EXPERIMENTAL AND ANALYTICAL STUDIES FOR A BWR NUCLEAR REACTOR BUILDING EVALUATION OF SOIL–STRUCTURE INTERACTION BEHAVIOUR

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SUMMARY

The purpose of this paper is to evaluate the spatial characteristics of dynamic properties, especially soil-structure interaction behaviour, or the BWR nuclear reactor building by experimental and analytical studies.

It is well known that the damping effects in soil-structure interaction are remarkable on the building with short periods by the dissipation of vibrational energy to the soil.

We reported an analytical method (SMiRT-1 Paper K 2/4) for estimating the damping effects the properties of which are characterized as follows:

1) The complex damping is used, because the so-called structural damping may be more suitable for estimating the damping effects of an elastic structure.
2) H. Tajimi's theory is used for estimating the dynamical soil-foundation stiffness with the dissipation of vibrational energy on the elastic half-space soil.

In this paper, an approximate explanation is presented in regard to the more developmental mathematical method for estimating the damping effects than the above-mentioned previous method, which is "Modes Superposition Method for Multi-Degrees of Freedom System" with the constant complex stiffness showing the structural damping effects and the dynamical soil-foundation stiffness approximated by the linear or quadratic functions of the eigenvalues. Next, an approximate explanation is presented in regard to the experimental results of the No. 1 reactor building (BWR) of Hamaoka Nuclear Power Station, The Chubu Electric Power Co., Ltd. The regression analyses of the experimental resonance curves by one degree system show that the critical damping ratio is larger than the 0.10 used in the design for the fundamental natural period.

It is attempted to simulate the experimental results by the above-mentioned method. The simulated model is a fourty-eight degrees of freedom spring mass system because of the eight masses for the eight floors including the base foundation and the six degrees of freedom for a mass.

Through these studies, it is concluded that the radiation damping effects are remarkable in the reactor building and the above-mentioned method is reasonable for estimating the damping effects in soil-structure interaction because of a good agreement with the experimental results.

1. Introduction

The most important problems for dynamical analysis are the selection of the suitable earthquake ground motion for design and the use of a suitable mathematical model according to the dynamical characteristics of structures. Especially, the damping mechanics of structures are the most complex problems in mechanics and its properties were unknown.

The damping effects of elastic structures may be considered as structural damping mechanics, of which the damping factors or logarithmic damping ratios are independent of frequencies of the disturbing force or the eigenvalues of structure. On the other hand, it is well known that the damping effects in soil-structure interaction is remarkable for a building with a short natural period and radiation damping mechanics is approximately similar to the viscous damping mechanics.

At the previous conference in 1971, one of the authors reported the analytical method estimating the above-mentioned damping effects. In this report, the authors report a more developmental mathematical method than the previous one, which is "Modes Supperposition Method to Multi-Degrees of Freedom System "with constant complex stiffness showing the structural damping effects and dynamical soil-foundation stiffness showing the damping effects by the dissipation of vibrational energy to the ground.

In 1974, the authors preformed a precise forced vibration test at the Hamaoka Nuclear Power Plant No. 1 (BWR-type) of the Chubu Electric Power Co., Ltd. and estimated the natural periods and damping ratios of the reactor buildings. The authors found that the damping effects for the dissipation of vibrational energy is remarkable at the reactor building as expected. The authors attemped to estimate the analytical damping effects by the above-mentioned method and compare them with the experimental results.

2. Concept of Complex Damping

The authors propose the simplest mathematical method by way of complex numbers for expressing the structural damping effects.

Thus, the spring constant can be expressed as:

$$k = k_o \, e^{i \, sgn \, w\phi}$$

where $sgn\, w$: $sgn\, w = 1$ for $w > 0$, $sgn\, w = 1$ for $w = 0$, $sgn\, w = -1$ for $w < 0$. (1)

The $sgn\, w$ is necessary to obtain a pair of conjugate values, when the response values in the time domain are transformed into those in the grequency domain by Fourier's Transformation.

The analytical method of the vibrational equation of motion for a system with a single degree of freedom is expressed in the complex numbers as follows:

$$m \ddot{x} + k_o \, e^{i \, sgn \, w\phi} \, x = P(t) \tag{2}$$

where, $\phi = 2 \sin^{-1} h$, h: damping ratio.

The eigenvalues of eq.(2) are obtained as follows according to $sgn\, w$:

$$w > 0 \quad \lambda = i \, w_o \, e^{i \, \phi/2} = -w_o h + i \, w_o\sqrt{1 - h^2}$$
$$w < 0 \quad \lambda = i \, w_o \, e^{-i \, \phi/2} = -w_o h - i \, w_o\sqrt{1 - h^2}$$

where $w_o = \sqrt{k/m}$. (3)

As the result of application of this method, it can be concluded that the solution of eq.(2) is perfectly indentical to the solution of equation of the Voigt Model with the same period and damping ratio in the cases of both free and forced vibrations.

3. Dynamical Soil-Foundation Stiffness

This problem has been developed by a number of researchers such as Dr.Toriumi[3], Drs. Kobori and Minai[4], Dr.Tajimi[5],[6] and the others since Drs.Sezawa and Kanai reported their studies in 1935 in Japan.

The displacement at the bottom of the foundations can be generally assumed to be separated in the following two displacements:

1) The displacement of free surface except for the foundations by ground motion,
2) The local displacement caused by the upper structure through the foundations.

Therefore, assuming the horizontal exciting force $Q = Q_o \ell^{i\omega t}$ transmitted in the soil, the local displacement also responding harmonically will be expressed by the following equation:

$$U = Q_o \ell^{i\omega t} / (k_{H1}(\omega) + i\, k_{H2}(\omega)) \tag{4}$$

Where, the complex variable $k_{H1}(\omega) + i\, k_{H2}(\omega)$ corresponding to the static spring constant is a complex function of the circular frequency ω of the exciting force, called the dynamical soil-foundation stiffness.

In this report, the dynamical soil-foundation stiffness is calculated by Dr.Tajimi's theory based on a solution of the propagation of seismic waves caused by point excitation on the surface of the elastic half-space medium.

Under vertical and horizontal exicitation, the displacement at an arbitrary point on the surface in the same direction is expressed respectively as the following approximated equations:

$$\text{Vertically:}\quad W|_{z=0} = \frac{P_o \ell^{i\omega t}}{2\pi G} \cdot \frac{1-\nu}{r}(f_{V1} + i f_{V2}) = W_s \ell^{i(\omega t - \Gamma_V \frac{\omega r}{V_s})}[1 + \alpha_V (\frac{\omega r}{V_s})^{\beta_V}] \tag{5}$$

$$\text{horizontally:}\quad U_x|_{z=0} = \frac{Q_o \ell^{i\omega t}}{4\pi G} \cdot \frac{1}{r}(f_{H1} + i f_{H2}) = U_s \ell^{i(\omega t - \Gamma_H \frac{\omega r}{V_s})}[1 - \alpha_H (\frac{\omega r}{V_s})^{\beta_H}] \tag{6}$$

where, W = vertical displacement, U = horizontal displacement

W_s, U_s = static displacement of each component

Γ_V, Γ_H, α_V, α_H, β_V, β_H = coefficient for approximation.

The above approximated equations are sufficiently applicable in the range of $\omega\gamma/V_s<2.0$. Using these expressions, the dynamic displacements of the foundations are derived on the assumption of the stress distribution on the contact surface by the following equations:

A = area of foundation,

$$\text{horizontally:}\quad k_H = \frac{Q_x}{\frac{1}{A}\iint u x dx, dy}, \quad \text{rotationally:}\quad k_R = \frac{M}{\Psi}, \quad \Psi = \frac{\iint_{A/2} W dxdy}{\iint_{A/2} X dxdy}. \tag{7}$$

These results give the vibrational impedances of the soil-foundation corresponding to the static soil coefficient. Fig.9 shows the calculated results in the case of Hamaoka Nuclear Power Plant No.1 Reactor Building.

In order to analyzed the eigenvalues and eigen vectors, the authors tried to approximate the dynamic soil-foundation stiffness with the linear or quadratic function of the exciting circular frequency as shown below:

$$K_R(\omega) + i\, K_I(\omega) = (k_{RO} + i\, \mathrm{sgn}\, \omega\, k_{I2}) + \lambda\, (k_{R1} + i\, \mathrm{sgn}\, \omega\, k_{I2}) + \lambda^2 (k_{R2} + i\, \mathrm{sgn}\, \omega\, k_{I2}) \tag{8}$$

where, λ = eigen values of vibrational system.

4. Modes Supperposition Method to System with Many Degrees of Freedom[7]

4.1 Equations of Motion for Soil-Structure System

When a structure is subjected to horizontal ground acceleration, we obtain the following equations including vertical, horizontal, rotational and torsional displacements for the system with many degrees of freedom.

$$[M]\{\ddot{x}\} + [K_R(\lambda) + i\ sgn\, w\, K_I(\lambda)]\{x\} = -\ddot{x}_0[M]\{C\} \tag{9}$$

where, $[M]$ = matrix of mass and moment of inertia of mass point

$[K]$ = complex stiffness matrix

$\{C\}$ = constant vector

In this report, the acceleration vector $\{\ddot{x}\}$ and displacement vector $\{x\}$ show $\ddot{u}$, $\ddot{v}$, $\ddot{w}$, $\ddot{O}y$, $\ddot{O}x$, $\ddot{O}z$ and u, v, w, Oy, Ox, Oz respectively, because the authors tried to set the six degrees of freedom on the individual floors as shown in Fig. 1.

Also, the mathematical model of the whole structure may be considered as the combined model of superstructure and soil-foundation so that the stiffness matrices are obtained by the superimposition of eq. (1) for the superstructure and eq. (8) for the soil-foundation.

So, the real and imaginary parts of stiffness matrix are shown as follows:

$$\begin{aligned} &\text{real part:} && [K_R(\lambda)] = [K_{R0}] + \lambda[K_{R1}] + \lambda^2[K_{R2}], \\ &\text{imaginary part:} && [K_I(\lambda)] = [K_{I0}] + \lambda[K_{I1}] + \lambda^2[K_{I2}]. \end{aligned} \tag{10}$$

4.2 Eigenvalue Analysis

The equation of motion eq. (9) can also be written in reduced form as follows:

$$\begin{bmatrix} O & M \\ M & O \end{bmatrix} \begin{Bmatrix} \ddot{x} \\ \dot{x} \end{Bmatrix} + \begin{bmatrix} -M & 0 \\ 0 & K_R(\lambda)+i\ sgn\, w\, K_I(\lambda) \end{bmatrix} \begin{Bmatrix} \dot{x} \\ x \end{Bmatrix} = -\ddot{x}_0 \begin{bmatrix} O & M \\ M & O \end{bmatrix} \begin{Bmatrix} C \\ O \end{Bmatrix}. \tag{11}$$

In order to obtain eigenvalues, the following characteristic equation is formed by substituting $\begin{Bmatrix} \dot{x} \\ x \end{Bmatrix} = e^{\lambda t} \begin{Bmatrix} \dot{X} \\ X \end{Bmatrix}$, $\begin{Bmatrix} \ddot{x} \\ \dot{x} \end{Bmatrix} = \lambda e^{\lambda t} \begin{Bmatrix} \dot{X} \\ X \end{Bmatrix}$ and $\{\dot{X}\} = \lambda\{X\}$ into the equations of motion.

When $sgn\ w = 1$,
$$\begin{bmatrix} M^{*-1}(K_{R1}+i\,K_{I1}) + \lambda I, & M^{*-1}(K_{R0}+i\,K_{I0}) \\ -I, & \lambda I \end{bmatrix} \begin{Bmatrix} \lambda X \\ X \end{Bmatrix} = 0, \tag{12}$$

and when $sgn\ w = -1$,
$$\begin{bmatrix} \overline{M}^{*-1}(K_{R1}-i\,K_{I1}) + \bar{\lambda} I, & \overline{M}^{*-1}(K_{R0}-i\,K_{I0}) \\ -I, & \lambda I \end{bmatrix} \begin{Bmatrix} \overline{\lambda X} \\ \overline{X} \end{Bmatrix} = 0, \tag{13}$$

where $[M^*]^{-1} = [M+K_{R2}+i\,K_{I2}]^{-1}$, $[\overline{M}^*]^{-1} = [K+K_{R2}-i\,K_{I2}]^{-1}$.

The values of eigenvalues λ for which eq. (12) and (13) have non-trivial solutions occur as complex conjugate pairs, λ for $sgn\ w = 1$ and $\bar{\lambda}$ for $sgn\ w = -1$.

The eigenvectors also occur as complex conjugate paris $\{X\}$ and $\{\overline{X}\}$ for λ and $\bar{\lambda}$ respectively.

According to the concept of complex damping, the j-th circular frequency wj and damping ratio hj are defined as follows:

$$\lambda j = -wjhj + iwj\sqrt{1-hj^2} \quad \text{for } sgn\ w = 1, \tag{14}$$

$$\overline{\lambda}j = -wjhj - iwj\sqrt{1-hj^2} \qquad sgn\ w = -1. \tag{15}$$

Hence, we must reject the eigenvalues and vectors which are not suitable for the term of $sgn\, w = 1$ or $sgn\, w = -1$.

4.3 Modes Superposition Method

If the $2n$ eigenvalues of a system with n degrees of freedom eq. (9) are all distinct, the $2n$ eigenvectors form a linearly independent set and any configuration of the system can be expressed as a linear combination of the conjugate eigenvectors.

That is, we can write

$$\begin{Bmatrix} \ddot{x} \\ \dot{x} \end{Bmatrix} = \sum_{j=1}^{n} \left[\dot{q}j \begin{Bmatrix} \lambda j Xj \\ Xj \end{Bmatrix} + \dot{\bar{q}}j \begin{Bmatrix} \bar{\lambda} j \bar{X}j \\ \bar{X}j \end{Bmatrix} \right] = [Z, \bar{Z}] \begin{Bmatrix} \dot{q} \\ \dot{\bar{q}} \end{Bmatrix} , \tag{16}$$

$$\begin{Bmatrix} \dot{x} \\ x \end{Bmatrix} = \sum_{j=1}^{n} \left[qj \begin{Bmatrix} \lambda j Xj \\ Xj \end{Bmatrix} + \bar{q}j \begin{Bmatrix} \bar{\lambda} j \bar{X}j \\ \bar{X}j \end{Bmatrix} \right] = [Z, \bar{Z}] \begin{Bmatrix} q \\ \bar{q} \end{Bmatrix} , \tag{17}$$

$$\begin{Bmatrix} C \\ O \end{Bmatrix} = \sum_{j=1}^{n} \left[\beta j \begin{Bmatrix} Xj Xj \\ Xj \end{Bmatrix} + \bar{\beta} j \begin{Bmatrix} \bar{X}j \bar{X}j \\ \bar{X}j \end{Bmatrix} \right] = [Z, \bar{Z}] \begin{Bmatrix} \beta \\ \bar{\beta} \end{Bmatrix} , \tag{18}$$

where $qj,\ \bar{q}j = j^{-th}$ generalized coordinates,

$\beta j,\ \beta j = j^{-th}$ modal participation factors,

$[Z, \bar{Z}]$ = modal matrices with suitable eigenvalues for the term of $sgn\ w$

By substituting eqs. (16) ∿ (18) into eq. (11) we obtain

$$[D][Z, \bar{Z}] \begin{Bmatrix} \dot{q} \\ \dot{\bar{q}} \end{Bmatrix} + [E^*][Z, \bar{Z}] \begin{Bmatrix} q \\ \bar{q} \end{Bmatrix} = -\ddot{x}_0 [D] \begin{Bmatrix} C \\ O \end{Bmatrix} , \tag{19}$$

where $[D] = \begin{bmatrix} O & M \\ M & O \end{bmatrix}$ and $[E^*] = \begin{bmatrix} -M & O \\ O & K_R(\lambda) + i\ sgn\, w\, K_I(\lambda) \end{bmatrix}$.

Premultiplying by $\begin{bmatrix} Y^T \\ \bar{Y}^T \end{bmatrix}$ gives

$$\begin{bmatrix} Y^T \\ \bar{Y}^T \end{bmatrix} [D][Z, \bar{Z}] \begin{Bmatrix} \dot{q} \\ \dot{\bar{q}} \end{Bmatrix} + \begin{bmatrix} Y^T \\ \bar{Y}^T \end{bmatrix} [E^*][Z, \bar{Z}] \begin{Bmatrix} q \\ \bar{q} \end{Bmatrix} = -\ddot{x}_0 \begin{bmatrix} Y \\ \bar{Y}^T \end{bmatrix} [D] \begin{Bmatrix} C \\ O \end{Bmatrix} , \tag{20}$$

where, $[Y, \bar{Y}]$ = modal matrices with non-suitable eigenvalues for the term of $sgn\, w$.

In view of the characteristic equation, we can write:

$$\begin{aligned} [Y^T E Z] &= -[Y^T D Z][\lambda] , \quad [\bar{Y}^T E Z] = -[\bar{Y}^T D Z][\lambda] \\ [Y^T \bar{E} \bar{Z}] &= -[Y^T D \bar{Z}][\lambda] , \quad [\bar{Y}^T \bar{E} \bar{Z}] = -[\bar{Y}^T D \bar{Z}][\bar{\lambda}] , \end{aligned} \tag{21}$$

where $[E] = \begin{bmatrix} -M, & O \\ O, & K_R(\lambda) + i K_I(\lambda) \end{bmatrix}$, $\quad E = \begin{bmatrix} -M, & O \\ O, & K_R(\lambda) - i K_I(\lambda) \end{bmatrix}$.

Eq. (21) then becomes

$$\begin{bmatrix} Y^T DZ, & Y^T D\bar{Z} \\ \bar{Y}^T DZ, & \bar{Y}^T D\bar{Z} \end{bmatrix} \left[\begin{Bmatrix} \dot{q} \\ \dot{\bar{q}} \end{Bmatrix} - \begin{bmatrix} \lambda & O \\ O & \bar{\lambda} \end{bmatrix} \begin{Bmatrix} q \\ \bar{q} \end{Bmatrix} \right] = -\ddot{x}_0 \begin{bmatrix} Y^T DZ, & Y^T D\bar{Z} \\ \bar{Y}^T DZ, & \bar{Y}^T D\bar{Z} \end{bmatrix} \begin{Bmatrix} \beta \\ \bar{\beta} \end{Bmatrix} , \tag{22}$$

which are reduced to

$$\dot{q}j - \lambda j\, qj = -\ddot{x}_0 \beta j, \quad \dot{\bar{q}}j - \bar{\lambda} j \bar{q}j = -\ddot{x}_0 \bar{\beta} j, \quad (j=1 \sim n), \tag{23}$$

$$\begin{Bmatrix} \beta \\ \bar{\beta} \end{Bmatrix} = \begin{bmatrix} Y^T DZ, & Y^T D\bar{Z} \\ \bar{Y}^T DZ, & \bar{Y}^T D\bar{Z} \end{bmatrix}^{-1} \begin{bmatrix} Y^T DF \\ \bar{Y}^T DF \end{bmatrix} , \text{ where } \{F\} = \begin{Bmatrix} C \\ O \end{Bmatrix} . \tag{24}$$

The generalized coordinates $qj,\ \bar{q}j$ and the modal participation factors $\beta j,\ \bar{\beta} j$ can be obtained as complex conjugate paris from the eqs. (23) and (24).

5. Experimental Study

5.1 Description of Building

This nuclear power plant is a boiling water reactor (540MW̄) and composed of the combined reactor building and the turbine building as shown in Figs. 1 and 2.

Both are reinforced concrete structures and located on separate foundation mats. The former combined reactor building includes the reactor building, the control room and the radioactive waste disposal building on 64 meters square and 4 meters deep foundation mat located directly on a firm layer of mudstone, which is extremely effective in protecting the building from the overturning moment for 0.3G earthquake ground motion. The transverse wave velocity of the supported layer was measured as 800 ∿ 1,000 m/sec by the elastic wave method. This building is also about 62 meters in height from the bottom of the foundation mat as shown in Fig. 2.

5.2 Description of Vibration Exciters and Instrumentation

In March 1974, the authors carried out forced vibration tests at the combined reactor building to obtain the dynamical properties of the building. The two vibration exciters used were designed and developed in Japan and were of the BCS-A200 type. The exciters employ counterrotating weights, thus applying intertial forces to the structure in one direction only, the forces applied are proportional to the square of the frequency and they vary sinusoidally with time. The range of frequencies that can be excited is 0.2 cps to 20.0 cps, but from 0.2 cps to 8.0 cps in case of double exciting. The speed control is extremely accurate since the speed can be controlled to an accuracy of about 0.5% and the phase control is less than 15 degrees in the case of double exciting. The vibration exciters were located on the fifth floor as shown in Fig. 3. Table I shows the exciting moments with the range of applied exciting frequencies.

Lateral motions of the building were measured by pick-ups with natural periods of 1.0 sec and digital values of displacement were obtained from gain-phase meters through amplifiers. Measurements were made using eight pairs of pick-ups located on the 2nd basement, 1st, 2nd, 3rd, 4th, 5th floors, crane girders and roof as shown Fig. 3.

The building was excited in the east-west and north-south directions. The machine speed was increased stepwise by 0.10 to 0.2 cps and the lateral motions of the floors were recorded as the amplitude and phase of displacement at each frequency after the steady state vibration of the building was evident.

5.3 Experimental Results

<u>Damping</u> The equivalent viscous damping can be determined directly from the measured displacement at resonance. The authors attempted to estimate the damping effects by regression analysis of the experimental resonance curves.[8] The equilibrium equation of motion of the one-degree system is shown as below with a sinusoidally exciting force:

$$m\ddot{q} + c\dot{q} + kq = m_0 r\, w^2 e^{iwt} \tag{25}$$

where m = mass, c = damping coefficient, k = spring constant,

m_0, r_0 = mass and arm of counter-rotating weights, w = exciting circular frequency.

The particular solution of eq. (3.1) is shown as follows:

$$q = \frac{m_0 r}{m} \cdot \frac{(w/w_0)^2}{\sqrt{\{1-(w/w_0)^2\}^2 + 4h_0^2 (w/w_0)^2}}\, e^{i(wt-\psi)}, \quad \psi = \tan^{-1} \frac{2h_0 (w/w_0)}{1-(w/w_0)^2} \tag{26}$$

where w_0 = natural circular frequency, h_0 = critical damping ratio.

Thus, the resonance amplitude Qd in steady state vibration will be shown as follows:

$$Qd = \frac{m_0 r}{m} \cdot \frac{(f/f_0)^2}{\sqrt{\{1-(f/f_0)^2\}^2 + 4h_0^2 (f/f_0)^2}}, \quad \text{where } f_0 = w_0/2\pi,\ f = w/2\pi. \tag{27}$$

Eq. (27) also will be written as follows: $y = cx/\sqrt{(a - x)^2 + abx}$. (28)

where $a = f_0^2$, $b = 4h_0^2$, $c = 1/m$, $x = f^2$ and $y = Qd/m_0 r$.

Eq. (28) shows that it is useful to decide the most suitable coefficients a, b and c for the groups of observed (x, y), i.e. (x_1, y_1), (x_2, y_2), ... (x_n, y_n) by regression analysis.

In this report, the discussion of damping effects is limited to the lowest mode alone in which the most remarkable effects of energy dissipation is expected from soil-structure interaction. Table II shows the natural periods and damping ratios determined by the abovementioned method. Table II shows the values of critical damping are more than 22.5% and 17.5% in the east-west and north-south directions respectively, which are certainly more than the 10% used in the design of the building.

Resonance curves

In order to estimate the exact properties of the building, the authors attempted to separate the coupled resonance motion of each floor into three components, i.e., lateral modes in parallel with the east-west and north-south directions and torsional mode, with regard to the center of gravity of each floor calculated in the analytical study. The results of some floors are shown in Figs. 5 and 6 in this report, although the measurements were made at each floor as shown in Fig. 3. The moment of vibration exciters also is transformed to a constant value of 10 Kgm in the results shown in these figures. Figs. 5 and 6 show that the damping effects from energy dissipation are remarkable in the lowest mode and this is the reason why the authors tried to estimate the damping effects by regression analysis.

6. Analytical Study

The authors tried to estimate the damping effects in the lowest mode by the analytical method described in sections 2 ∿ 4. The main resisting elements of the combined reactor building are assumed to be the reinforced concrete walls alone. The assumed mathematical model in Table III is a spring-mass system with fourty eight degrees of freedom by reason of six components for individual mass points. Table III shows the assumed weight, moment of inertia and center of gravity of individual masses.

The stiffness of the structure is calculated by considering the bending and shearing deformations of the thin walls as assumed by Dr. Kuranishi. The dynamical soil-foundation stiffness is assumed to be the complex simple spring and calculated by Dr. Tajimi's Theory except for the torsional spring extented by Drs. Kobori and Minai's results.[4]

It is necessary to assume the stress distribution on the contact surface between the soil and foundation to calculate the dynamical soil-foundation stiffness.

Fig. 7 shows the static vertical stress distribution observed under construction.

Although it is not expected that the static stress distribution is equal to the dynamical stress distribution, the authors assumed the former is almost the same as the latter to calculate the dynamical soil-foundation stiffness. Some of calculated results are shown in Figs. 8 and 9. The physical coefficients of concrete and soil are shown in Table IV. Table V shows the calculated frequencies and damping ratios with the superior components of modes. It was found that the damping ratios in the lowest modes are 17.7% and 19.8% in the east-west and north-south directions respectively, which are nearly the same as the experimental results.

7. Conclusion

In this report, the authors report the analytical method estimating the structural damping effects and the radiational damping effects as stated below:

1) Concept of Complex Damping, 2) Dynamical Soil-Foundation Stiffness, 3) Modes Super-

position Method to Multi-Degrees of Freedom Systems.

The authors also reported the experimental results of the forced vibration tests of Hamaoka Nuclear Power Plant No. 1 Reactor Building, which shows the damping effects by the energy dissipation is remarkable in the lowest modes. The authors finally showed the analytical results which are almost the same as the experimental results. Through these studies the authors found that the damping effects by the energy dissipation in the building with short natural period can be estimated by the author's method.

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TABLE I

APPLIED EXCITING FREQUENCIES AND MOMENTS

Frequency (c/s)	Moment (Kgm)	Exciter
3 ∿ 6	40	double
3 ∿ 8	20	double
6 ∿ 14	4	single
12 ∿ 20	2	single

TABLE IV

PHYSICAL COEFFICIENTS OF CONCRETE AND SOIL

		Concrete	Soil
Velocity of Shear Wave	(m/s)	–	800
Young's Modulus	(t/cm^2)	210	–
Density	(t/m^3)	2.4	2.01
Poisson's Ratio		1/6	1/3
Damping Ratio	(%)	2	0

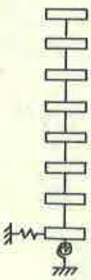
TABLE II

EXPERIMENTAL VALUES OF LOWEST TRANSLATIONAL MODE

Exciting Direction		East-West			North-South	
Range of Frequency		2.95∿5.95 c/s			2.95∿5.45 c/s	
Location		f_0 (c/s)	h_0 (%)	Location	f_0 (c/s)	h_0 (%)
Roof	N	5.0	23.5	E	4.9	23.9
	S	4.8	24.6	W	4.9	17.5
5th Floor	N	4.9	23.4	E	4.8	20.6
	S	4.7	22.5	W	4.8	19.4
2nd Floor	N	4.8	23.7	E	5.0	20.7
	S	4.8	23.8	W	5.0	20.2
B2 Floor	N	4.7	28.3	E	5.7	31.5
	S	4.8	26.7	W	5.1	23.6

TABLE III

ASSUMED MATHEMATICAL MODEL AND DIMENSION

Floor	Model	Height*1 (m)	Weight (t)	Moment of Inertia x10^7 ($t.cm.S^2$) E-W	N-S	Torsion	Center of Gravity*2 (m) E-W	N-S
R		58.7	1,048	0.271	0.204	0.470	0.0	-5.06
C		49.1	1,510	0.396	0.314	0.696	0.0	-5.11
5		42.4	4,826	0.946	0.630	1.550	-2.26	-3.90
4		34.7	8,265	1.804	1.516	3.254	0.24	2.14
3		28.9	12,453	3.442	2.767	6.129	-2.66	3.00
2		21.2	20,118	6.938	6.367	13.134	-1.86	2.31
1		12.7	32,139	13.018	12.632	25.111	-0.12	0.31
B2		0.0	63,731	22.183	23.201	43.706	-0.78	0.35

*1 Zero Level is set on Top of Foundation mat.

*2 Center of Coordinate is set at Center of Reactor Vessel.

TABLE V

ANALYTICAL VALUES OF LOWEST TRANSLATIONAL AND TORSIONAL MODES

Order	f_0 (c/s)	h_0 (%)	Direction of Superior Mode
1	4.95	17.7	East - West
2	5.22	19.8	North - South
3	7.31	14.5	Torsion

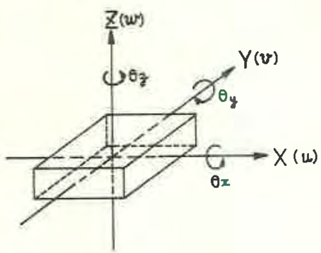


FIG. 1 ASSUMED COORDINATES AND DEGREES OF FREEDOM

FIG. 2 GENERAL VIEW, LOOKING SOUTHWEST

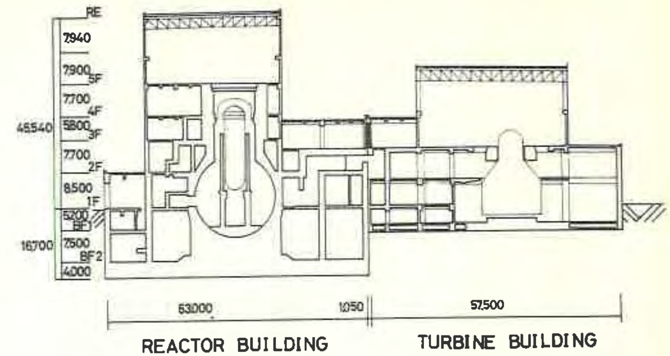


FIG. 3 SECTIONS OF REACTOR AND TURBINE BUILDINGS

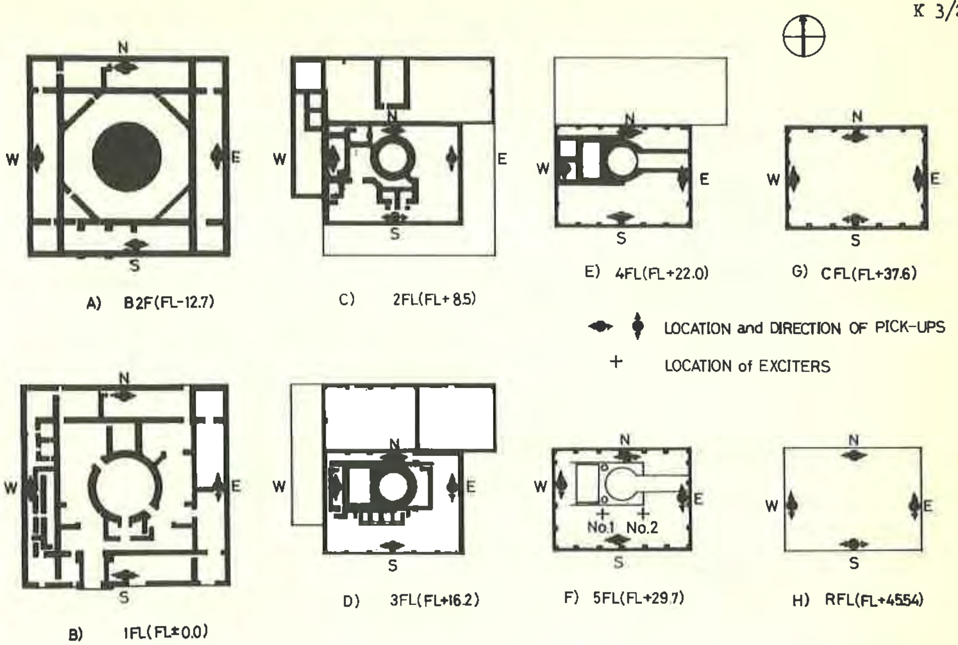


FIG. 4 POSITION OF EXCITERS AND PICK-UPS

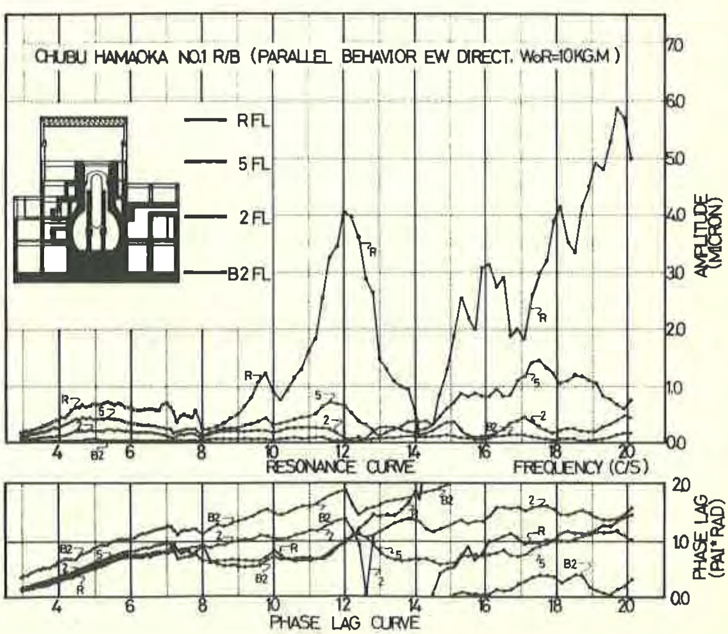


FIG. 5 TRANSLATIONAL MODES, EAST-WEST DIRECTION

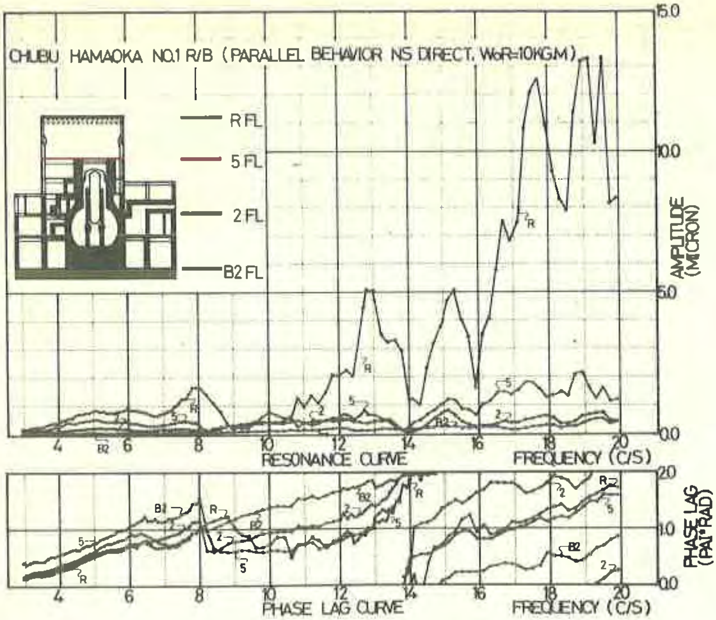


FIG. 6 TRANSLATIONAL MODES, NORTH-SOUTH DIRECTION

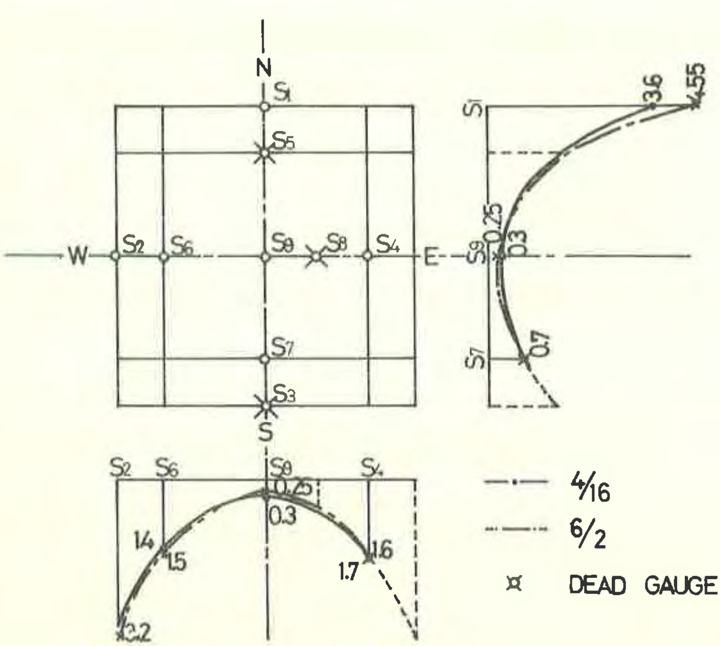


FIG. 7 OBSERVED STATIC VERTICAL STRESS DISTRIBUTION

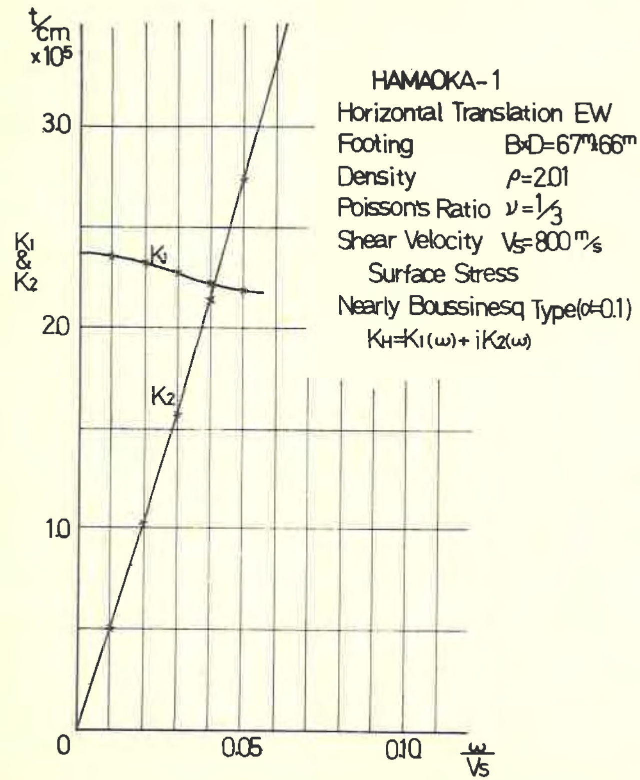


FIG. 8 DYNAMICAL SOIL-FOUNDATION STIFFNESS, HORIZONTAL

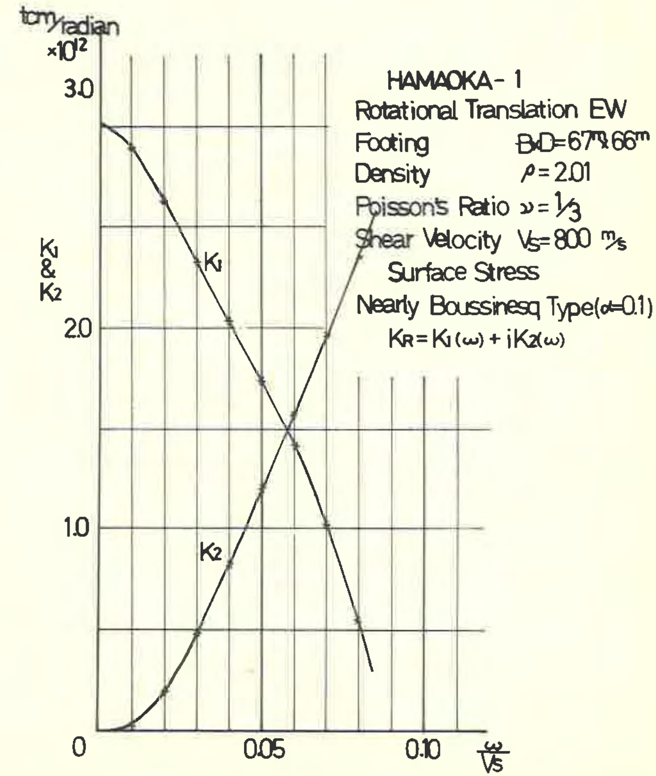


FIG. 9 DYNAMICAL SOIL-FOUNDATION STIFFNESS, ROTATIONAL