchrotron Magnet | 1 | Largest cycle at low frequency | 7 | $103 \times 10^{-4}$ | $2.3 \times 10^{-4}$ |
| Electron Hall Floor | 2 | Largest cycle at high frequency | 25 | $47 \times 10^{-4}$ | $0.3 \times 10^{-4}$ |
| Electron Hall Floor | 2 | Largest cycle at low frequency | 8 | $70 \times 10^{-4}$ | $1.4 \times 10^{-4}$ |
| Synchrotron Magnet | 2 | Largest cycle at high frequency | 25 | $63 \times 10^{-4}$ | $0.4 \times 10^{-4}$ |
| Synchrotron Magnet | 2 | Largest cycle at low frequency | 8.7 | $104 \times 10^{-4}$ | $1.9 \times 10^{-4}$ |

### Shot Details

Shot 1: 50 lb, depth 65 ft, distance to Electron Hall 1300 ft (Borehole 21).

Shot 2: 15 lb, depth 40 ft, distance to Electron Hall 650 ft (Borehole 24).

## 4. CALCULATIONS

The depth of sub-surface layers can be calculated from the expression

$$d = \frac{Lx}{2} \sqrt{\frac{V2 - V1}{V2 + V1}},$$

where d is the average depth to the layer under the measurement path. In this expression Lx is the distance in feet measured along the x axis to the point where the slope of the travel time graph changes from its initial value to a new, relatively steady, value representing change in seismic velocity from V1 to V2.

The velocities V1 and V2 can be obtained in feet per second by direct measurement from the graphs. The values of Lx are estimated by assuming an abrupt change of slope at a point where the projected initial and ultimate slopes intersect.

In figure 8, from the results with geophone at position G1, line 1,

$$Lx = 12 \text{ ft},$$

$$V1 = 666 \text{ f/s},$$

and $$V2 = 2704 \text{ ft/s},$$

$$d = \frac{12}{2}\sqrt{\frac{2038}{3370}}$$

$$= \underline{4.7 \text{ ft.}}$$

With geophone at position G2, line 1,

$$Lx = 24 \text{ ft},$$

$$V1 = 832 \text{ ft/s},$$

and $$V2 = 4000 \text{ ft/s},$$

$$d = \frac{24}{2}\sqrt{\frac{3168}{4832}}$$

$$= \underline{9.7 \text{ ft.}}$$

In figure 9 from results with geophone at position G5, line 3,

$$Lx = 12.4 \text{ ft},$$

$$V1 = 631 \text{ ft/s},$$

$$V2 = 2908 \text{ ft/s},$$

$$d = \frac{12.4}{2}\sqrt{\frac{2277}{3539}}$$

$$= \underline{5 \text{ ft.}}$$

and with geophone at position G7, line 3,

$$Lx = 19.4 \text{ ft},$$

$$V1 = 800 \text{ ft/s},$$

$$V2 = 3740 \text{ ft/s},$$

$$d = \frac{19.4}{2}\sqrt{\frac{2940}{4540}}$$

$$= \underline{7.8 \text{ ft.}}$$

4.1 Modulus of elasticity

$$E = \mu \frac{(3\lambda + 2\mu)}{(\lambda + \mu)}, \qquad .....(1)$$

$$C^2 = \frac{(\lambda + 2\mu)}{\rho}, \qquad .....(2)$$

$$\sigma = \frac{\lambda}{2(\lambda + \mu)}, \qquad .....(3)$$

where E is Young's modulus,
$\lambda$ is Lamé's constant,
$\mu$ is the modulus of rigidity,
$\rho$ is the density of the material,
$\sigma$ is Poisson's ratio,
c is the velocity of compressional waves in the material.

For many common rocks $\sigma \simeq 0.25$, therefore from (3) $\lambda = \mu$.

With this assumption (1) and (2) can be simplified giving $E = \frac{5}{6}\rho c^2$

For saturated sandstone of specific gravity 2.19, density $\rho$ in lb/in./s units

$$= 2.19\ (9.3 \times 10^{-5} \text{ lb s}^2 \text{ in.}^{-4}),$$

where $9.3 \times 10^{-5}$ lb s$^2$ in.$^{-4}$ is the density of water.

From the travel time graphs, the average maximum value of C is 3800 ft/s = $3.8 \times 12 \times 10^3$ in./s,

therefore $E = \frac{5}{6}(2.19 \times 9.3 \times 10^{-5}) \text{ lb s}^2 \text{ in.}^{-4} (3.8 \times 12 \times 10^3)^2 \text{ in.}^2 \text{ s}^{-2}$

$$= \underline{0.35 \times 10^6 \text{ lb in.}^{-2}}$$

5. DISCUSSION OF RESULTS

The test shots produced vibrations with maximum amplitudes about an order above the normal background noise level on the site.

However, these amplitudes were somewhat less than the vibration levels of 3 microns peak-to-peak produced intermittently by machinery already installed permanently in the synchrotron magnet area. The larger vibrations were in the frequency band 2 - 9 Hz.

The observed vibration levels were far below those having any structural significance, but could possibly affect optical measurement systems associated with the synchrotron if the optical components have resonant frequencies in the main vibration band.

The profile measurements give an indication of the depth below surface of the sandstone layer under the site. Reference to figure 2 and the calculated values of depth at points $G_1$, $G_2$, $G_5$ and $G_7$ suggest that there is between 5 and 10 ft of less consolidated material overlaying the sandstone, with the greater depth in towards the site area, away from the hill.

At the depths measured the seismic velocity in the sandstone has a rather low value for this rock type. The calculated value of Young's modulus has been compared with data given by Brown and Robertshaw [1] and are plotted for comparison with these data in figure 10.

## 6. CONCLUSIONS

On the basis of the results, unduly large vibrations would not be expected from the maximum instantaneous charges of 15 lb suggested by ICI for future blasting operations.

The thickness of the uppermost low velocity layer on the site is not more than 10 ft.

## PART 2: INVESTIGATION OF STRUCTURAL FAILURES IN A DOCKSIDE BUILDING

### 7. INTRODUCTION

The building which was the subject of this investigation is situated next to a lock connecting the Humber Estuary with the Immingham dock inner basin (figure 11). It serves as a Mission to Seamen and is built as a two-storey residential block with conjoining single-storey recreational areas (figure 12).

The purpose of this investigation was to determine whether vibrational or some special factors in the environment could account for severe cracking of the walls in the single storey area of the building. In this part of the building the external walls have moved in an outward direction above the damp proof course level so that they overhang the foundation course by $\frac{1}{2}$ to 1 in.

This kind of structural failure is not dissimilar to that seen in buildings subjected to horizontal shear stresses by earthquakes. (figures 13 and 14). No satisfactory explanation had been found in earlier investigations by architects seeking conventional causes of structural failure.

### 8. ARRANGEMENT OF INSTRUMENTS

In an initial series of measurements, groups of seismometers were placed at various points on the ground floor of the building and on the ground outside (figure 15). With this arrangement, horizontal and vertical component vibrations were recorded for a period of some days during which all shipping and vehicle movements and other activities in the vicinity of the building were logged. A caravan equipped as a field laboratory with multi-channel taper and chart recording facilities was used in this first stage of measurements (figure 12).

At a later date horizontal and vertical component vibrations at a single point in the building were recorded continuously for several weeks by a very slow speed, unattended tape recording system. A group of 3 strain gauges was mounted on an external wall at a point where the overhang above the damp course was greatest. Two of the gauges bridged the damp course itself so that they responded to any relative movement across it and the third gauge was mounted on the upper wall brickwork to serve as a control for temperature effects etc (figure 16). The outputs of these gauges and the output of a sensitive electronic level mounted on the building floor were also recorded on the tape recorder together with marker pulses from a simple timing unit.

A short series of comparative measurements were also made at a higher tape recording speed with seismometers situated on the roof of the building, in a window recess of the south west wall and on the ground outside.

## 9. RESULTS

In the first series of measurements, vibration patterns from vessels ranging in size from tugboats to a cargo ship of 30000 tons were recorded. Vibration patterns for vehicles, train shunting and locking operations was also observed.

For ships it was found that vibrations were set up by the use of engines when manoevring in the locks. The resulting bursts of vibration sometimes lasted for periods of minutes duration during which time the amplitude remained fairly steady and the frequency assumed some value which was characteristic of the particular vessel. The levels of vibration were similar at all the measurement points on the building floor and outside ground and the vertical component motion was generally rather greater than the horizontal motion. A typical result is illustrated in figure 17.

For vehicles, vibration amplitudes produced on the building floor were comparable with those for ships. Large vertical component motion was always produced, the horizontal component amplitude was very variable, but sometimes equally large. The duration of vibration bursts from vehicles were shorter than those from shipping, corresponding to the time required for a vehicle to pass the building. On the other hand, the rate of occurrence of these bursts was very much greater, traffic was very heavy at times with loaded trucks passing at frequent intervals. This resulted in a semi-continuous spectrum of vibrational noise during part of each day (figure 18).

Vibrations from most other activities such as shunting on nearby railway lines and crane movements on the opposite wharf were relatively insignificant with the exception of the operation of the sluice gates in the lock. The effect of closing these gates was to produce a damped wave train of around 1 Hz. Slight impacts from ships on the wall of the lock were found to produce a similar result on the horizontal component channels (figure 19).

The results of this stage of measurements can be summarised as follows:-

| | | |
|---|---|---|
| Ground motion | $100 \times 10^{-4}$ cm peak/peak | Average maxima for both ships and vehicles. |
| Particle velocity | $0.05$ cm $s^{-1}$ | |
| Acceleration | $1.0$ cm $s^{-2}$ | |

Ship engine vibrations fell in a frequency band between 0 - 11 Hz and produced the systematic ground motion shown by the particle orbit representations of figure 20. These were produced by applying the recorded radial and transverse horizontal components respectively to the X and Y plates of an oscilloscope. The corresponding orbits for vehicle vibrations indicate a random type of ground motion. Vehicle vibration frequencies were predominantly around 3 - 5 Hz.

## 10. DISCUSSION

Criteria for damage to buildings from vibration have been postulated in the past by various authorities, often to define limits for acceptable levels of vibration from blasting operations. Acceleration, displacement and particle velocity have each been favoured by particular writers, but it has been shown by Duvall and Fogelson [2] that a criterion based on particle velocity can apply to a variety of physical conditions. These authors quote a value for particle velocity of 5 cm $s^{-1}$ as the threshold at which minor damage may be caused in buildings. This is a level below which the probability of damage is small, but nevertheless it has been noted more recently by Wall [3] that vibrations with particle velocities of the order of 0.1 cm $s^{-1}$ can hasten the onset of what is described as "natural" cracking in the walls of residential buildings.

The superficial conclusion to be drawn from the first stage of measurement was that the vibrations to which the building was subjected were not large enough to cause the degree of damage observed. However, it was supposed that the prolonged nature of the vibrations must tend to cause the damage threshold to be low rather than high, also that amplitudes in other parts of the structure might be greater than those measured on the floor. Angenheister and Fortsch [4] state that it is virtually impossible to predict what the effects of particular levels of vibration on a structure will be, as the dynamic properties are highly dependent on shape, method of construction materials etc. Shepherd and Walpole [5] show that foundation compliance can allow coupled modes of vibration to occur when a natural period of a building corresponds to a vibration period in the foundation subsoils. In such cases of resonance flexible buildings on soft soils are penalized compared to stiffer structures.

Another, longer series of measurements seemed necessary to resolve a number of points, ie, (1) by providing a longer statistical sample to show whether occasional episodes of larger amplitude vibrations occurred (eg, from greater ship impacts with the lock walls), (2) to attempt to relate the occurrence of any new strains in the building to such episodes, and (3) to obtain some comparative measurements of vibrations at and above ground level.

Results and analysis of this second set of measurements are summarised below.

## 11. SUMMARY OF RESULTS, STAGE 2

### 11.1 Vibrations

Daily average maximum levels of vibration on the floor were similar to those previously obtained. Ships' engines commonly produced particle velocity values in the range 0.03 to 0.05 cm $s^{-1}$ for one or two minutes duration several times each day. Occasional days were exceptionally noisy with heavy ship and vehicle traffic. Rather infrequently, the average maximum level was exceeded by vibration bursts of short duration which were thought to be episodes of ship "bumping" and gave values of particle velocity of the order of 0.1 cm $s^{-1}$.

Examination of several weeks combined seismometer and strain gauge records using fast tape replays did not indicate sudden changes in strain associated with the larger bursts of vibration. On two occasions, slight changes in strain were observed at the same time as changes in tilt and strong vibrations on the vertical component seismometer. These effects were of short duration and may have been caused by an exceptional vehicular load passing close to the building. No other significant tilting effects were apparent from the records produced by the electronic level.

## 11.2 Strain variations

The results of strain gauge checks made at 2 week intervals in a continuous period of operation from 5th June to 31st July 1969 are shown in table 3. Positive values of strain indicate an outward movement of the wall above dpc level and the table shows that an apparently steady outward trend was occuring.

TABLE 3

| Date | | June 5th | June 19th | July 3rd | July 17th | July 31st |
|---|---|---|---|---|---|---|
| Cumulative Strain, in. × $10^{-3}$ | Gauge 1 (dpc) | 0 | +6.5 | +12.5 | +18.5 | +21 |
| " | Gauge 1 (dpc) | 0 | +6.0 | +12.5 | +18.5 | +25 |
| " | Gauge 3 Control | 0 | -0.5 | 0 | +1.0 | -2.5 |

A detailed examination of the strain gauge tape records was made to determine how the wall movement took place in the intervals between strain measurements. This revealed a considerably more complex behaviour than was apparent from the simple spot checks on cumulative strain. Diurnal swings of double the amplitude of the ten day cumulative strain level ($Y_1 - Y_2$) can be seen in the section from a replayed strain record shown in figure 21. The output of this gauge was recorded on a high gain tape channel.

## 11.3 Meteorological effects

A study was made of meterological data from the nearest Meteorological Station, Kilnsea, on the north bank of the Humber Estuary and, showed that the peaks of strain occurred when hours of sunshine and ambient temperature were highest.

To enable a qualitative estimation of the apparent relationship of strain variations and warm weather conditions to be made, an electrical analogue model approximating to the total heat received by the building was produced.

This analogue was derived by summing two voltage waveforms (figures 22(a) and (b)) which represented the heat assumed to have been received by the building from solar radiation and the ambient environment. These waveforms were produced by integrating circuits into which voltage input steps based on the Daily Meteorological Data for sunshine hours and ambient temperature were fed at scaled time intervals of 24 hours.

Some arbitrary assumptions were made about rates of rise and decay of temperature in the brickwork in order to arrive at suitable scaled time constants for the integrators. If the movements of the walls are further assumed to be proportional to the total heat flux, then the summed waveform (figure 22(c)) can also represent wall movements and these are seen from the figure to be reversible over the ten day period (as shown by broken line $X_1 - X_2$).

To this waveform was added one other function representing an extra, irreversible, component of strain. This was chosen for simplicity, to be a voltage ramp (broken line, figure 22(d)) which rose to a final value equivalent to about half the voltage of the larger diurnal swings. The result of summing these three simple analogues can be seen in figure 22(d), which compares the simulated and actual strain records.

## 11.4 Noise power spectra

A number of the tape recorded noise samples were Fourier analysed after digitisation, using a computer program due to A. Douglas, Blacknest. Noise power spectra produced in this way provided comparisons between ground and building vibrations.

The spectrum shown in figure 23 is typical of results obtained from a horizontal seismometer oriented transversely to the south west wall on the ground outside the building.

Figure 24 is from a transversely aligned horizontal seismometer on the roof, time related to the ground sample. It shows a similar spectrum but with an enhanced high frequency content.

Figure 25 is from a vertical seismometer mounted on the ledge of the window aperture on the South West wall, inside the building.

Figure 26 is the time related spectrum from a horizontal seismometer mounted on the ledge next to the vertical instrument. This result indicated that the wall had resonant modes at 5.1 and 13.6 Hz. Examination of the analogue replay of this noise sample showed that particle velocity frequently reached 0.05 cm $s^{-1}$ in vibration bursts, although the site background noise at this time was low.

## 12. DISCUSSION OF STAGE 2 RESULTS

Beard et al. [6] have recently published results of six years' measurement of unrestrained experimental walls and also describe a case history of movements in the walls of a single storey factory building. These authors conclude that brick walls can expand when there is insufficient restraint and that the rate of expansion soon falls from an initially higher rate to a rate which remains steady for years.

Much of the movement is the result of thermal cycling. On hot summer days the maximum temperature of all walls of a building can exceed maximum shade temperature because even north facing walls receive indirect solar radiation. Walls receiving direct radiation can reach mean temperatures of over 100°F.

Thermal movement is substantially reversible but, in the case of an unrestrained wall laid on a bituminous clamp proof course of low shear resistance, a sliding movement may occur which is irreversible. This is because numerous fine cracks develop in the wall due to tensile stresses. Such movement occurs only in the brickwork above the dpc since the lower brickwork is restrained by its bond to the foundation.

In the six year measurements, expansion rates of 0.005% per annum were obtained from north facing leaves of free standing cavity walls and 0.01% per annum from south facing leaves. Slightly lower rates of expansion occurred in long continuous walls of the single storey factory building.

Whilst extrapolation of these statistics to other structures should be made with some caution, it does seem valid to compare them with the measurements obtained at Immingham in view of several point of similarity between the Mission building and the cases described by Beard.

The age of the various structures are similar and the sliding evident in all external single storey walls of the Immingham building is greatest for walls with southerly aspects. The absence of sliding in the 2 storey block can be explained by the shear strength of the dpc being increased above some critical value by the greater weight of the structure above it.

Applying the higher rate of 0.01% to the Immingham case, an expansion of 0.54 in. for the 90 ft longest side of the single storey part would be expected if the wall were one continuous length without abutments. The actual expansion is 0.8 in. and there are walls abutting at two points along its length. For the 40 ft south west wall a length expansion of 0.25 in. would be expected, whereas the actual overhang is 0.625 in.

At the most westerly end, which has only very short adjoining walls around a corner with somewhat complex contours, approximately only 0.1 in. movement would be expected, but in fact the whole south west wall is thrust out by over 1 in. at this point. On the north side of the building, short walls surrounding an enclosed yard have moved sufficiently to deform the wooden framework of the entrance. The expected expansion of these walls would be negligible.

Clearly, these large movements are difficult to explain in terms of the experimental wall statistical data alone. It might be expected that the more integrated structure of an actual building would resist expansion to some degree and produce smaller movements than those in free standing walls, yet factors of between 1.5 to 10 times greater are seen to exist in this building.

Similarly, if the vibration measurements are considered in isolation, the maximum level observed appear to be at, or just below, levels which might intensify natural cracking, but not of an order which would cause significant movements.

A combination of thermal cycling, vibration effects and a timing factor seems to offer the true explanation. Thermal heating of the walls provides the large motive force tending to move them outwards, vibration acts as a lubricant which, by overcoming static friction at the split plane of the dpc, enables them to move freely at that point. The thermal expansion taking place during a hot summer day might be expected to be wholly reversible during the cooler night as suggested by the graph in figure 22(c). However, because of the time pattern of activities in the dock, vibration is greatest at the times of greatest expansion in the afternoons and least at the time of contraction at night. This differential straining of the walls involving the diffusion of fine cracking throughout the structure has caused the unidirectional creep outwards shown qualitatively by the dotted line in figure 22(d) and by the chronological observations illustrated in figure 27.

## 13. CONCLUSIONS

(1) Irreversible sliding movements can take place in lightly loaded walls with a horizontal bituminous damp proof course due to the effects of thermal cycling.

(2) The existence of semi-continuous vibrational energy in the environment can, by reducing the static friction bond at the dpc, increase these movements to unacceptable proportions.

(3) The particle velocity of damaging ground vibration in this particular context may be more than an order below levels usually considered to be the threshold of damage to buildings.

(4) Vibration amplitudes may be greatly increased by wall resonances. Resonant modes have been found at frequencies commonly occurring in the vibration spectrum produced by shipping and vehicles. (Energy may be received from both seismic and acoustic propagation, but the fractions from each have not been separated in these measurements.)

(5) With the general increase in volume of industrial traffic, situations of this kind could possibly recur. The damage in this building could have been minimised by a stiffer form of wall construction, the use of stanchions for reinforcement and damp proof course material of much higher shear strength.

## PART 3: THE NEAR-IN EFFECTS OF A DEMOLITION BY EXPLOSIVES

### 14. INTRODUCTION

Because of the deterioration in the structural condition of 4 cooling towers at Ince Power Station, Cheshire, the Central Electricity Generating Board has begun a phased programme for demolition of these structures.

The programme was designed to allow the station to remain operational during the work and involved the initial demolition of the worst damaged, outermost, tower and construction of a single large tower of new design to replace all four towers. It was proposed that as much information as possible should be obtained about the tower collapse and the nearby effects because of the proximity to the power station of other towers scheduled for demolition. Also, as only a few comparable demolitions had been made, there was a scarcity of data in this field and it was desired to increase the knowledge of the subject for application to further operations of the kind likely to be needed in the near future.

### 15. INSTRUMENT ARRANGEMENT

A plan of the cooling tower site area is shown in figure 28. By arrangement with CEGB the instrumentation of the site was to be shared between J.H. Crockett and Associate and UKAEA Blacknest. The former were to make a number of vibrograph recordings, acoustic tape recordings and arrange cine film coverage, UKAEA were to provide additional vibration measurements and an atmospheric overpressure measurement.

The UKAEA recording van was situated near to No. 2 cooling tower and cables were run to groups of Willmore seismometers in various positions and to 2 diaphragm strain, blast gauges mounted in an open sector adjacent to No. 3 tower which was to be demolished.

The seismometers were sited as shown in figure 28. These positions were as follows:-

(1) Horizontal seismometer on ground near tower 4 base, 105 ft from tower 3.

(2) Vertical seismometer on ground near tower 4 base, 105 ft from tower 3.

(3) Horizontal seismometer on pond wall, tower 4 base, 110 ft from tower 3.

(4) Horizontal seismometer on ground far side tower 4, 330 ft from tower 3.

(5) Horizontal seismometer on pond wall, tower 2 base, 110 ft from tower 3.

(6) Horizontal seismometer on upper rim tower 2, 250 ft above ground level, vertically above installation position 5.

(7) Horizontal seismometer on ground beyond tower 2, 600 ft from tower 3.

(8) Vertical seismometer on ground beyond tower 2, 600 ft from tower 3.

All horizontal seismometers were aligned with the instrument axis in the direction of the explosion.

## 16. SHOT DETAILS

The demolition technique employed was to emplace 130 lb of explosives in 1390 shot holes distributed in the concrete legs which supported the cooling tower shell. To ensure that the tower would fall away from the adjacent towers, shot holes were not drilled in the legs in a sector of the base perimeter facing towers 1 and 2.

## 17. DISCUSSION AND SUMMARY OF RESULTS

The equipment in the mobile unit is designed basically to record weak signals from distant seismic sources. Because of the high sensitivity of the Willmore seismometers and the associated recording system it was therefore necessary to introduce very large amounts of attenuation in each channel to limit outputs to levels suitable for the tape recorder.

In arriving at the attenuation factors required, consideration was given to the reduction of seismic amplitude which might result from exploding a given charge weight distributed in numerous small holes in a structure above ground level as compared to instantaneous detonation of the same total weight in a single hole in the ground. When the records were replayed after the demolition it was found that all seismic channels had been overloaded by the signal actually received. From this it was concluded that the spatial distribution of charge had in fact provided a very efficient generator of seismic waves rather than the reverse, producing amplitudes not less than those given by Teichman and Westwater [7] for a single fully-tamped charge of equivalent weight.

From the results, it was noted that the dominant frequencies are of the order of 6 Hz. As the explosive charge was distributed around an area of approximately 80 m, the source was thus extended to dimensions comparable to wavelengths of components in the recorded signals.

Table 4 summarises the results of all the seismic recordings and figure 29 shows the output of seismometers 1 - 6. Because of the overloading of the system it is only possible to give estimates of the ground displacement and particle velocity. The estimates given apply to the probable average maximum amplitudes in the wave train. Individual pulses in any of the seismometer outputs may have exceeded these estimates by an unknown factor.

The seismometers mounted on the pond walls of towers 2 and 4 both recorded a vibration component of 22 Hz and 2 - 3 s duration.

The horizontal motion recorded on tower 4 pond wall was lower in frequency producing a factor of 2 greater in estimated amplitude than that for tower 2 . It may be valid to extrapolate this factor for movement at the tops of towers 2 and 4 where only one of the towers (No. 2) was instrumented.

TABLE 4

| Seismometer Position | Estimated Seismometer Output, V | Corresponding Peak/Peak Particle-Velocity, cm $s^{-1}$ | Principal Frequency Components, Hz | Corresponding Estimated Peak/Peak Ground Motion, cm $\times 10^{-3}$ | Duration of Wave Train, s |
|---|---|---|---|---|---|
| 1 | 1.25 | 0.37 | 3.2, 8, 22 | 18, 7, 3 | 6 |
| 2 | 2.5 | 0.74 | 7.8 | 15 | 7 |
| 3 | 2.5 | 0.74 | 3.6, 22 | 33, 5 | 16 |
| 4 | 2.0 | 0.59 | 4.8 | 20 | 7 |
| 5 | 1.5 | 0.44 | 4.8, 22 | 15, 3 | 9 |
| 6 | 3.5 | 1.0 | 5.2 | 30 | 25 |
| 7 | 0.7 | 0.2 | 6.0 | 5 | 13 |
| 8 | 0.7 | 0.2 | 5.8 | 6 | 11 |

Although the explosion record for the 250 ft high instrument on tower 2 was spoilt by overloading, good records of the very large natural vibrations prevailing were obtained prior to the explosion. Portions from these noise records were digitised and Fourier analysed using a program which gave both numerical and graphical outputs. Analysis of long and short portions of noise data gave very similar results, an example of the frequency spectrum is shown in figure 30.

From these results it becomes apparent that the explosion excited the 4th harmonic of the tower's fundamental mode of 1.3 Hz. The 2nd, 3rd and other harmonics of this mode can be seen in the spectrum analysis of the tower noise.

## 17.1 Air blast results

The strain blast gauge sited at 130 ft distance from the base of tower 3 produced an overpressure record with the classic Friedlander waveform (figure 29). The peak amplitude of the overpressure phase was 5 psi and the duration 0.65 s. The underpressure phase had a negative maximum of 1 psi and duration of 4 s.

The blast gauge at 300 ft failed to record. This may have been the result of failure of a cable connection. Many of the cables in the blast recording area were disturbed by the blast and connecting leads to the batteries supplying the gauges were hit by flying debris.

## 17.2 Photographic result

A photograph (figure 31) taken at an early stage in the collapse of tower 3 after the explosion shows that vertical cracks were spreading downwards from the upper rim of the tower. This suggests that the structural steelwork of the rim must have failed immediately after the initial blast from the explosion. The time for the tower to collapse completely was about 7.5 s and the success of the demolition can be judged by the illustration in figure 32 which shows the debris with the 130 ft blast gauge mounting in the foreground.

## 18. CONCLUSIONS

Although only estimates of the ground motion could be obtained from the records it is clear that the distributed explosive charge was a no less efficient generator of seismic waves than a point source would have been.

However, the resulting vibrations in the adjacent towers would appear from these estimates unlikely to have particle velocities in excess of a few centimeters per second. These values are below the levels given by various authors for the threshold of minor damage to structures [2,7,8].

Experience has been gained in the requirements for work of this kind. With the existing cabling and interconnections of the caravan recording unit it is necessary to include all the associated field and caravan amplifiers. Using Willmore seismometers at such close range to large explosive charges makes these items redundant and the system becomes unnecessarily complicated. A simpler system using the minimum of cable connectors and ancilliary units is to be preferred.

## ACKNOWLEDGMENTS

The Author wishes to acknowledge the cooperation received from the various establishments involved and the efforts of colleagues who were associated with these experiments.

In particular the assistance received from Mr. C.W. Jones of Civil Engineering Group, DNPL and from Rev. W. Smurthwaite, Padre in Charge at Immingham was much appreciated.

The organisation of the field operations and instrument recording systems required for all the experiments was due to Mr. E. James of Blacknest and Mr. B. Hopkins of Blacknest also collaborated in the field work of the second and third projects and was responsible for much of the subsequent data processing involved in them.

Mr. T. Whiteside and Mr. G. Harwood of AWRE Foulness kindly supplied the air blast gauges required for the third project together with calibration and other information.

## REFERENCES


1. P.D. Brown and J. Robertshaw: "In Situ Measurement of Young's Modulus for Rock by Dynamic Methods". Geotechnic, 3, 7, 283-286 (September 1953)

2. W.I. Duvall and D.E. Fogelson: "Review of Criteria for Estimating Damage of Residences from Blasting Vibrations". US Department of the Interior, Bureau of Mines Report Invest 5968, 19 (1962)

3. J.F. Wall Jr.: "Seismic Induced Architectural Damage to Masonry Structures at Mercury Nevada". Bull Seism Soc Amer, 57, 5, 991-1007 (October 1967)

4. G. Angenheister and O. Fortsch: "Destructive Effects on Buildings, Traffic Lines and other Installations as a Result of Vibrations caused by Heavy Underground Explosions". University of Munich Report Auftrag No. 62/6576, 51 (1962)

5. R. Shepherd and W.R. Walpole: "Dynamic Interaction of Structures and Soils". NZ Engineering, 21, 2, 56-62 (February 1966)

6. R. Beard, A. Dinnie and R. Richards: "Movement of Brickwork". Trans Brit Ceram Soc, 68, 2, 73-90 (March 1969)

7. G.A. Teichman and R. Westwater: "Blasting and Associated Vibrations". Engineering (12th April 1957)

8. M.E. Nadolski: "Architectural Damage to Residential Structures from Seismic Disturbances". Bull Seism Soc Amer, 59, 2, 487-502 (April 1969)

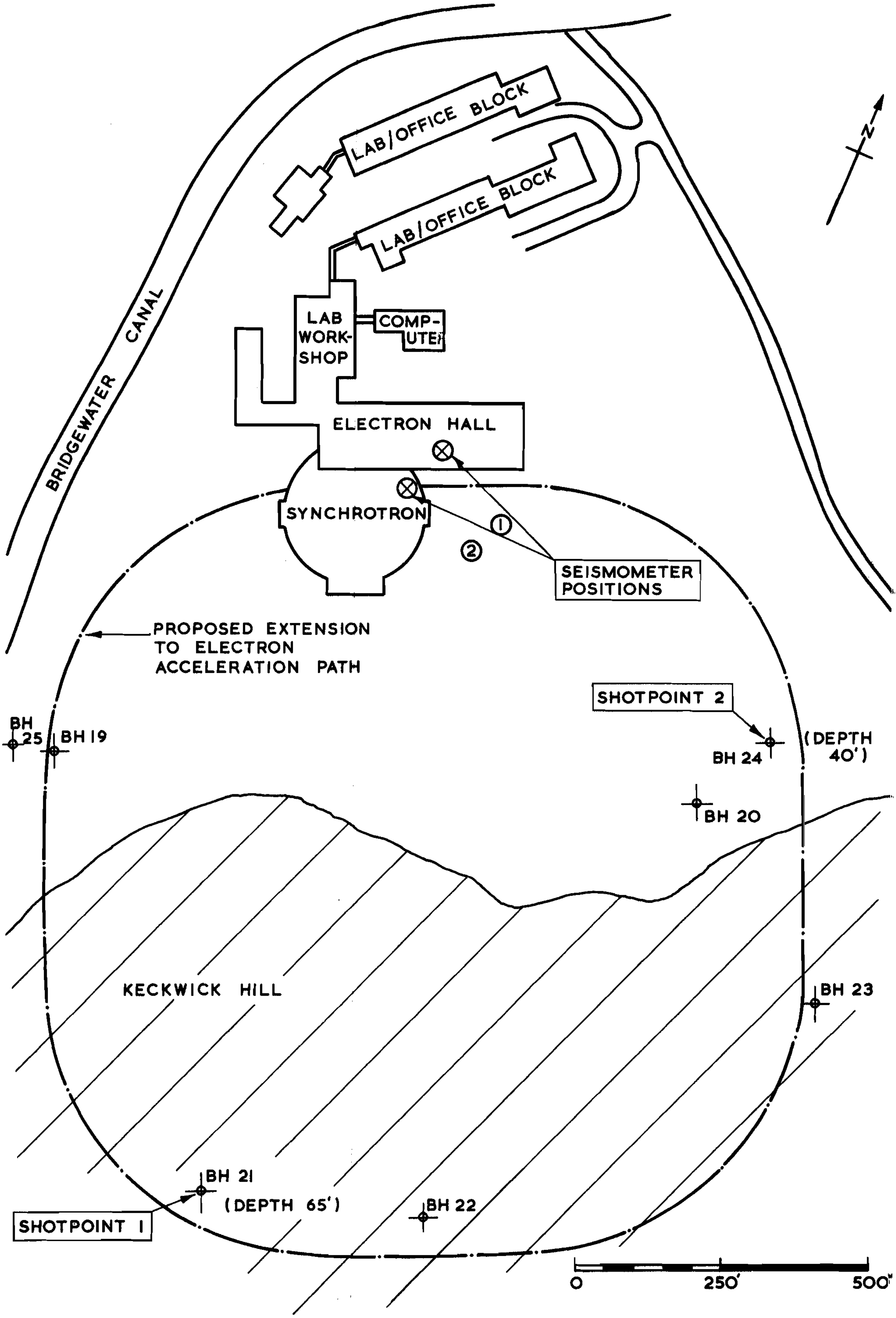



**FIGURE 1 DARESBURY NUCLEAR PHYSICS LABORATORY (DNPL) SITE – SHOWING SHOTPOINTS AND SEISMOMETER POSITIONS**

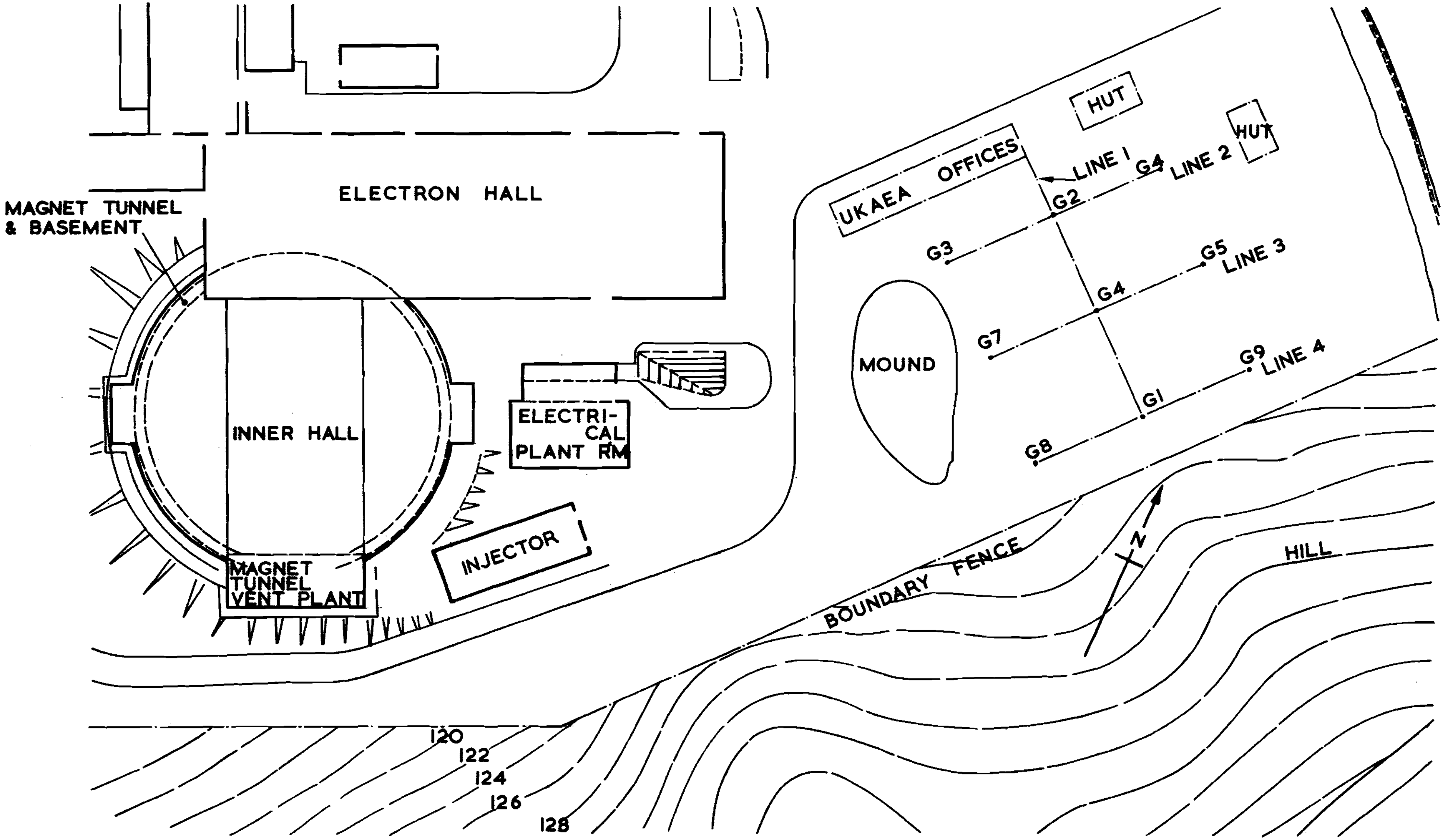



FIGURE 2. DNPL SITE SHOWING POSITIONS OF SEISMIC REFRACTION MEASUREMENTS LINES 1-4

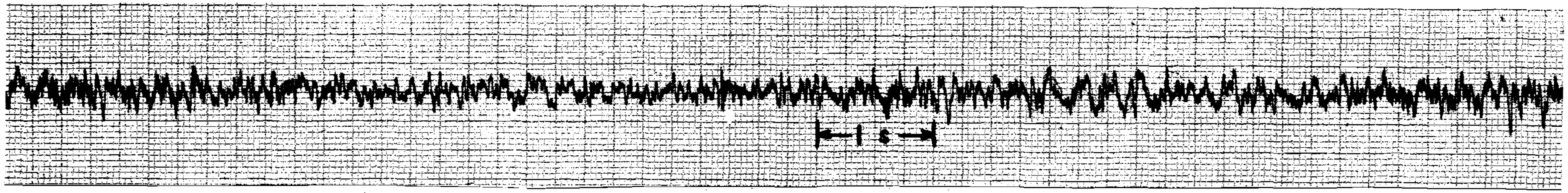



FIGURE 3. SEISMIC BACKGROUND NOISE, ELECTRON HALL FLOOR

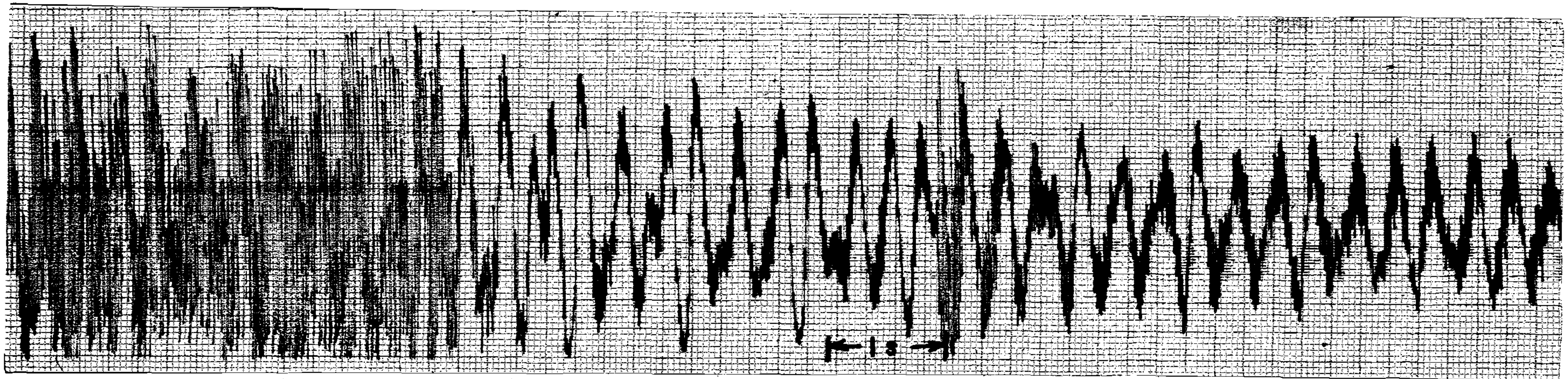



FIGURE 4. VIBRATION CAUSED BY OVERHEAD CRANE

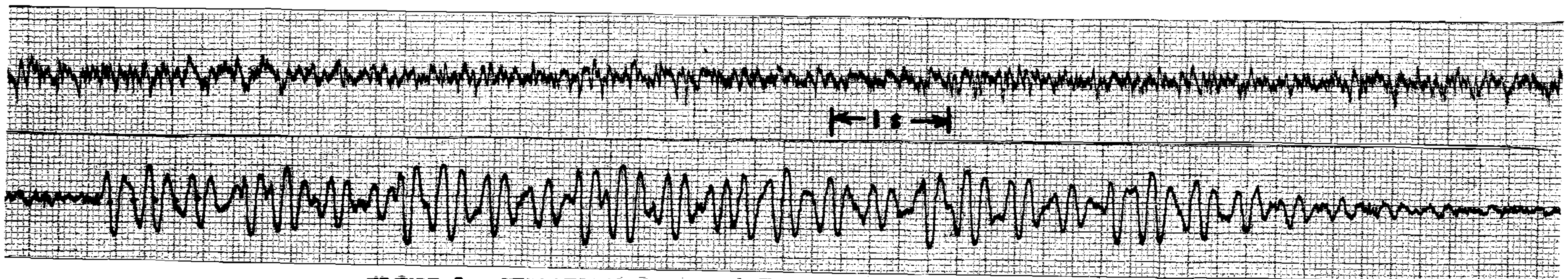



FIGURE 5. VIBRATIONS CAUSED BY VACUUM PUMP COMPARED WITH SEISMIC BACKGROUND NOISE

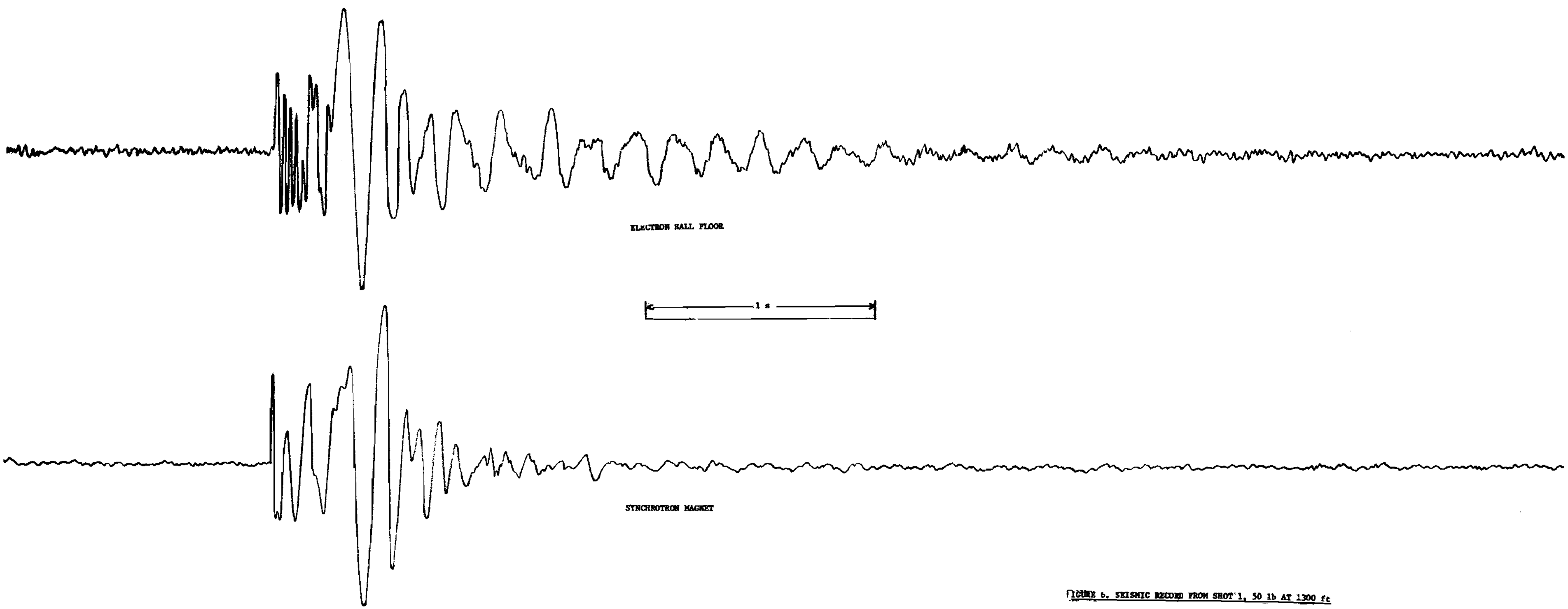



FIGURE 6. SEISMIC RECORD FROM SHOT 1, 50 lb AT 1300 ft

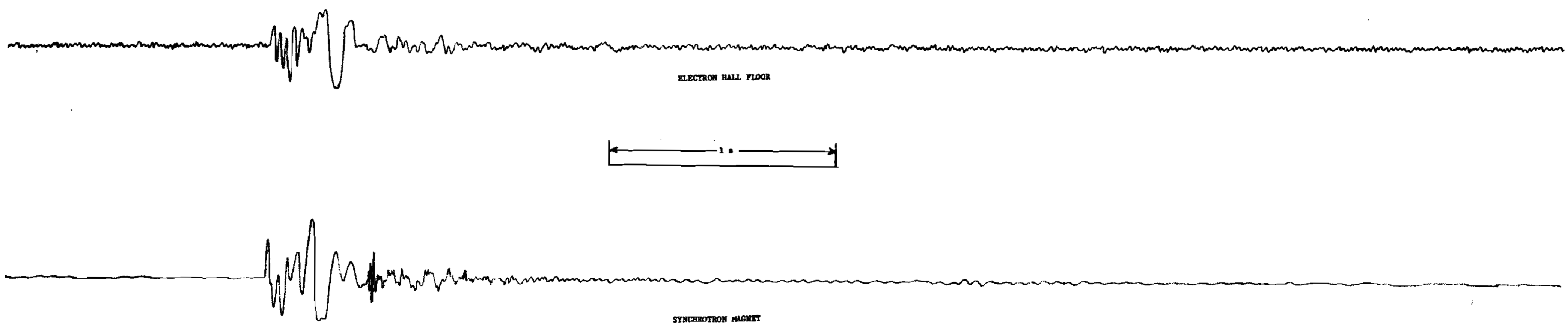



FIGURE 7. SEISMIC RECORD FROM SHOT 2, 15 lb AT 650 ft

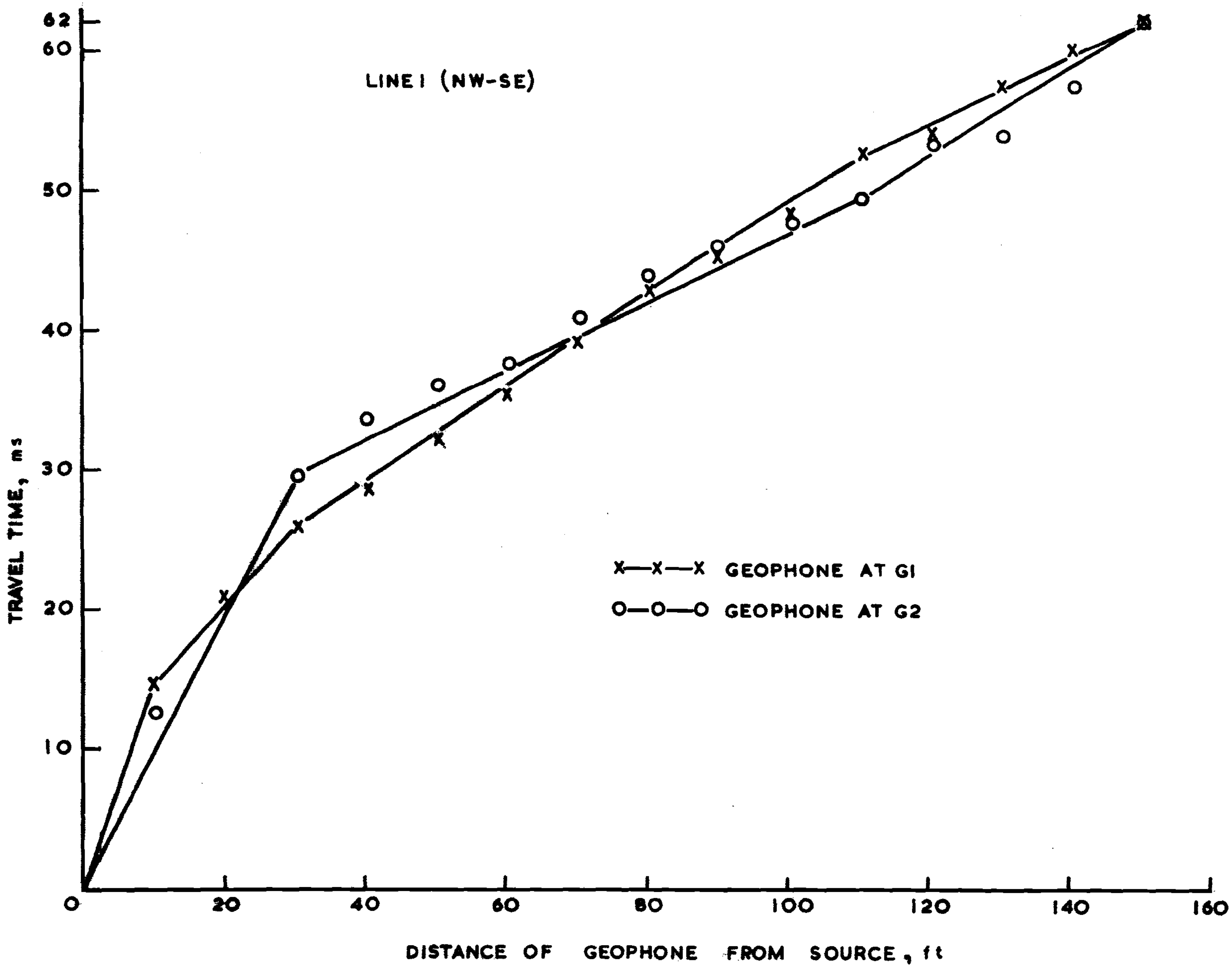



FIGURE 8. DISTANCE TRAVELLED VERSUS TIME FOR REFRACTION LINE I

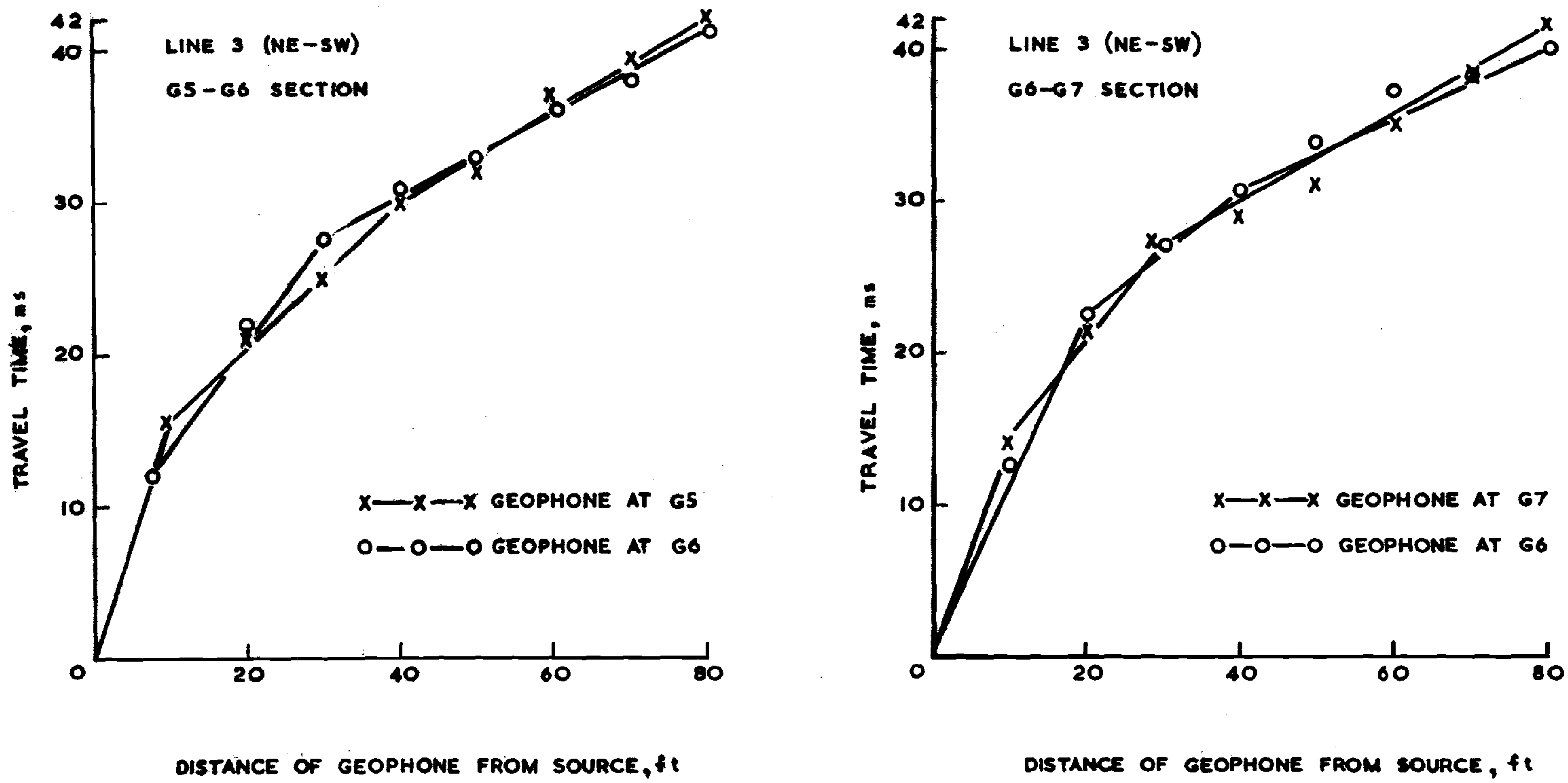



FIGURE 9 DISTANCE TRAVELLED VERSUS TIME FOR REFRACTION LINE 3

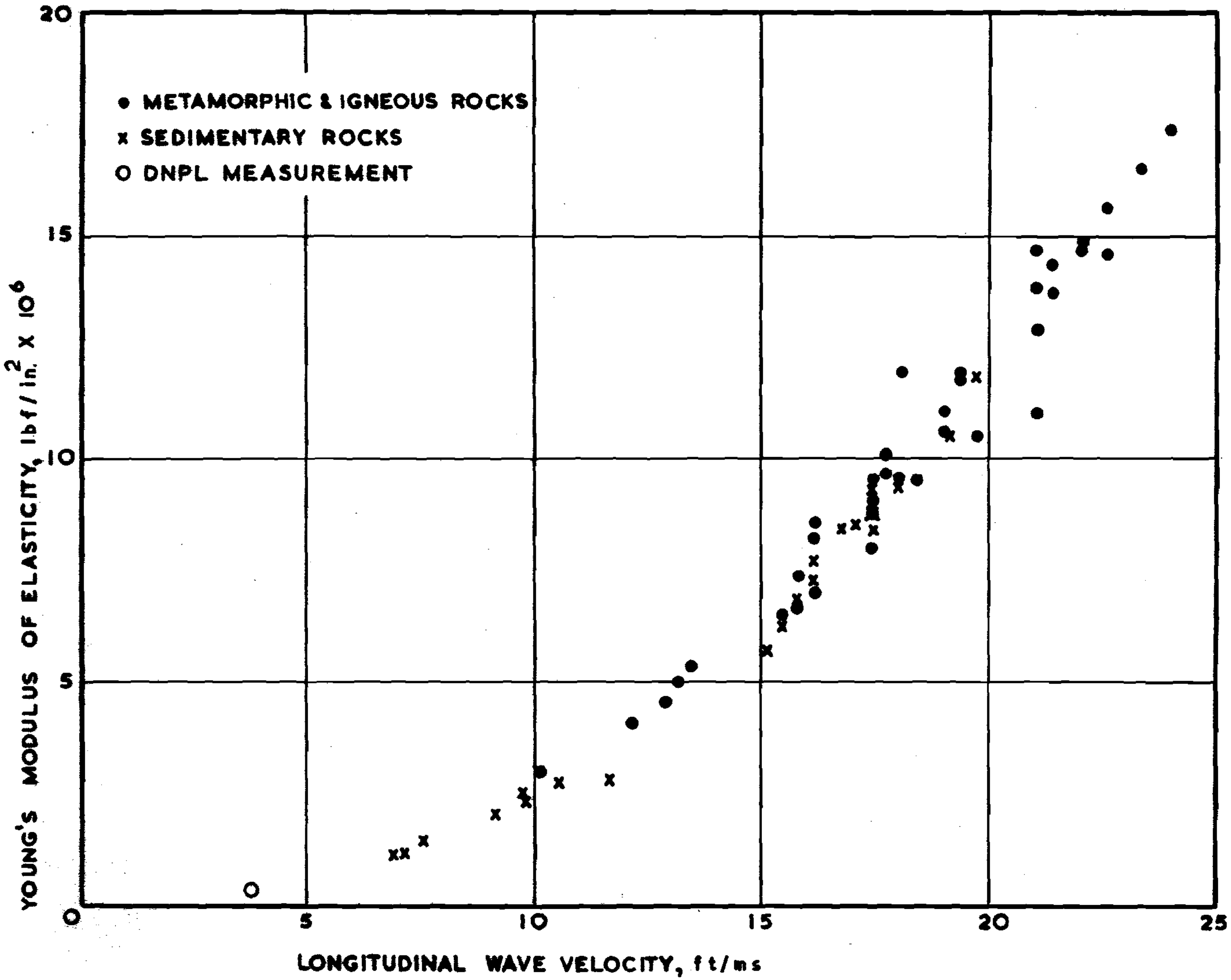



FIGURE 10. RELATION BETWEEN LONGITUDINAL WAVE VELOCITY AND YOUNG'S MODULUS OF ELASTICITY FOR ROCKS

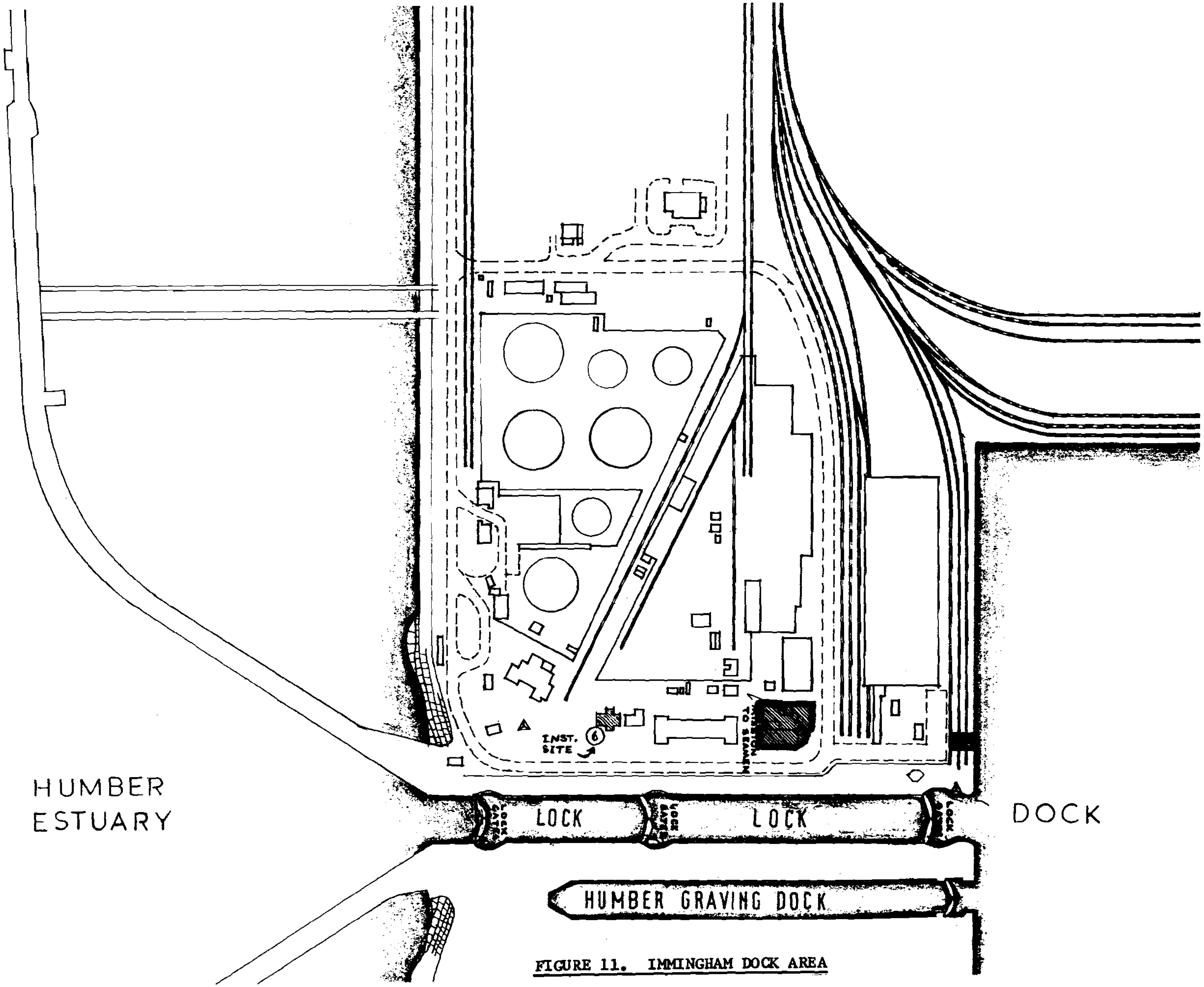



FIGURE 11. IMMINGHAM DOCK AREA

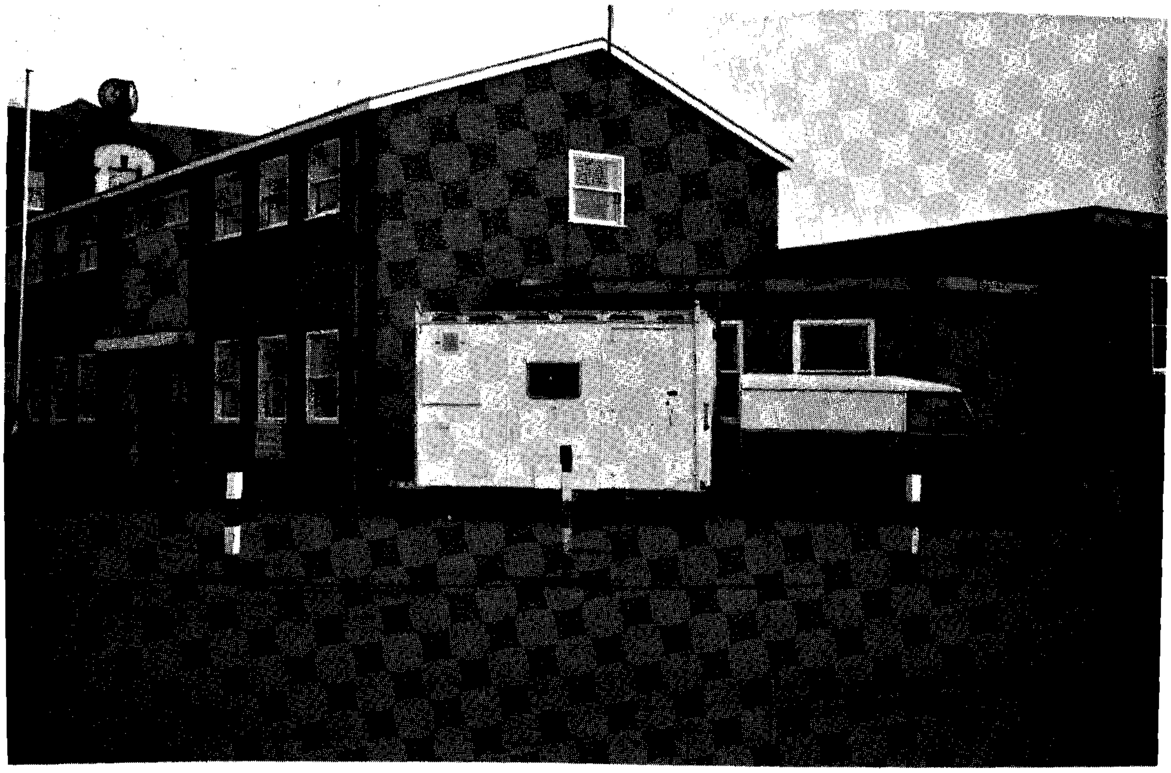
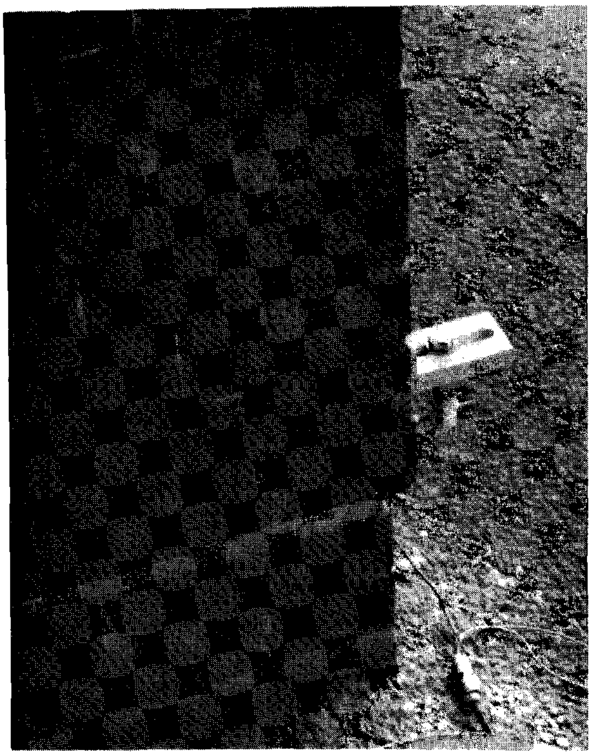
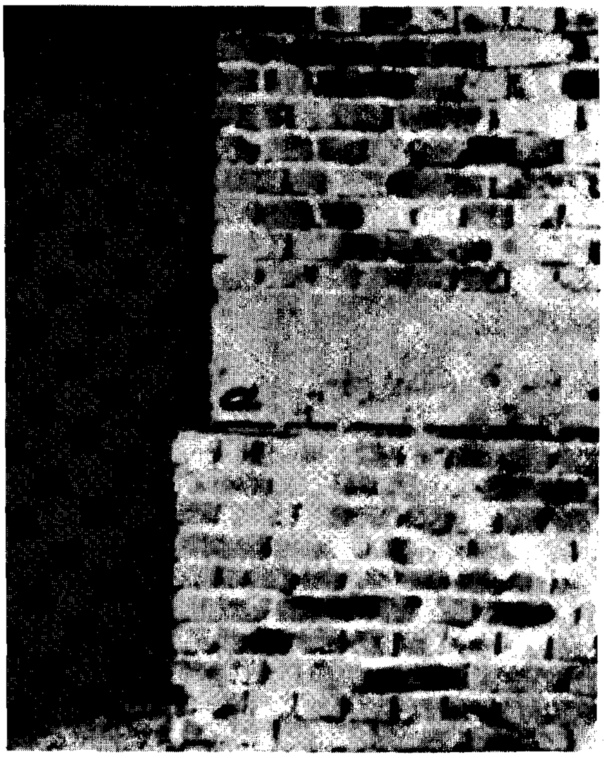


FIGURE 12. MISSION BUILDING WITH RECORDING VAN IN FOREGROUND

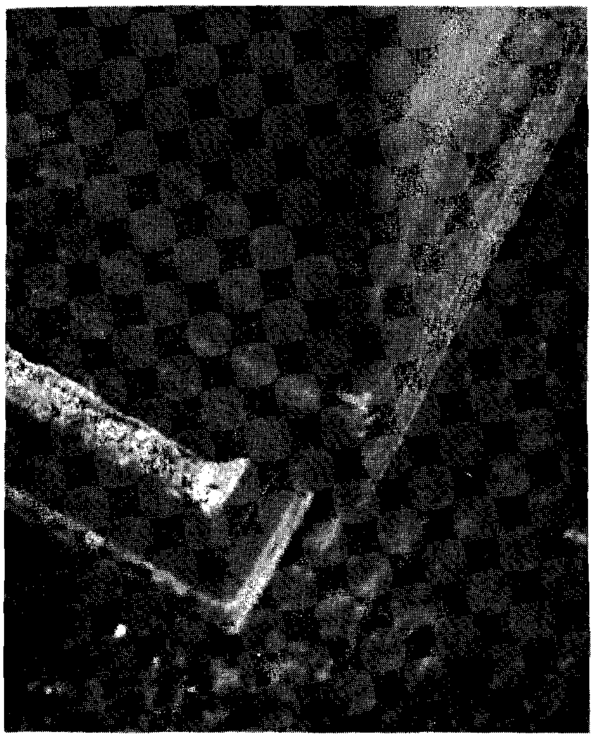


MISSION BUILDING AT IMMINGHAM

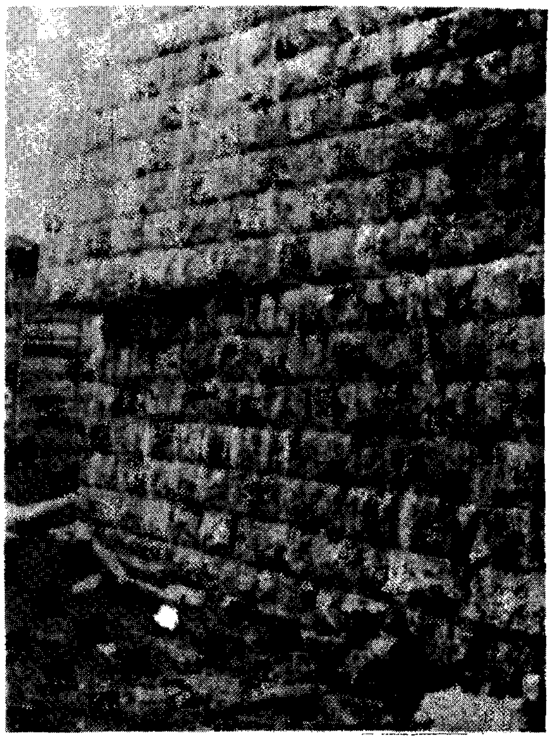


EARTHQUAKE DAMAGED BUILDING IN JAPAN

FIGURE 13. SHEAR FAILURES



MISSION BUILDING AT IMMINGHAM



EARTHQUAKE DAMAGED BUILDING IN JAPAN

FIGURE 14. TWISTING EFFECTS

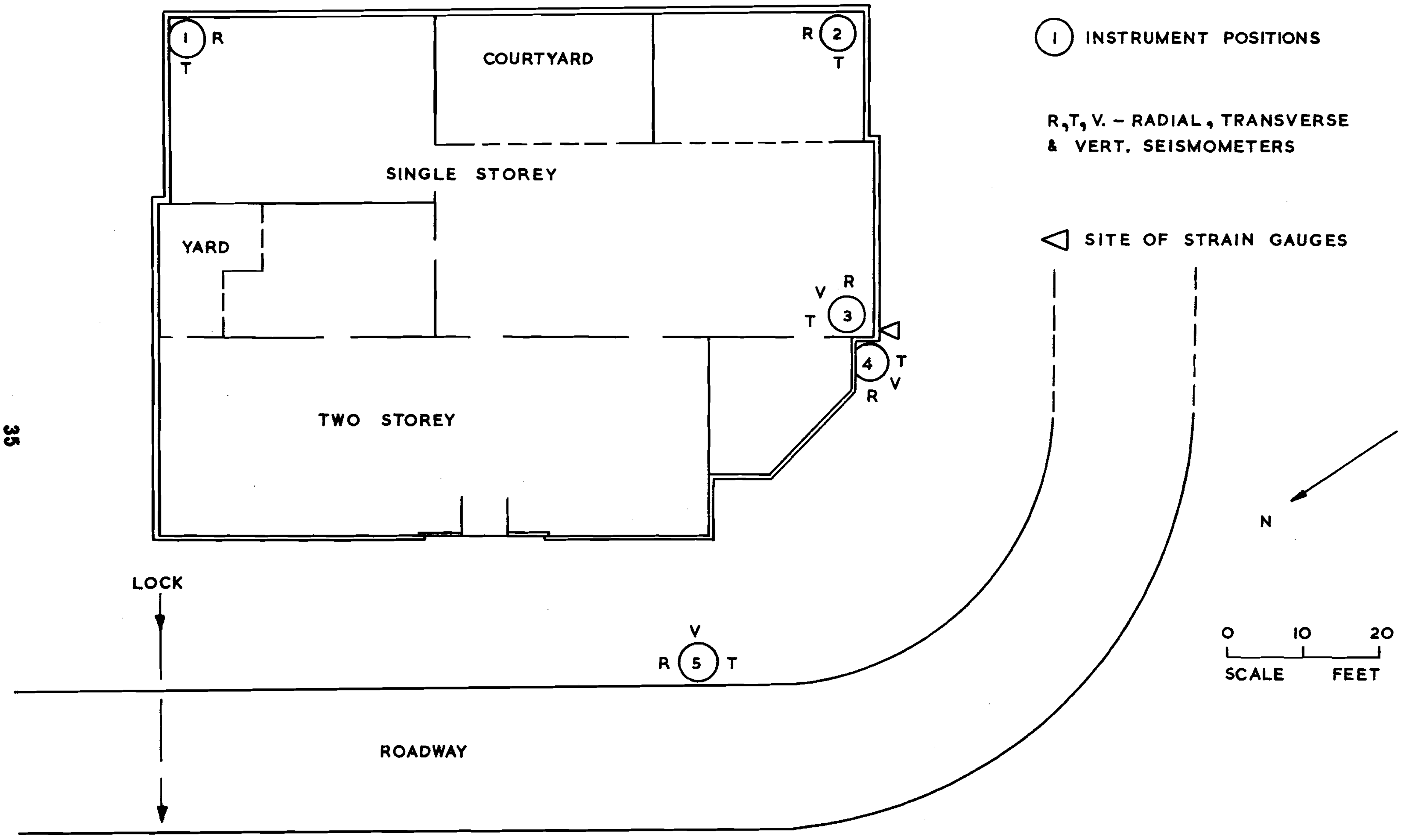



FIGURE 15. PLAN OF MISSION BUILDING AT IMMINGHAM SHOWING INSTRUMENT POSITIONS

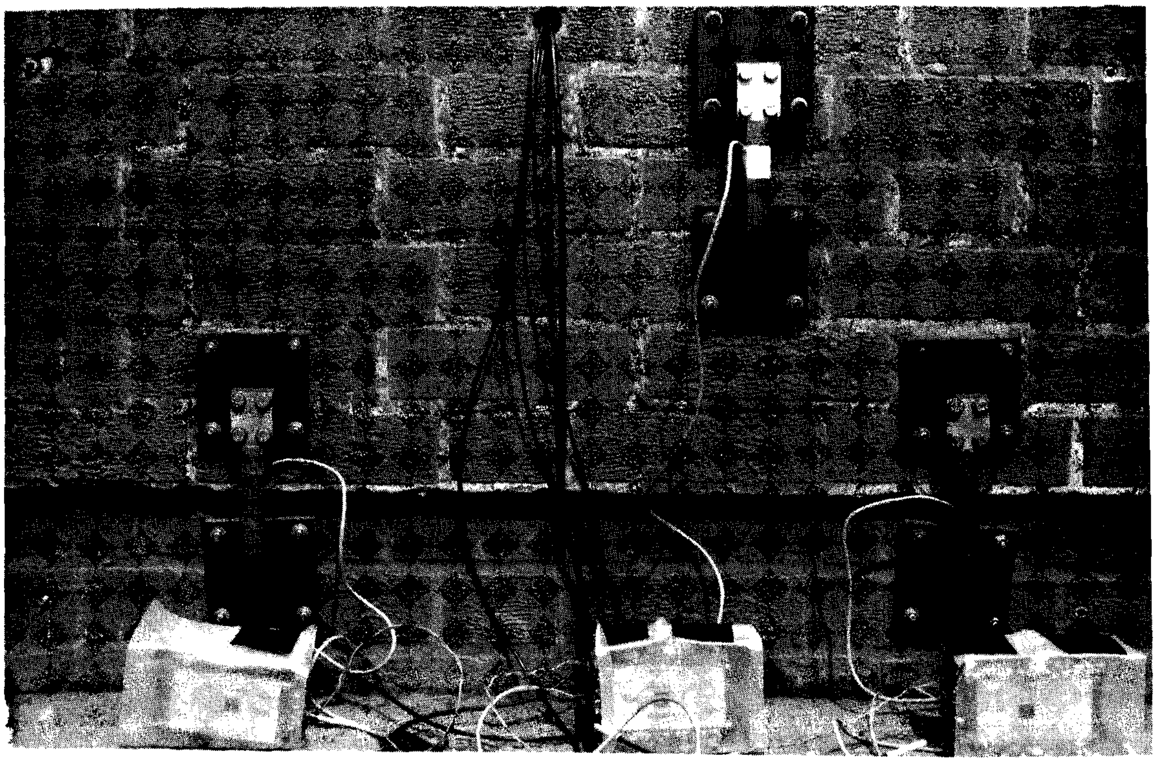


FIGURE 16. STRAIN GAUGES MOUNTED ON SOUTH WEST WALL

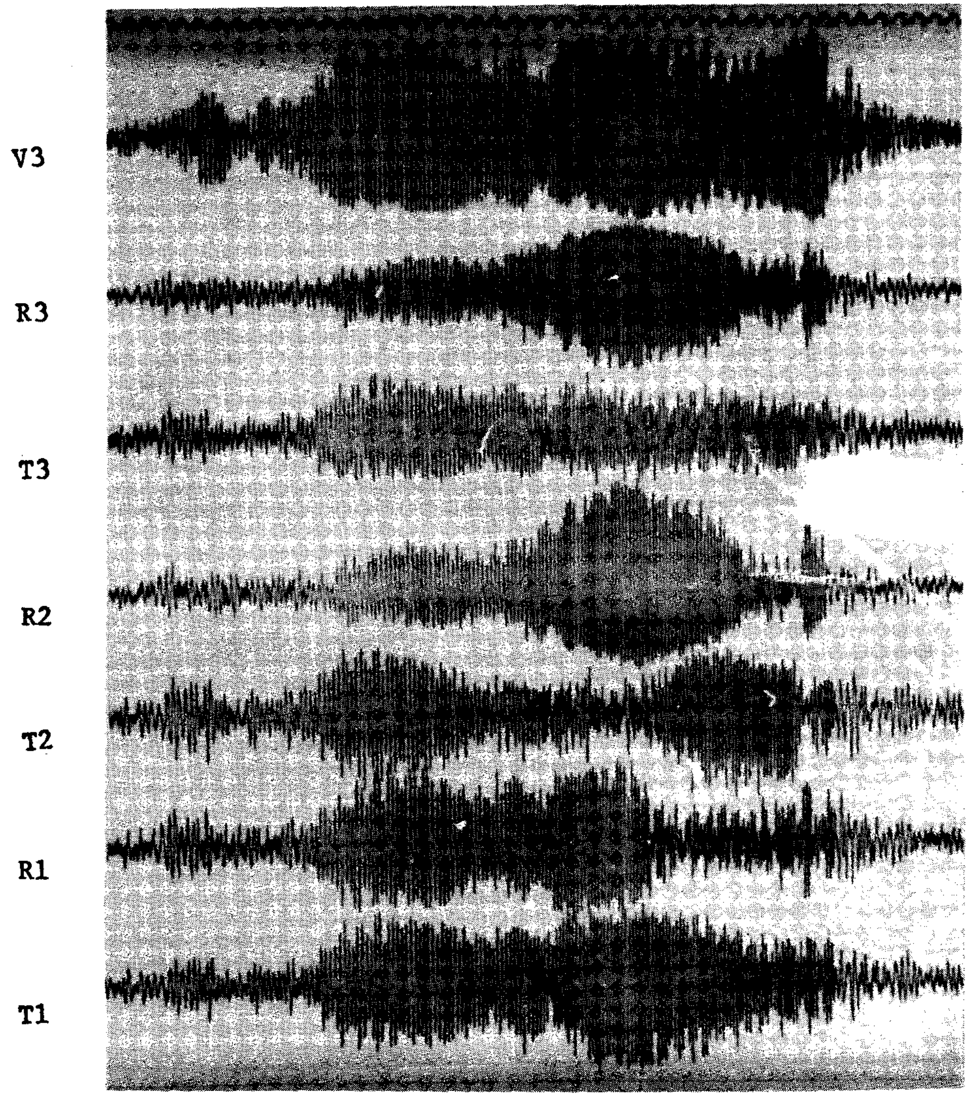



FIGURE 17. SHIP ENGINE VIBRATIONS

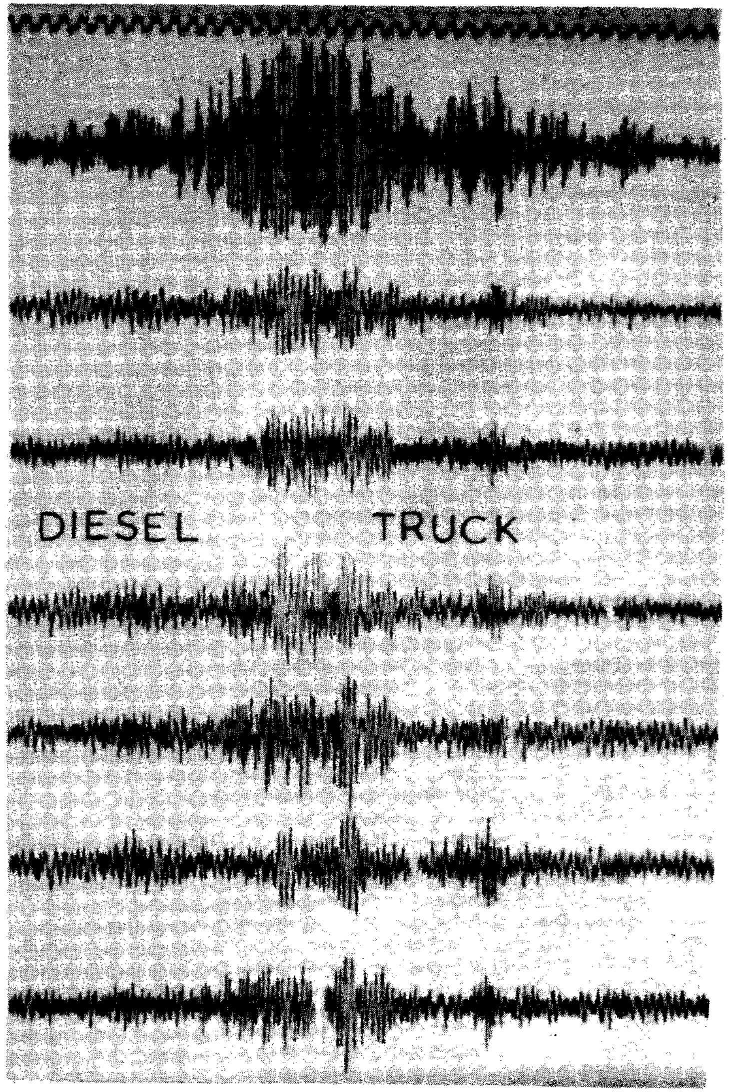



FIGURE 18. VEHICLE VIBRATIONS

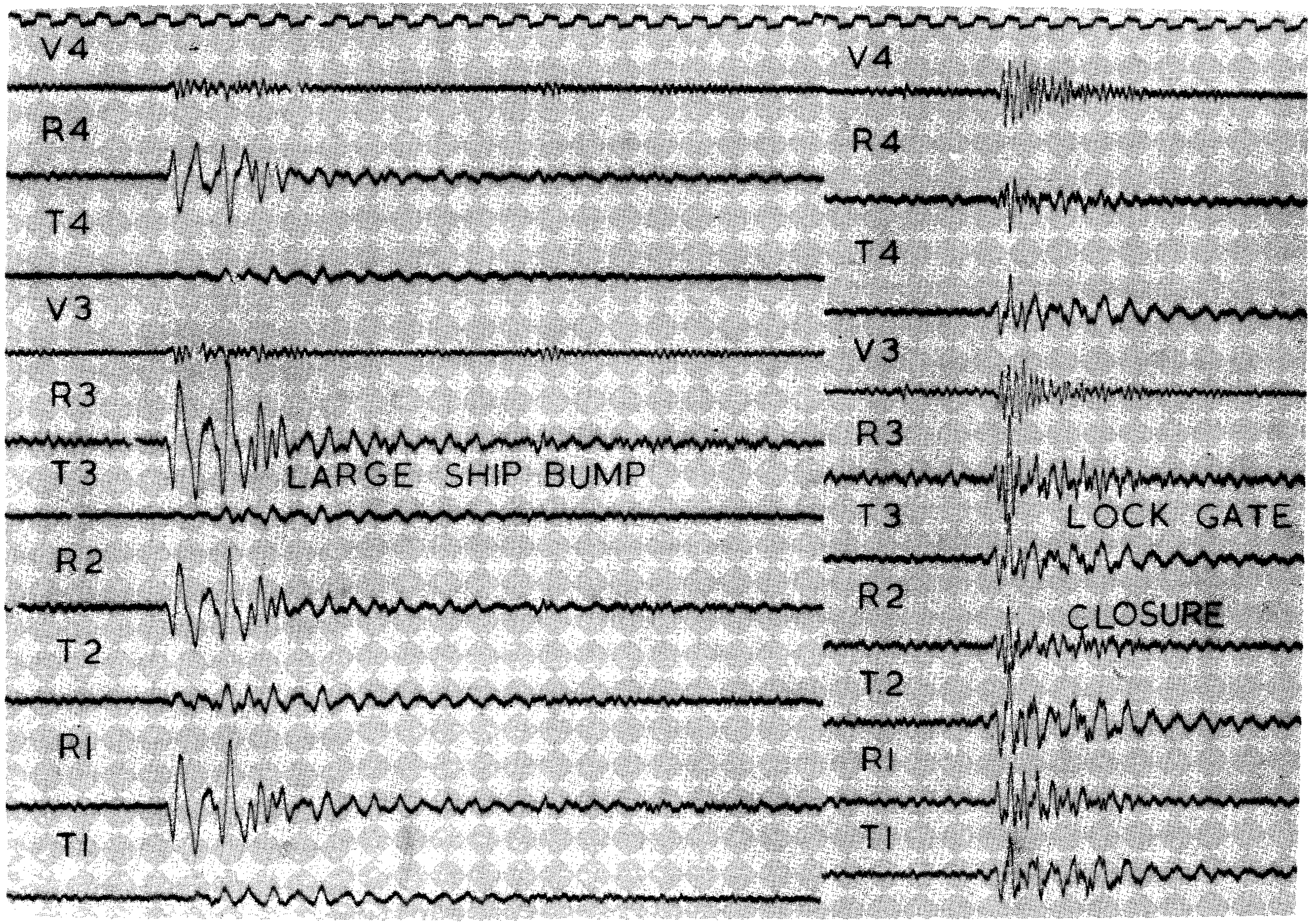
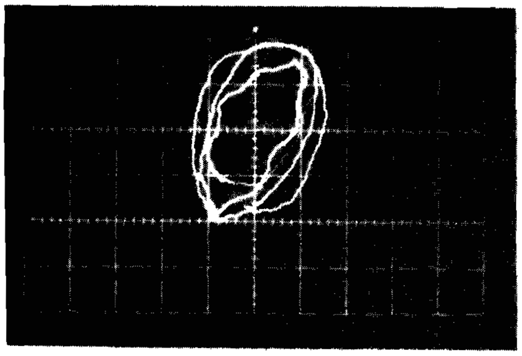
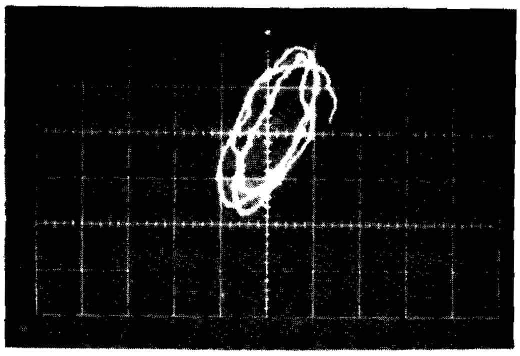



FIGURE 19. VIBRATIONS CAUSED BY SHIP BUMPING SIDE OF LOCK AND BY CLOSURE OF LOCK GATES

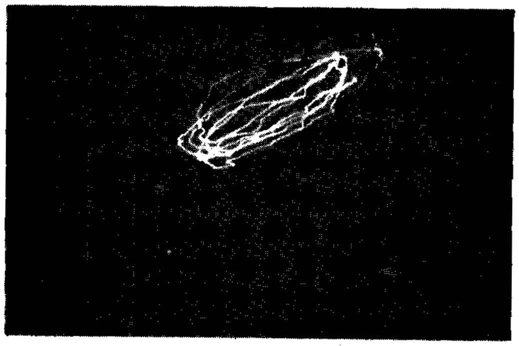


A

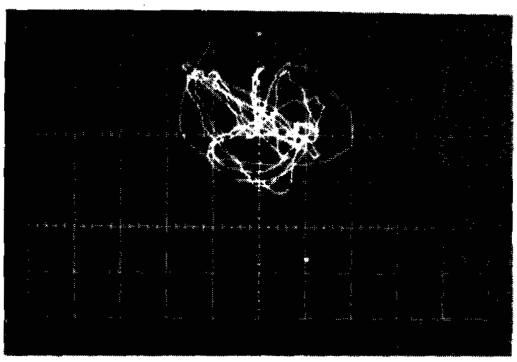


B

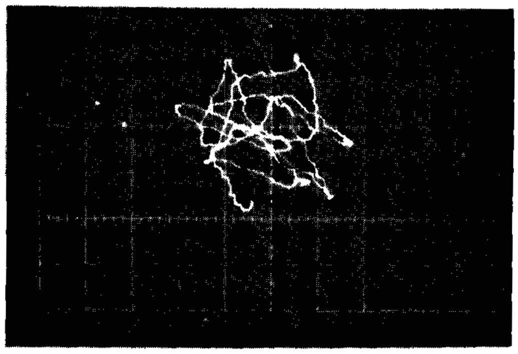
C



D



E



F

FIGURE 20. PARTICLE ORBITS

| ORBIT | INSTRUMENT SITE | VIBRATION SOURCE |
|---|---|---|
| A AND B | 1 AND 2 | SHIP NO. 1 |
| C AND D | 1 AND 2 | SHIP NO. 2 |
| E AND F | 1 | LARGE VEHICLES |

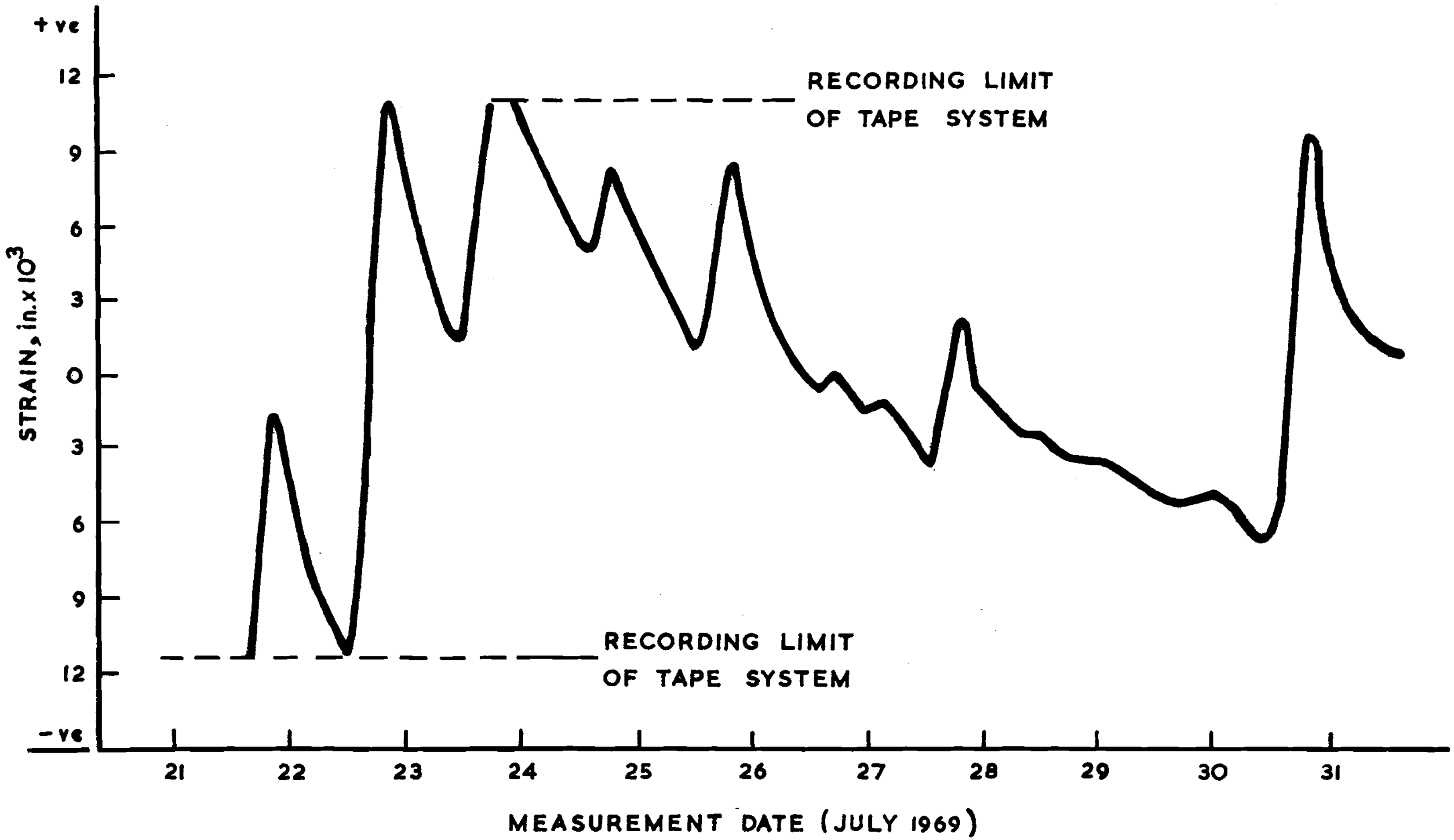



**FIGURE 21. SECTION OF HIGH GAIN STRAIN GAUGE RECORD**

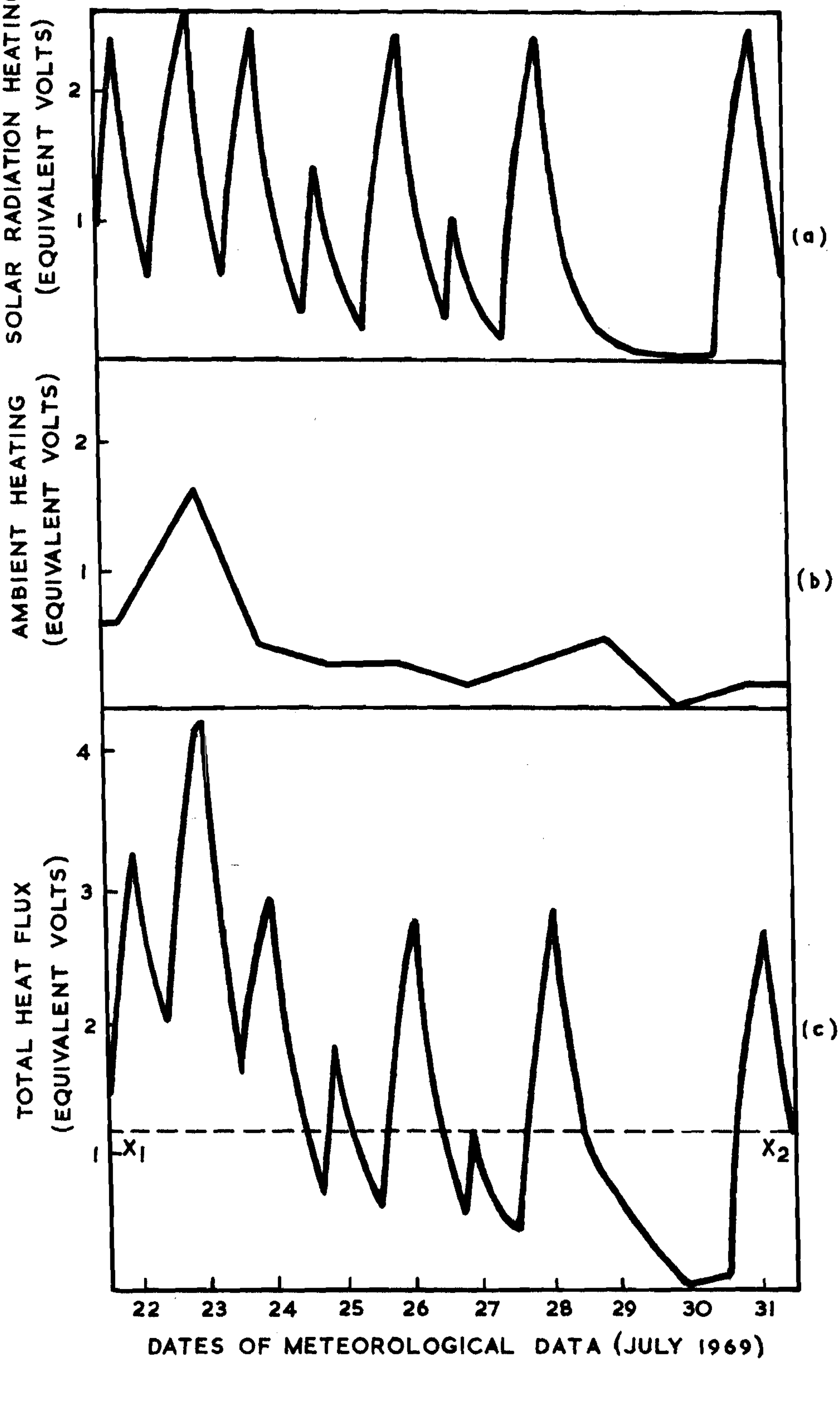

SOLAR RADIATION HEATING
(EQUIVALENT VOLTS)
AMBIENT HEATING
(EQUIVALENT VOLTS)
TOTAL HEAT FLUX
(EQUIVALENT VOLTS)
(a)
(b)
(c)
$X_1$
$X_2$
22 23 24 25 26 27 28 29 30 31
DATES OF METEOROLOGICAL DATA (JULY 1969)

**FIGURE 22**

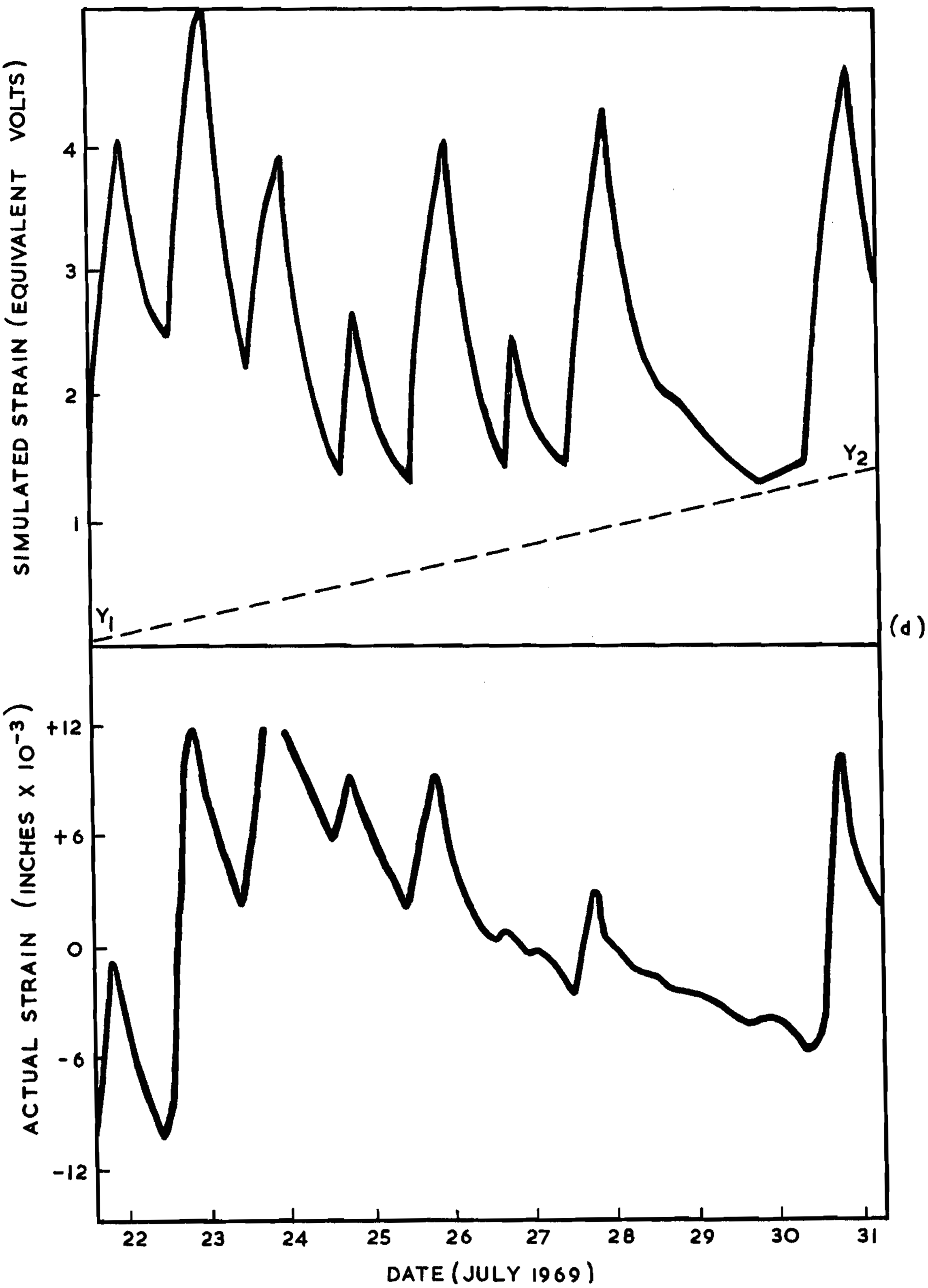



FIGURE 22 (CONT.)

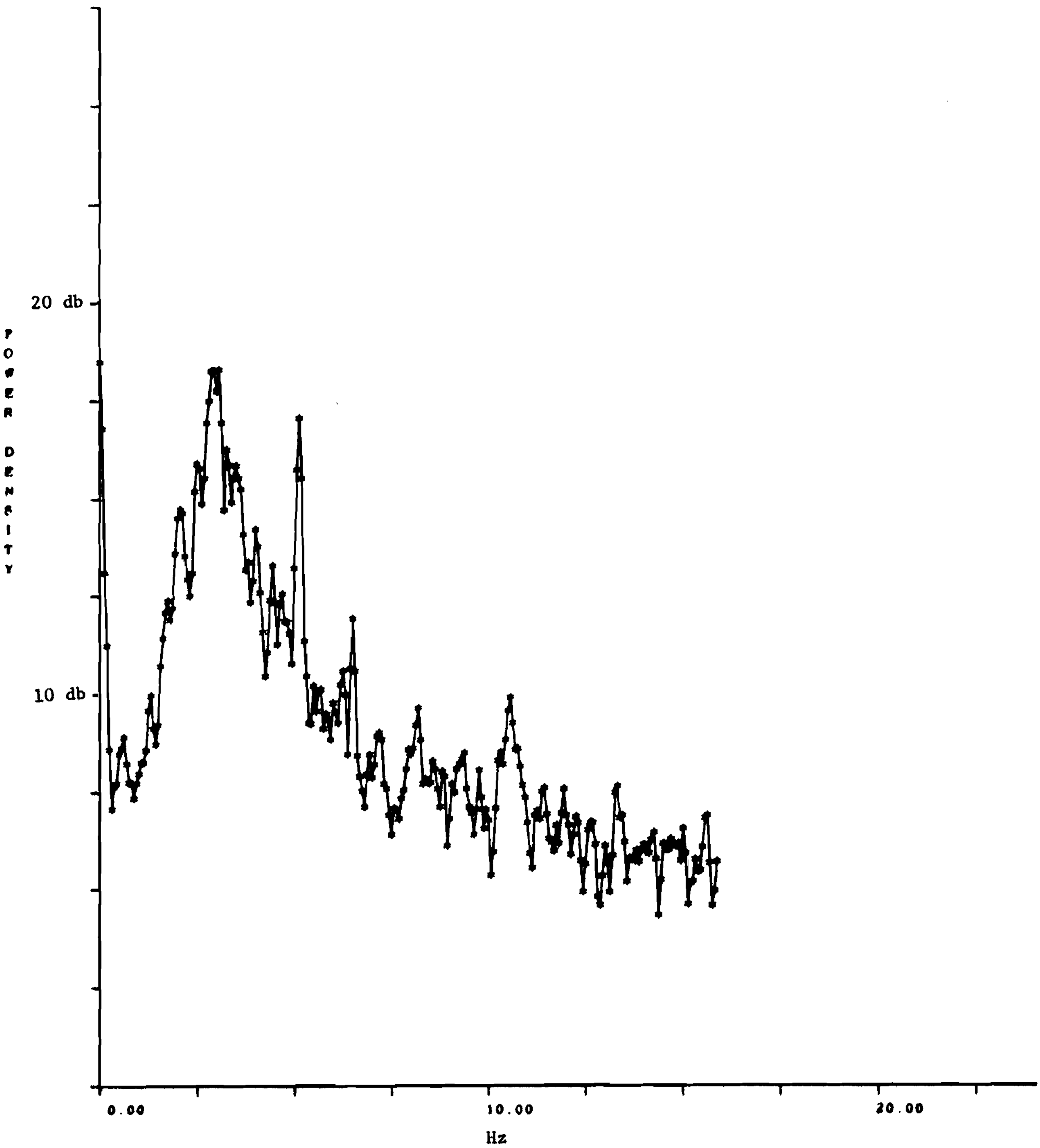



FIGURE 23. NOISE POWER SPECTRUM FROM TRANSVERSE HORIZONTAL SEISMOMETER ON GROUND OUTSIDE BUILDING

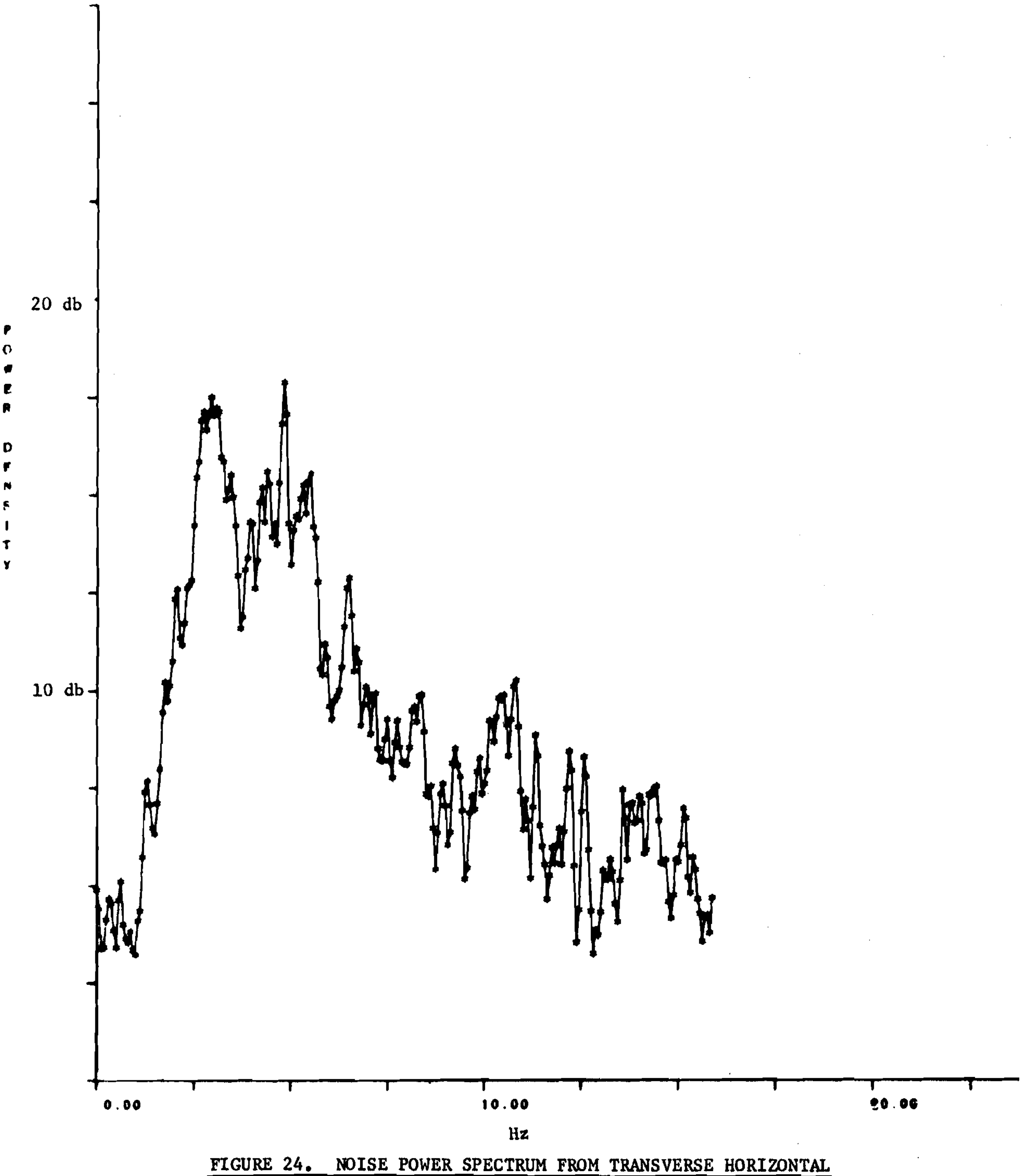



FIGURE 24. NOISE POWER SPECTRUM FROM TRANSVERSE HORIZONTAL SEISMOMETER ON ROOF OF BUILDING

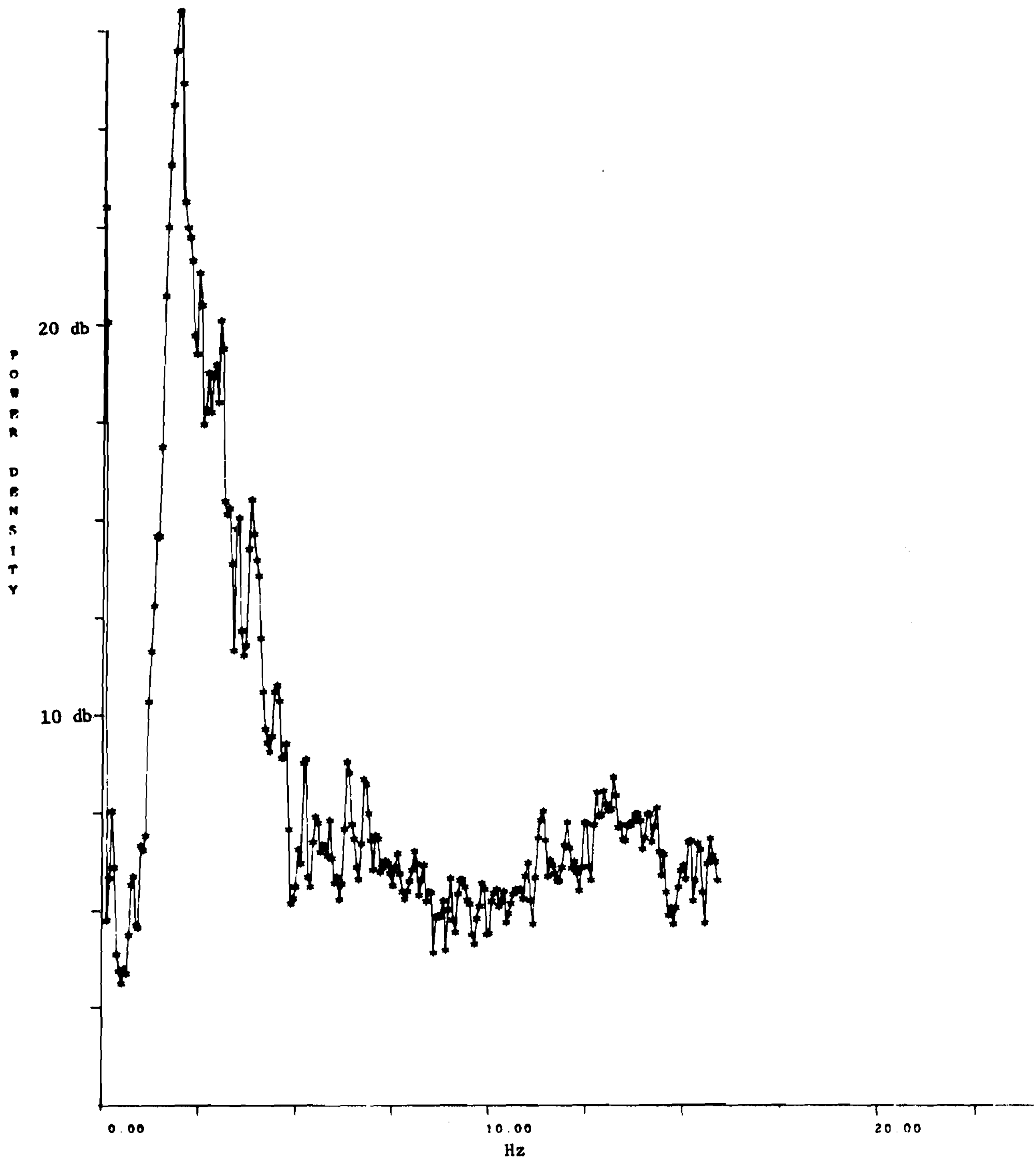



FIGURE 25. NOISE POWER SPECTRUM FROM VERTICAL SEISMOMETER ON SOUTH WEST WALL OF BUILDING

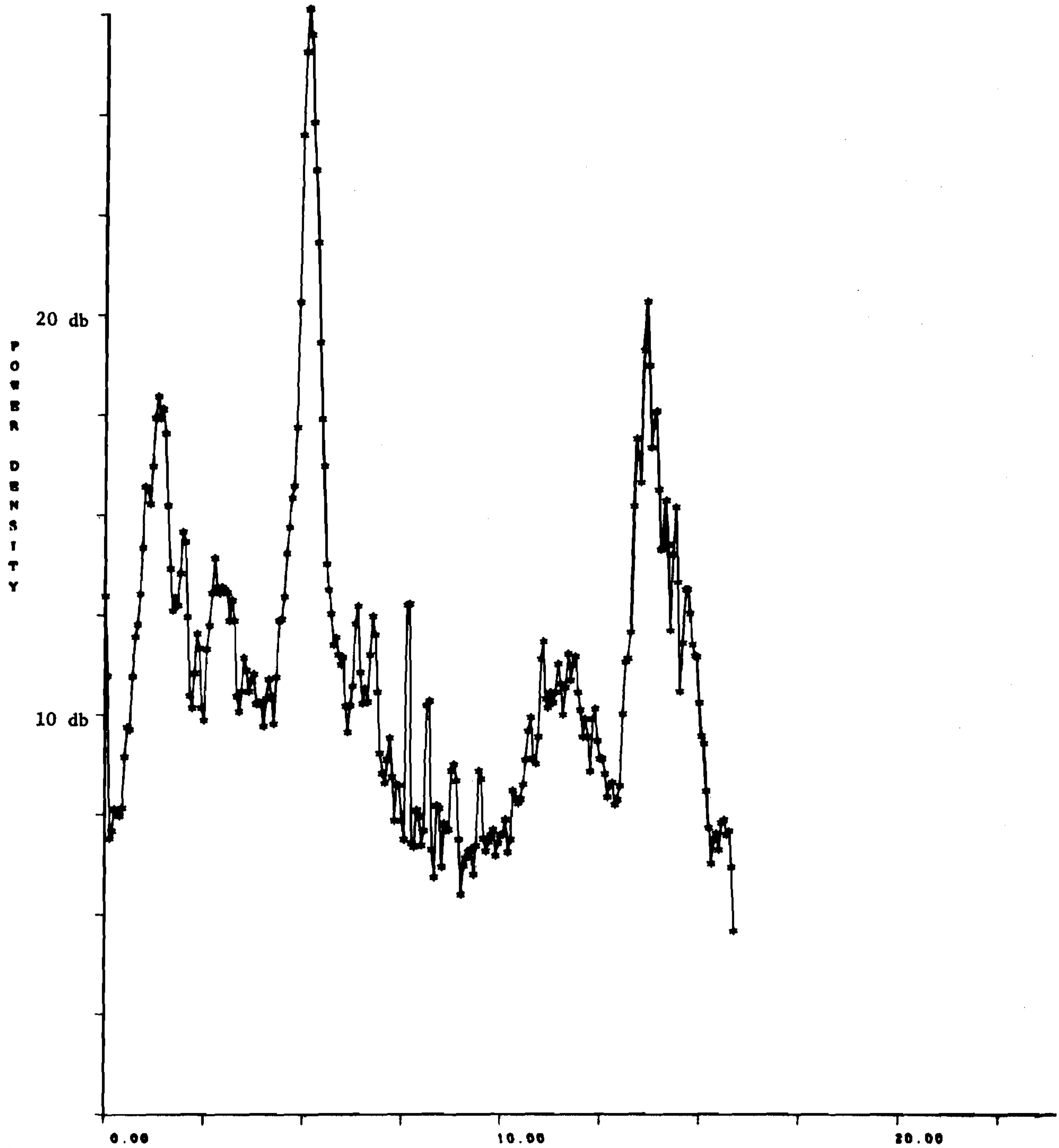



FIGURE 26. NOISE POWER SPECTRUM FROM TRANSVERSE HORIZONTAL SEISMOMETER ON SOUTH WEST WALL OF BUILDING



032/70

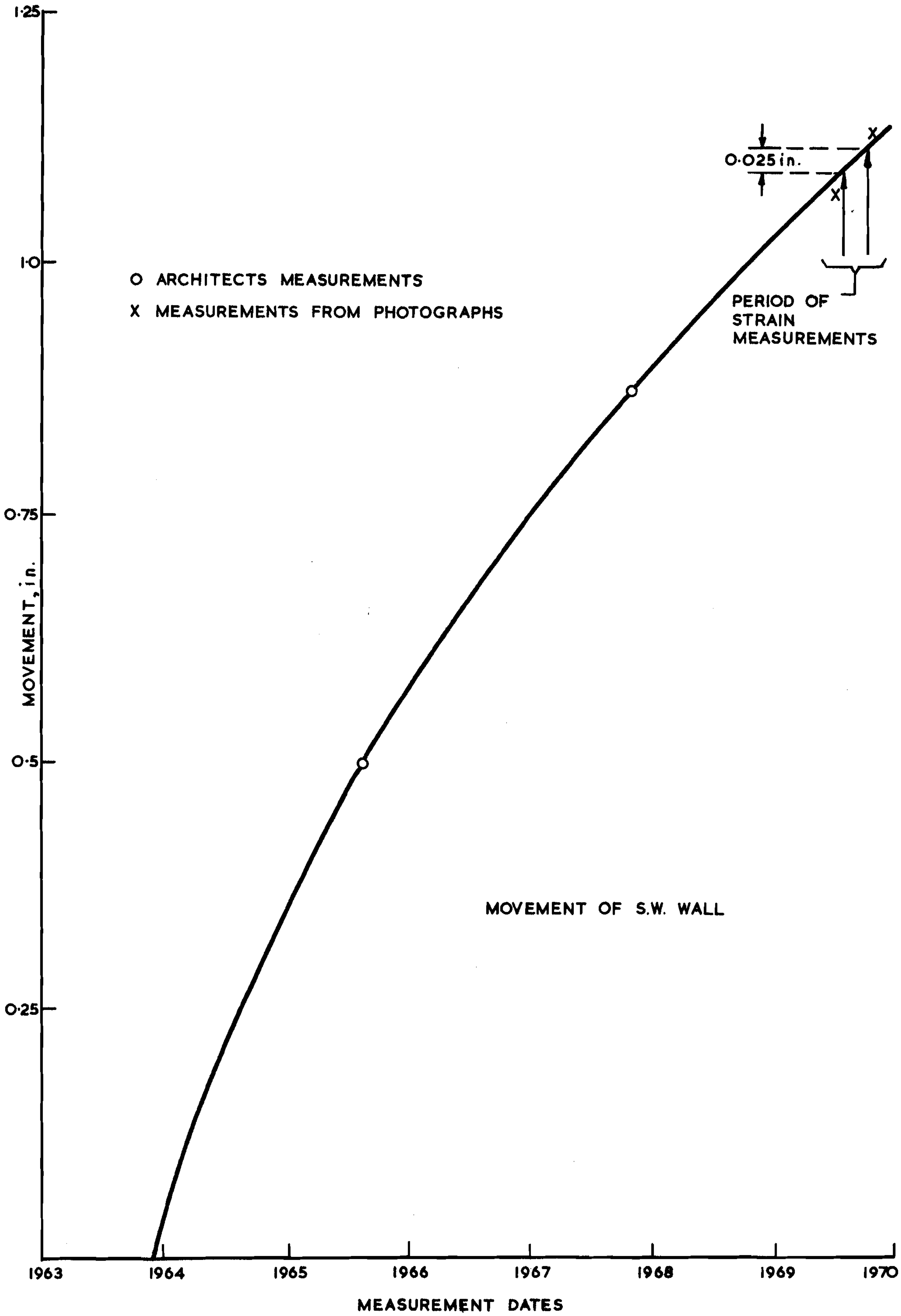



**FIGURE 27. MOVEMENT AT SOUTH WEST WALL CORNER**

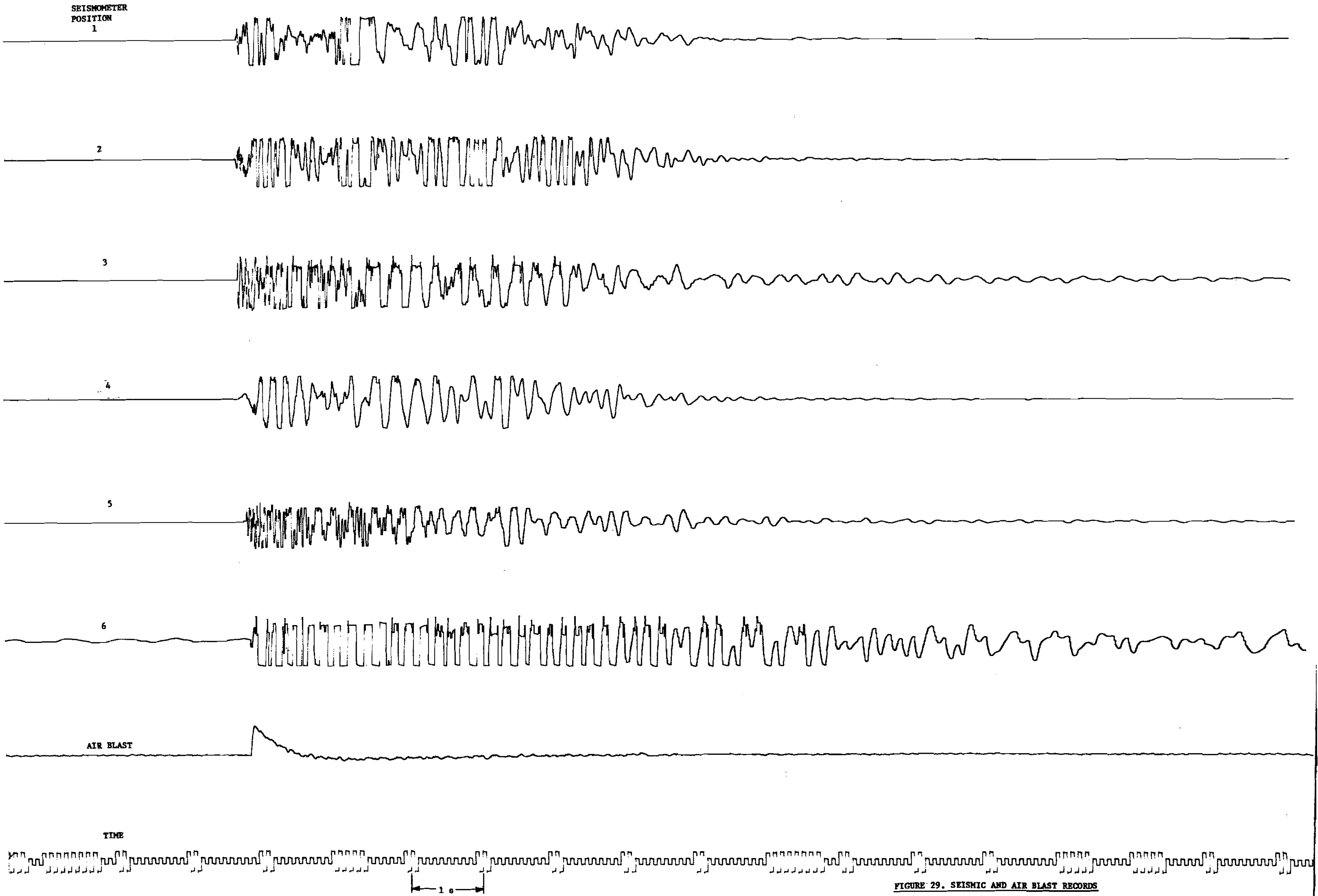



FIGURE 29. SEISMIC AND AIR BLAST RECORDS

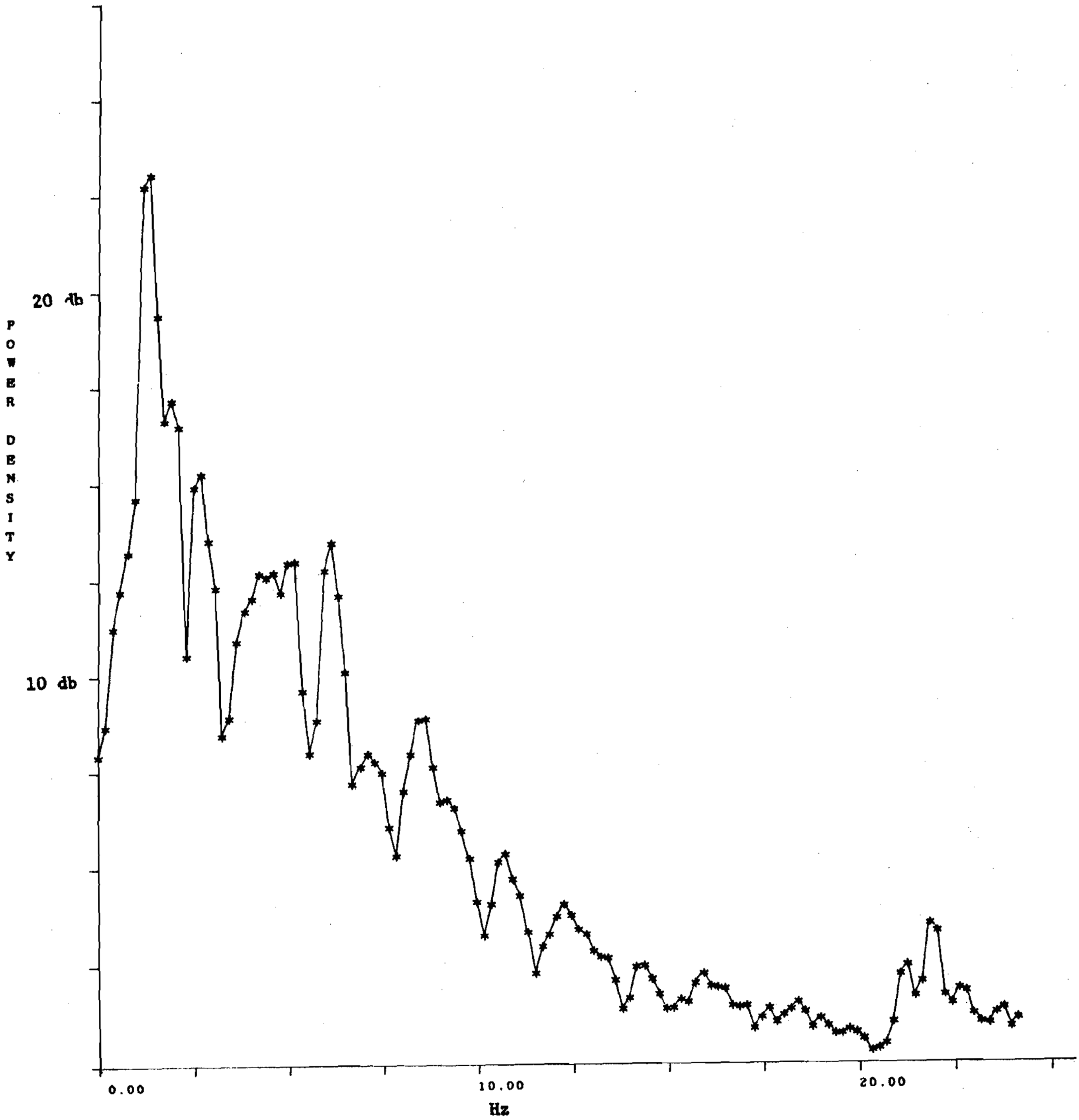



FIGURE 30. FREQUENCY SPECTRUM OF COOLING TOWER VIBRATIONS

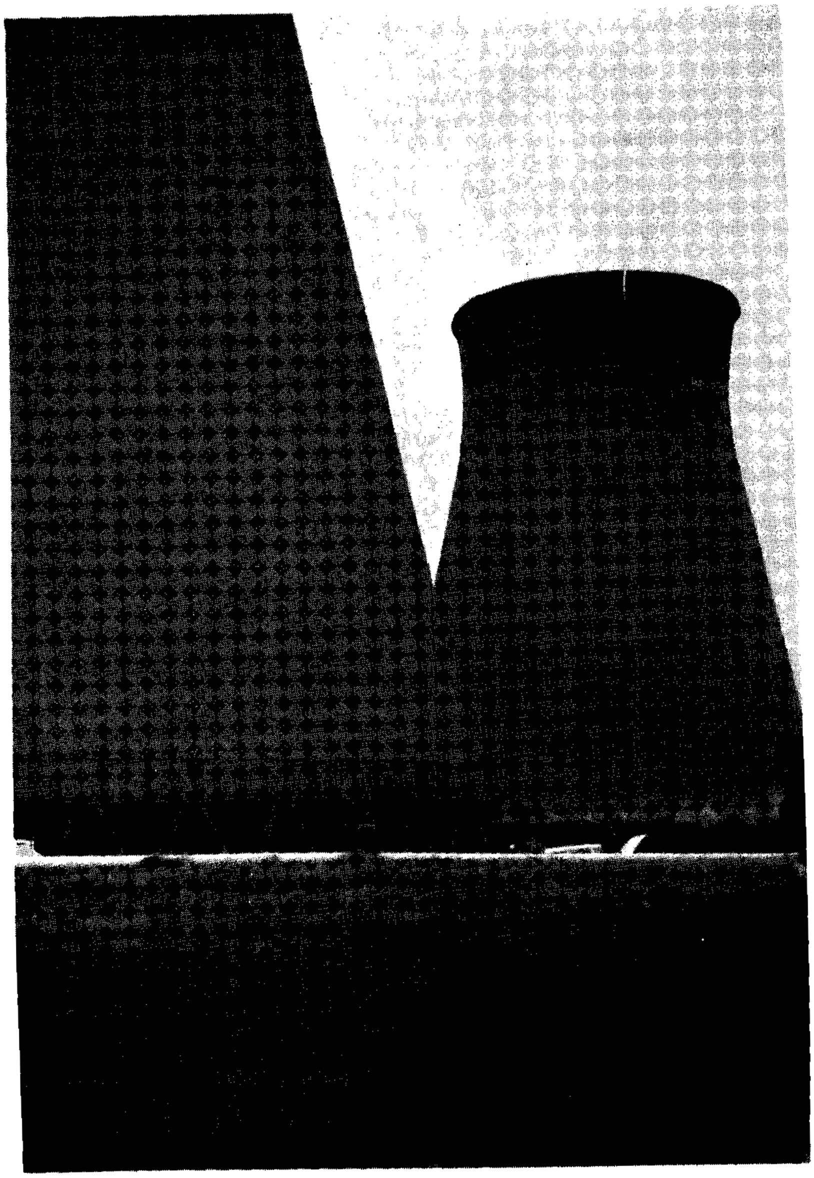


FIGURE 31. EARLY STAGE IN COLLAPSE OF TOWER NO. 3

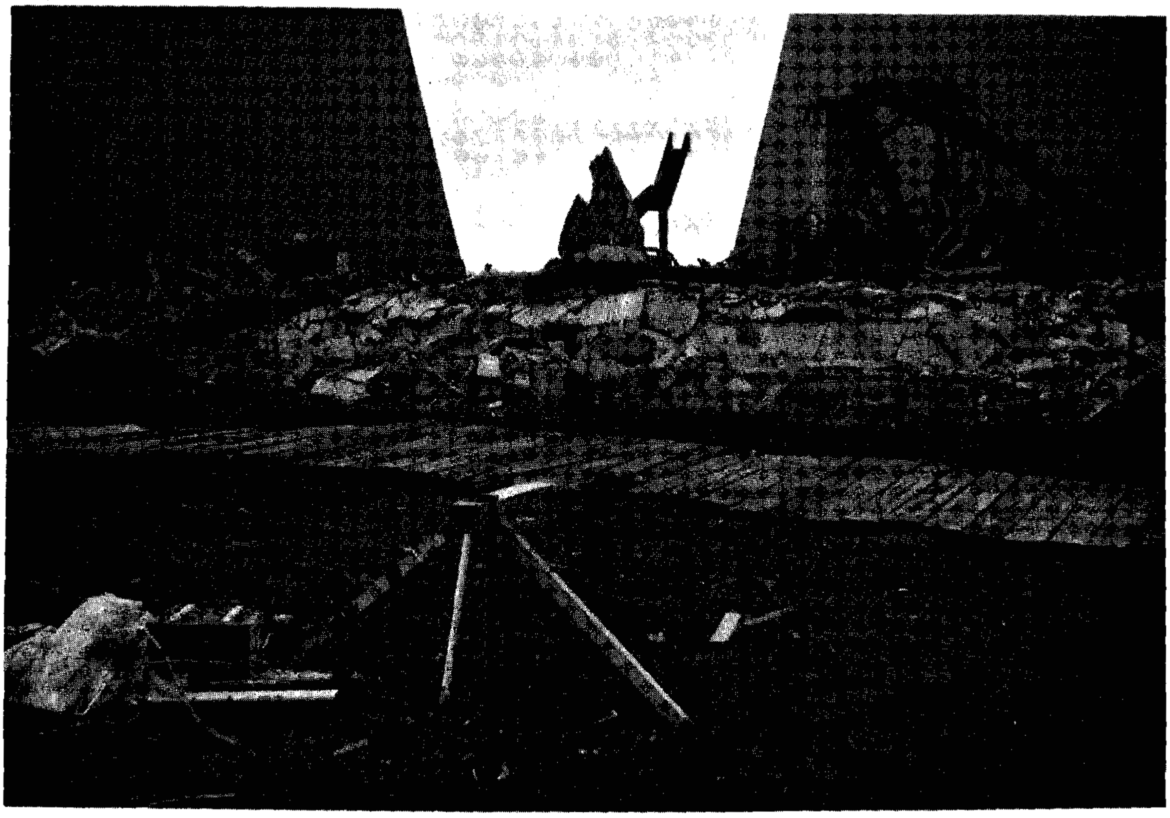


FIGURE 32. DEMOLISHED COOLING TOWER WITH BLAST GAUGE MOUNTING IN FOREGROUND