Observed Vibrations of a Nuclear Reactor Building During Some Weak Earthquakes

by
H. Tajimi*, M. Ohmura**, T. Uchida***,
and K. Akino****

Abstract

A seismic observation system of the container building of the Japan Power Demonstration Reactor has recently been put in operation. This paper presents some of the measurement data and the results of analysis. As a result, an outline of vibrational properties of this building is derived.

Introduction

Nuclear reactor buildings are made generally to be very heavy and rigid. For the purpose of obtaining the basic data on the earthquake response of such a kind of structure, the authors have planned the seismic observation of the container building of the Japan Power Demonstration Reactor (JPDR), which was constructed at the site in the Tokai Establishment of the Japan Atomic Energy Research Institute and has been operated since September 1963. The observation has started at the same time with its completion and now is continued. Owing to the short period up to the present, no more than five weak earthquakes have been recorded. This paper describes an outline of the measurement data and their tentative analyses, particularly with respect to the vibrational properties of this structure.

Brief Description of Structure and Soil Conditions(1)

The containment structure which is 15.25 meters in diameter and 38.10 meters in height, is cylindrical shaped vessel with hemispherical ends, made of welded steel plates containing internal concrete structures, and is supported firmly on the massive foundation, as it is buried 14.55 meters in depth into the caisson and filled up by concrete. The profil of this structure is drawn schematically with the subsoil conditions beneath it in Fig. 1. The bottom of the foundation rests directly upon the sandy shale, lying at a depth of 17.2 meters below the ground surface.

Observation

The measurement instruments consist of seismometers, soil pressure gauges, clinometers, recording apparatus and a starting device. The block diagram of the observation system is illustrated in Fig. 5. For the container building, eight accelerometers are installed in the cells provided on the head of the caisson wall and designated respectively by A_1 to A_8, in which A_1 to A_4 are horizontal accelerometers and A_5 to A_8 are vertical accelerometers. The cells are located at the four points which quadrante

* Professor of Structural Engineering, Nihon University
** Chief, Construction Division, Japan Atomic Energy Research Institute
*** Construction Division, Japan Atomic Energy Research Institute
**** Construction Division, Japan Atomic Power Co. Ltd.

the circumference of the caisson (see Fig. 6) and each contains a pair of accelerometers, one of which serves as a pick-up of the horizontal component towards the tangential direction to the circumference and the other as a pick-up of the vertical component. The control building adjacent to the container building is used as a center of the observation system, where other two accelerometers are installed together with the recording apparatus and other devices. All accelerometers are the same type of electromagnetic (MTDH-3C) and substantially alike to that already reported[2]. They are connected with an electromagnetic oscillograph, which operates automatically by an excitation of the starter.

Data and Analyses

Table 1 lists the data concerning the five earthquakes which were recorded. Samples of tracing of earthquake records are indicated in Fig. 7 and 8. (In these figures, the records of A_2 and A_6 are omitted, because the cell which holds these seismometers was submerged by a rain.) In addition, the resultant acceleration of two horizontal components at the center of the caisson area obtained from the measured records are drawn in a form of trajectory diagram, as shown in Fig. 9.

The analysis of earthquake records has been made for the following items:

(i) Frequency curve of period.

(ii) Spectrum.

(iii) Observed mode of rocking vibration.

Some examples of these results are presented in the paper (Figs. 10, 11 and 12). As to the terminology used in item (i), the "period" is defined here as the time which is twice as large as the interval between successive zeros of an accelerograme. For item (ii), two kinds of spectra[3] of the acceleration $\alpha(t)$ are presented, one of which is the Fourier spectrum defined by

$$F(\omega)=\sqrt{\left[\int_0^T \alpha(\tau)\cos\omega\tau\, d\tau\right]^2+\left[\int_0^T \alpha(\tau)\sin\omega\tau\, d\tau\right]^2} \qquad (1)$$

where T denotes the duration of the earthquake, and the other is the zero damped response spectrum defined by

$$S_v=\left(\sqrt{\left[\int_0^t \alpha(\tau)\cos\omega\tau\, d\tau\right]^2+\left[\int_0^t \alpha(\tau)\sin\omega\tau\, d\tau\right]^2}\right)_{max} \qquad (2)$$

for $0<t<T$. In this analysis, the duration is taken as five seconds from the beginning of the principal part of the earthquake.

For item (iii), it is convenient to take account of a distribution of vertical and horizontal accelerations along a diameter of the caisson. Fig. 10 shows successive variation of such distributions versus the time, while they are confined to an initial stage of the earthquake and associated with the maximum acceleration. This figure indicates a explicit form of rocking motion and suggests the possibility of obtaining the position of the nodal axis of rocking motion from the radius of caisson multiplied by the ratio of the horizontal acceleration to the vertical acceleration. Then it

needs a presumption that the maximum response acceleration may occur with a phase difference of about ninety degrees to the ground acceleration, in consistency with the state of "quasi-resonance". This phenomenon, however, can be approximately recognized in a graphical comparison of the measured horizontal acceleration to the corresponding rocking component of vertical acceleration (Fig. 13). The latter means here a half of difference between the vertical accelerations measured at the two points, which are located symmetrically with respect to the center of the caisson.

Discussion of Results

The following conclusions were obtained as a result of the tentative analysis given:

(1) The natural period of this structure is about 0.22 seconds.

(2) The nodal axis of the rocking vibration in the first mode is located approximately at a distance of 2.35 meters below the bottom surface of the foundation.

The following description relates to an examination in the reliability of the above results. In general, the natural frequency of the rocking vibration in the first mode, f_1, is given by

$$f_1 = \frac{1}{2\pi}\sqrt{\frac{K_1}{I_1}} \tag{3}$$

where I_1 = mass moment of inertia of the structure with respect to its nodal axis of rocking in the first mode,

K_1 = spring constant of soil system for the rocking motion of the structure in the first mode.

These constants I_1 and K_1 are, in turn, expressed in the form,

$$\begin{aligned} I_1 &= M(i_0^2 + z_1^2) \\ K_1 &= K_h\{e_0^2 + (z_1 - s)^2\} \end{aligned} \tag{4}$$

where M = mass of the structure,

K_h = spring constant of soil system for the purely lateral movement of the structure,

i_0 = radius of gyration of the structure about its center of gravity,

z_1 = distance of the nodal axis of the first mode from the center of gravity,

e_0 = elastic radius of the foundation area of the structure, which means the square root of the ratio of the soil spring constant for pure rotation to that for pure translation.

In this case the following relations exsit:

$$\begin{aligned} z_1 \cdot z_2 &= -i_0^2 \\ (z_1 - s)(z_2 - s) &= -e_0^2 \end{aligned} \tag{5}$$

where z_2 = distance of the nodal axis of the 2nd mode from the center of gravity.

Prior to the calculation of the above expressions, the distance "s" should be determined from the measurement. For this purpose, many earth pressure gauges were mounted to the caisson, but no satisfactory results have been obtained, because of being much lower in the sensitivity of gauges for the observed earthquakes. Hence, an assumption is made in this analysis that the line of action of the spring K_h is situated at a distance of 1.0 meter to 1.5 meters above the bottom surface of the foundation. Let s' denote this distance.

Using z_1 from the measurement results and s' thus assumed, the values of z_2 and e_o are determined from the expression (5). These results are indicated in Table 2, for the case of s' = 1.0 m. According to the theoretical investigation of the e_o-value for a circular footing resting upon the elastic soil, it is found to nearly equal to 0.8R, where R denotes the radius of the footing. It might be supposed, however, that the actual value of e_o would be reduced to (0.7~ 0.75)R, because the actual foundation does not so perfectly contact with the soil as that assumed in the theory. Such reduction of e_o would be still applied to the present case that the caisson has small effects on the soil reaction. In this sense, the value of e_o above obtained satisfies the theoretical prediction.

Thus the total lateral stiffness K_h can be calculated from the expressions(3) and (4). Apparently, the stiffness K_h is divided into the two parts, one of which is related to the bottom surface of the foundation, and the other related to the caisson wall. Now, using the notation of k_h for the coefficient of subgrade reaction against the bottom surface, the notation of k_q for the coefficient against the projected area on the vertical section of the caisson wall in contact with the sand stratum, and furthermore assuming the coefficient of $2.5k_q$ for that in contact with the gravel stratum, the total stiffness K_h can be expressed as

$$K_h = k_h \cdot A + k_q \cdot 2R \cdot l_a + 2.5\, k_q \cdot 2R \cdot l_b \qquad (6)$$

where l_a = thickness of sand stratum,
l_b = thickness of gravel stratum,
A = area of the bottom surface,
R = radius of the foundation.

Since the position of the line of action of K_h has been given by s' from the bottom surface, the following relation exists:

$$K_h \cdot s' = k_q \cdot 2R \cdot (l_a/2 + l_b) + 2.5\, k_q \cdot 2R \cdot l_b/2 \qquad (7)$$

From the expressions (6) and (7), the values of k_h and k_q are determined, as indicated in Table 3. (In this tentative analysis, the weight of the whole structure is taken as 12,653 tons and its radius of gyration, i_o, is 8.98 m.)

The theoretical expressions of k_h for a circular footing with radius R is written as

$$k_h = \frac{2G}{2-\mu} \frac{1}{R} \qquad (8)$$

where G = shearing modulus of soil,
μ = Poisson's ratio of soil.

Introducing the numerical values of k_h above obtained into the expression (8) with an assumption of $\mu = 0.4$, the shearing modulus G can be obtained as about 8×10^3 kg/cm^2 and 6×10^3 kg/cm^2 for both cases of s' given. These values would be admitted to be reasonable for the dynamic modulus, comparing with the static value $G = 2 \times 10^3$ kg/cm^2 derived from the data of the bearing test on the same stratum.

Reference

(1) Devision of JPDR Project, "Description and Hazard Analysis of Japan Power Demonstration Reactor", JAERI 6005, June 1960, (in Japanese).

(2) E. Shima, T. Tanaka and N. Den, "Some New Instruments Used in Earthquake Engineering in Japan", Proc. the 2nd W.C.E.E., 1960.

(3) D.E. Hudson, "Some Problems in the Application of Spectrum Techniques to Strong-Motion Earthquake Analysis", Bulletin of Seis. Soc. Amer., 52:417-430.

Table 1

Date	Epicentral distance, km	Recorded max. acceleration, gals
Oct. 16, 1963	21	NS 4.76 EW 6.08
Dec. 19, 1963	48	NS 12.89 EW 11.80
Dec. 24, 1963	63	NS 6.93 EW 17.40
Jan. 10, 1964		NS 6.80 EW 5.32
Feb. 5, 1964	37	NS 19.06 EW 35.70

Table 2

z_1, m	s, m	z_2, m	e_o, m
12.67	9.32	-6.37	7.25

Table 3

s', m	k_h, kg/cm^3	k_q, kg/cm^3
1.0	10.1	1.6
1.5	7.7	2.0

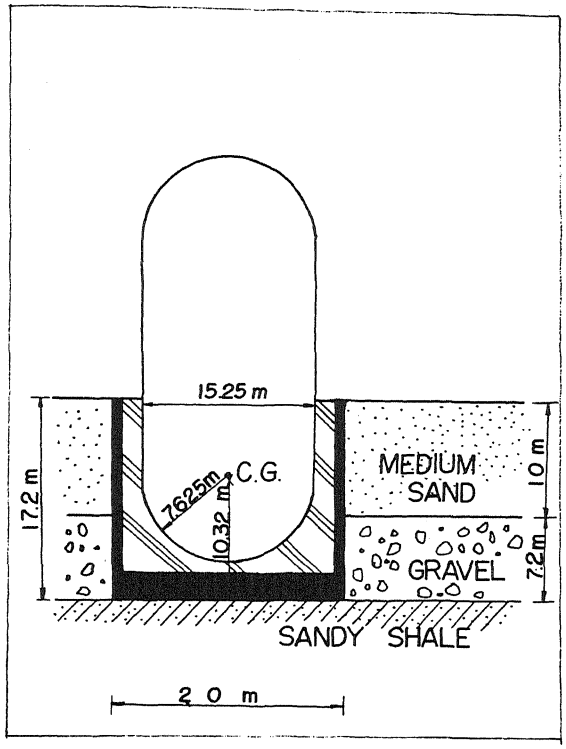


FIG.1

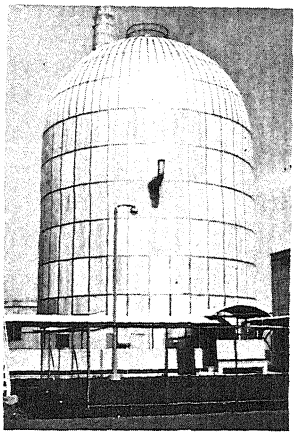


FIG.2

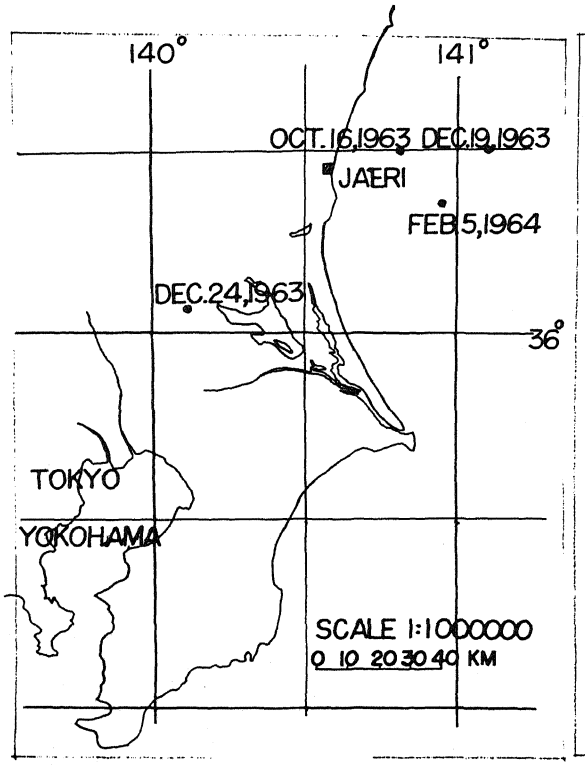


FIG.3

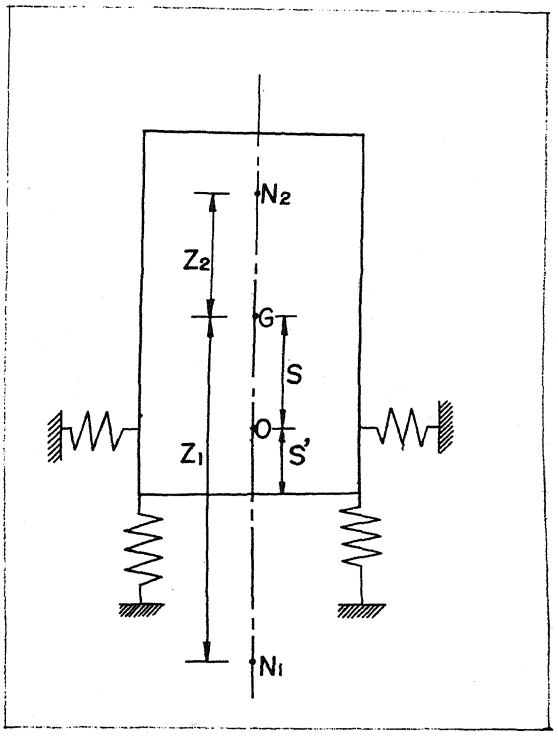
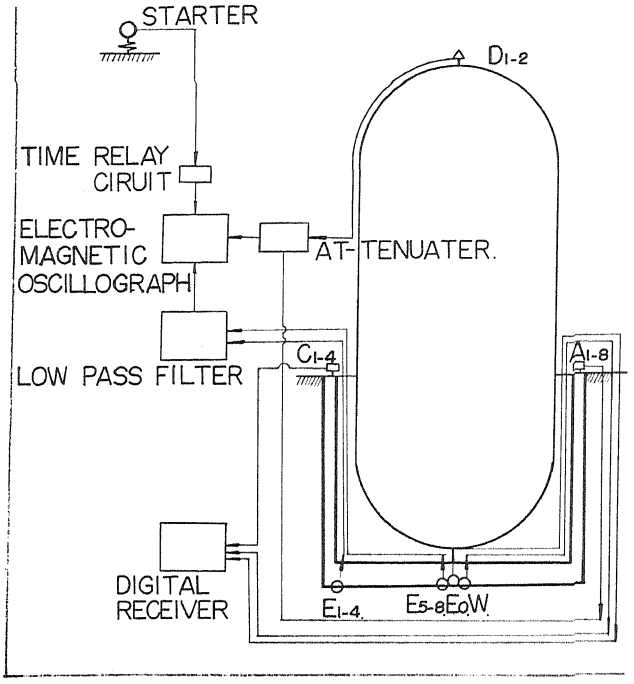


FIG.4



A_{1-4} ACCELEROMETER FOR HORIZONTAL COMPONENT

A_{5-8} ACCELEROMETER FOR VERTICAL COMPONENT

C_{1-4} CLINOMETER

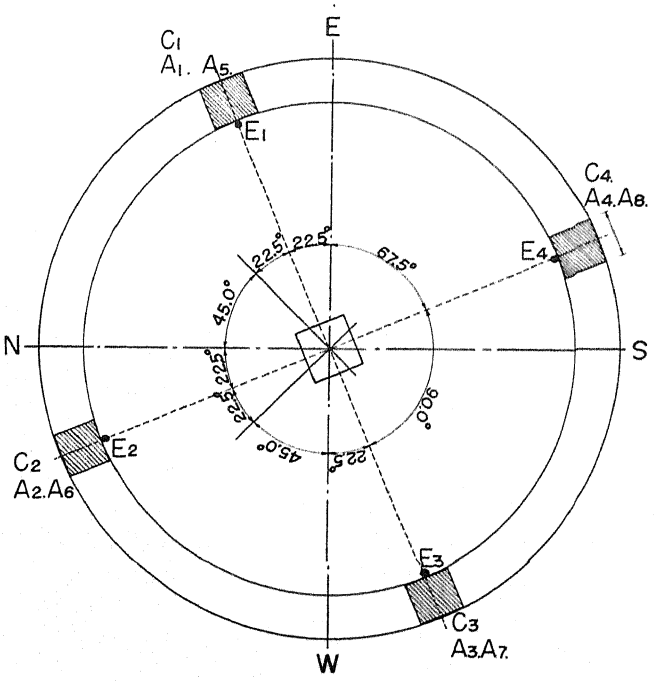
D_{1-2} DISPLACEMENT METER

E_0 MAIHAC-TYPE EARTH PRESSURE GAUGE

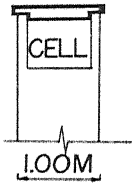
E_{1-8} CARLSON-TYPE EARTH PRESSURE GAUGE

W WATER PRESSURE GAUGE

FIG.5



4-CELLS FOR EQUIPMENT TOP OF THE CAISSON WALL



HATCHED AREAS ARE REQUIRED FOR CELLS TO INSTALL EQUIPMENT A_1-A_8 AND C_1-C_4 THEREIN.

FIG. 6

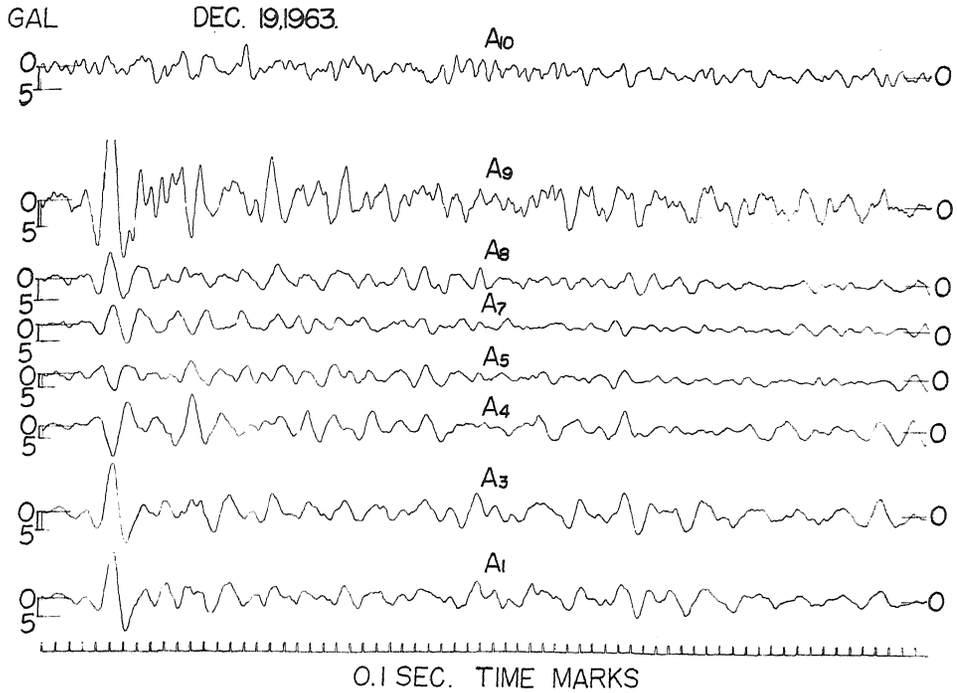

GAL
DEC. 19,1963.
A10
A9
A8
A7
A5
A4
A3
A1
0
5
0.1 SEC. TIME MARKS

FIG. 7

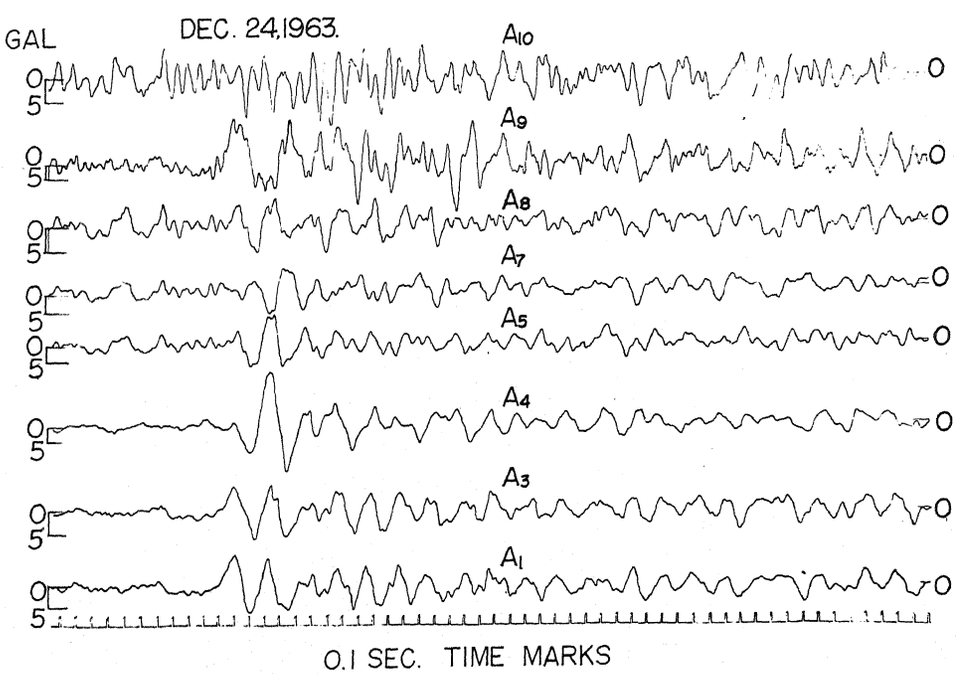

GAL
DEC. 24,1963.
A10
A9
A8
A7
A5
A4
A3
A1
0
5
0.1 SEC. TIME MARKS

FIG. 8

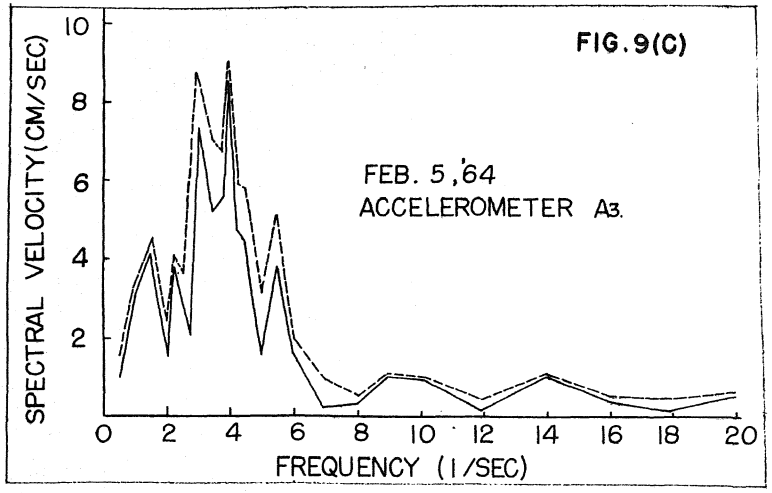


FIG. 9(C)
FEB. 5, '64
ACCELEROMETER A3.
SPECTRAL VELOCITY (CM/SEC)
FREQUENCY (1/SEC)

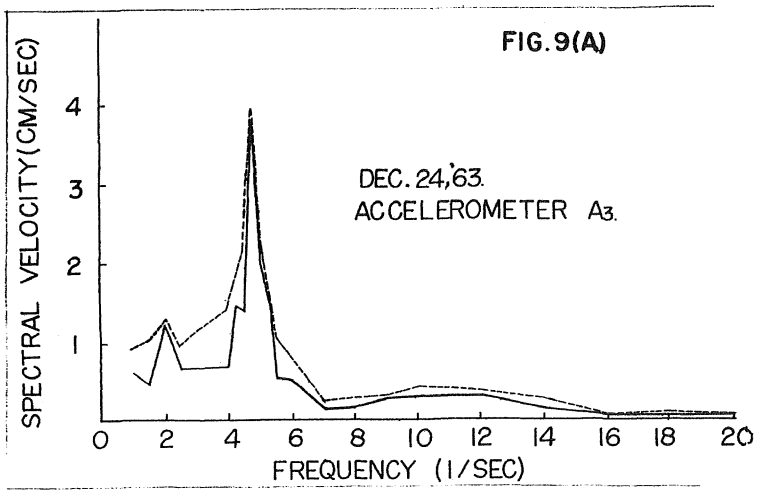


FIG. 9(A)
DEC. 24, '63.
ACCELEROMETER A3.
SPECTRAL VELOCITY (CM/SEC)
FREQUENCY (1/SEC)

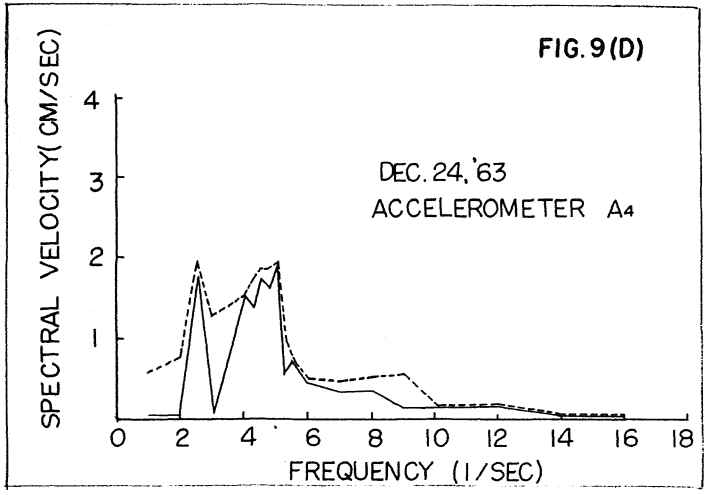


FIG. 9(D)
DEC. 24, '63
ACCELEROMETER A4
SPECTRAL VELOCITY (CM/SEC)
FREQUENCY (1/SEC)

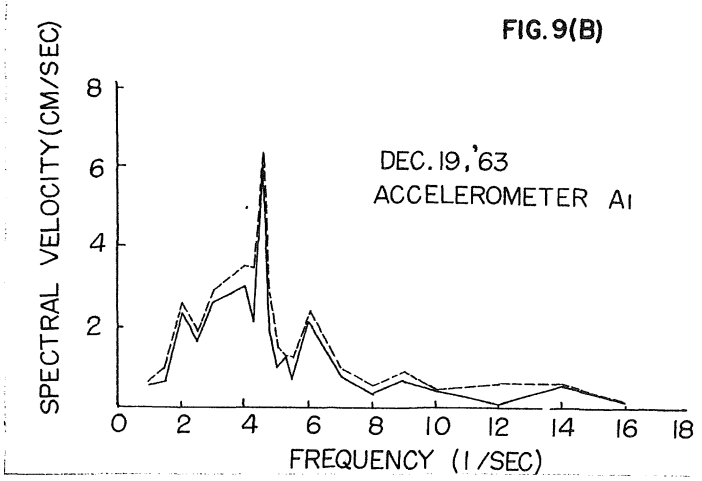


FIG. 9(B)
DEC. 19, '63
ACCELEROMETER A1
SPECTRAL VELOCITY (CM/SEC)
FREQUENCY (1/SEC)

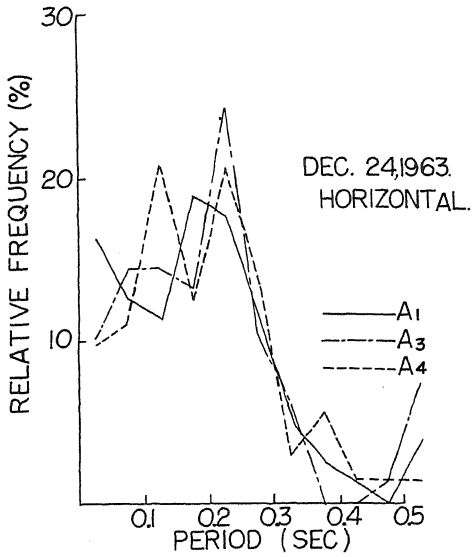


FIG.10(A)

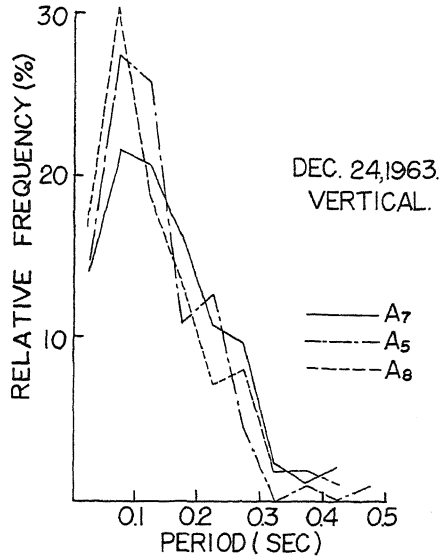


FIG.10(B)

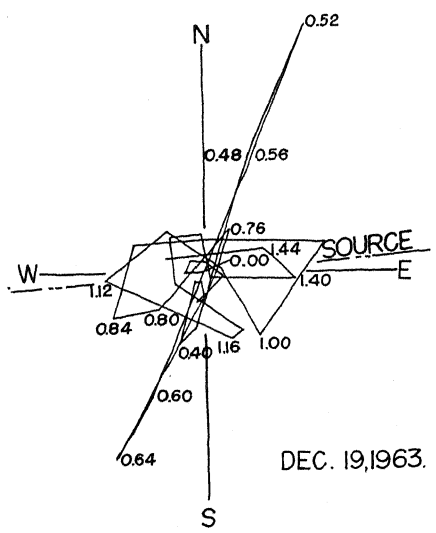


FIG.11(A)

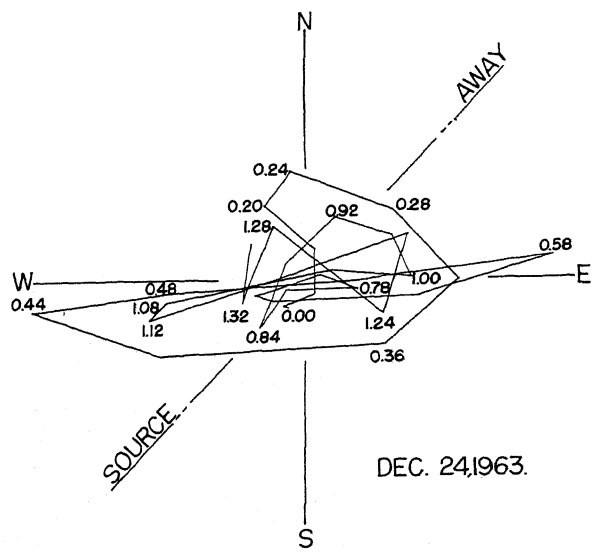
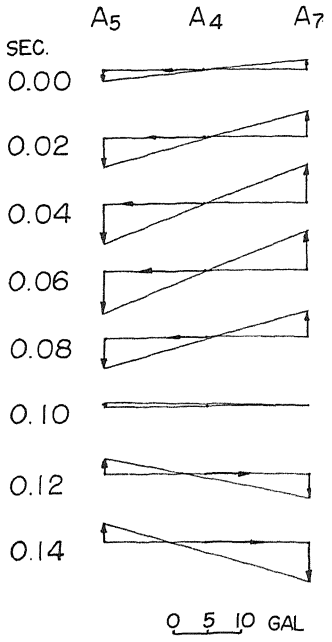
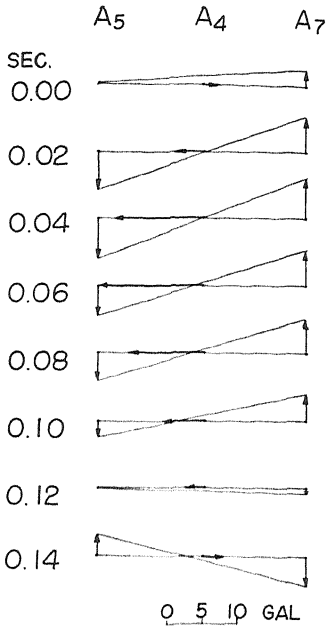


FIG.11(B)



DEC. 19, 1963

FIG. 12(A)



DEC. 24, 1963

FIG. 12(B)

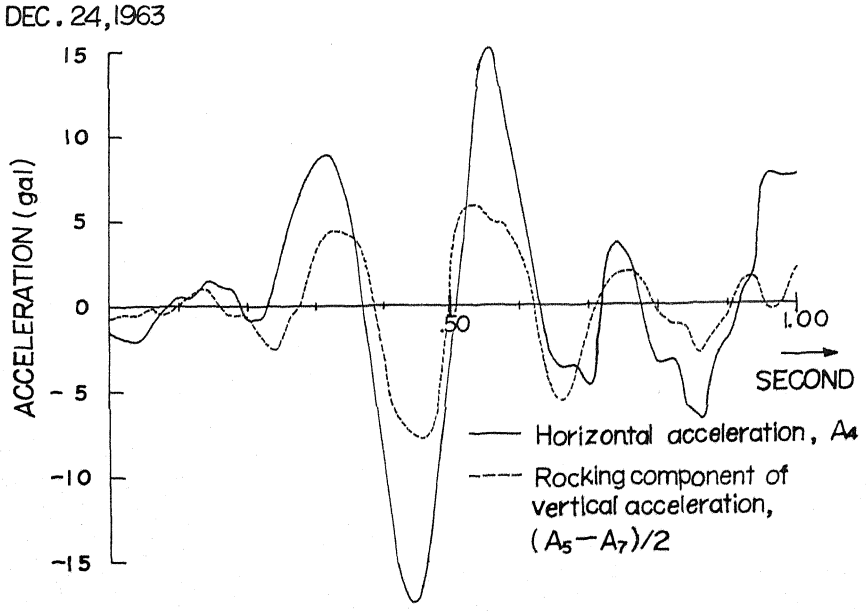


FIG. 13