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WAVE AND EARTHQUAKE RESPONSE OF OFFSHORE STRUCTURES WITH SOIL-STRUCTURE INTERACTION

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Dynamic response of offshore structures to random sea waves and strong earthquake motions is investigated. Sea waves are modelled by Bretschneider's wave energy spectrum and ground motions are represented by Kanai's power spectrum. Governing equation of motion is obtained by the substructure method. Response analysis is carried out using frequency-domain random vibration approach. Wave responses are found to be generally larger whereas seismic responses are smaller when soil-structure interaction effects are considered. Reliability studies show that earthquake loadings provide comparable results to those of wave loadings, but the latter have more significant effects on response evaluations because of the longer duration time.

Keywords: offshore structures, dynamic response, soil-structure interaction, sea waves and earthquake motions

1. INTRODUCTION

A realistic design method in any offshore construction project should incorporate the dynamics of sea waves in a probabilistic sense, since they are random in nature. The nondeterministic analysis methods for offshore structures have been proposed by Foster[1], and Malhotra and Penzien[2]. For a typical offshore structure located in a seismically active region, earthquake loading should also be considered in the design. Earthquake motions are generated through numerous random phenomena and are essentially random in nature. Several methods[3),4)] are available for the seismic response analysis of structures on land. But these methods can not be directly applied to offshore structures due to the presence of surrounding water and sea waves. Dynamic analysis is complicated due to the fact that hydrodynamic forces are coupled to the dynamic response. Penzien, et al.[5] presented a stochastic method of analysis for rigidly supported offshore towers subjected to random sea waves and strong motion earthquakes. They observed that the hydrodynamic drag effects on the seismic response become important with increasing tower period or water depth. Seismic response of offshore structures considering soil-structure interaction has been studied by Takemiya, et al.[6]. They concluded that the internal forces are halved if soil-structure interaction is not included. However, in their analysis, the hydrodynamic forces due to sea waves were not included.

In this study, dynamic response of offshore structures subjected to random sea waves and strong motion

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earthquakes is investigated. Emphasis is placed on the evaluation of dynamic soil-structure interaction effects. The structure is discretized using the finite element method. The governing equation of motion is derived, independently for the sea wave loading case and the earthquake loading case, using the substructure method. Response analysis is carried out using the mode superposition method and the frequency-domain random vibration approach. Reliability of the offshore structure for typical sea states and strong motion earthquakes is also examined. The effects of soil-structure interaction on the responses are discussed.

2. EQUATION OF MOTION

The offshore structures can be discretized by lumping masses at selected nodal points and using the resultant forces over the corresponding projected areas and volumes. Fig. 1 shows the elevation of a tower model. The equation of motion is derived by substructure method[6),7)] in which the structure-pile-soil system is hypothetically divided into two substructures : the structure and the pile-soil foundation. The equation of motion is expressed in the general form as

$$\begin{bmatrix}[M_{aa}] & [M_{ab}]\\ [M_{ba}] & [M_{bb}]\end{bmatrix}\begin{Bmatrix}\{\ddot{u}_a\}\\ \{\ddot{u}_b\}\end{Bmatrix}+\begin{bmatrix}[C_{aa}] & [C_{ab}]\\ [C_{ba}] & [C_{bb}]\end{bmatrix}\begin{Bmatrix}\{\dot{u}_a\}\\ \{\dot{u}_b\}\end{Bmatrix}+\begin{bmatrix}[K_{aa}] & [K_{ab}]\\ [K_{ba}] & [K_{bb}]\end{bmatrix}\begin{Bmatrix}\{u_a\}\\ \{u_b\}\end{Bmatrix}=\begin{Bmatrix}\{F_a\}\\ \{F_b\}\end{Bmatrix} \quad \cdots\cdots (1)$$

in which the suffices a and b denote the unrestrained nodal point and the restrained nodal point at the base respectively, $[M]$ is the lumped mass matrix, $[C]$ is the structural damping matrix, $[K]$ is the stiffness matrix, $\{F\}$ is the external force vector, and $\{u\}$ is the displacement vector. $[C]$ is assumed to be small and proportional to the stiffness matrices of the structure. $(\cdot)$ indicates the derivative of the variable with respect to time t.

The equation of motion for the pile-soil foundation is

$$[M_p]\{\ddot{u}_p\}+[C_p]\{\dot{u}_p\}+[K_p]\{u_p\}=\{F_p\} \cdots\cdots (2)$$

in which $[M_p]$ is the mass matrix, $[C_p]$ is the damping matrix, $[K_p]$ is the stiffness matrix of the pile-soil foundation and $\{F_p\}$ is the interacting force vector from the superstructure on the foundation. The dynamic stiffness coefficients of the pile-soil foundation are interpreted as a generalized spring-dashpot system (Fig. 2). They are assumed to be independent of the excitation frequency, and are computed[8)] for lateral and rotational modes of vibration of the piles embedded in soft uniform soils. The equation of motion for the structure-pile-soil system is obtained by substructure method by combining eqs. (1) and (2) and by satisfying the compatibility conditions of displacements and the equilibrium conditions of forces at the base nodes.

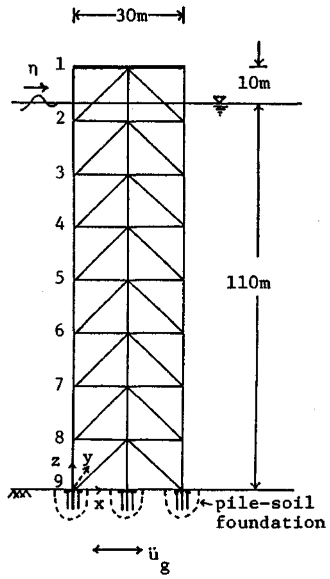

Fig. 1 Analytical model of structure-pile-soil system.

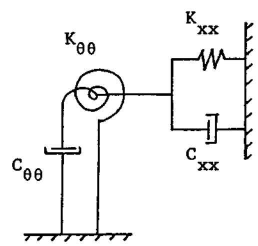

Fig. 2 Modelling of pile-soil foundation.

3. DYNAMIC RESPONSE TO RANDOM SEA WAVES

In the following development, linear wave theory is assumed. Vertical force and interference from other legs are neglected. Considering the relative motion between the waves and the structure, the wave force vector $\{F_a\}$ is expressed using the Morison equation. The nonlinear relative-velocity squared drag term in this equation is replaced by an equivalent linearized drag term in a classical manner[2),9)] by assuming Gaussian random process for the relative velocity distribution. The linearized Morison equation has the form

$$\{F_a\}=[C_M]\{\dot{V}_{oa}\}-[C_A]\{\ddot{u}_a\}+[C_D]\{V_{oa}-\dot{u}_a\} \quad (3)$$

where

$$[C_M]=[\diagdown\ \rho C_m V \diagdown],\ [C_A]=[\diagdown\ \rho(C_m-1)V \diagdown],\ [C_D]=[\diagdown\ \rho C_d A\sqrt{\frac{8}{\pi}}\,\sigma_r \diagdown],\ r=V_{oa}-\dot{u}_a$$

in which ρ is the mass density of water, V is the enclosed volume with respect to flow, A is the area projected in the direction of flow, C_m is the inertia coefficient, C_d is the drag coefficient, $\{V_{oa}\}$ is the water particle velocity vector and $\{\dot{V}_{oa}\}$ is the water particle acceleration vector at the undeflected structure coordinate locations, and σ_r is the rms value of the relative velocity which is obtained by an iterative procedure.

On the other hand, the compatibility condition of displacement at the base nodal points is

$$\{u_b\}=[G]\{u_p\} \quad (4)$$

where

$$[G]=\begin{bmatrix}\begin{bmatrix}1 & Z_c\\ 0 & 1\end{bmatrix} & & \\ & \ddots & \\ & & \cdot\end{bmatrix}$$

and the equilibrium equation of forces at each of the base nodal point is

$$[G]^T\{F_b\}+\{F_p\}=0 \quad (5)$$

in which Z_c is the depth of footing connecting the base of the tower with the pile head.

Now using Eqs. (3), (4) and (5), the equation of motion for the structure-pile-soil system subjected to sea wave loading becomes

$$\begin{bmatrix}[I] & [\tilde{M}_{ap}]\\ [\tilde{M}_{pa}] & [\tilde{M}_p]\end{bmatrix}\begin{Bmatrix}\{\ddot{q}\}\\ \{\ddot{u}_p\}\end{Bmatrix}+\begin{bmatrix}[2\beta_{fj}\omega_{fj}] & [\tilde{C}_{ap}]\\ [\tilde{C}_{pa}] & [\tilde{C}_p]\end{bmatrix}\begin{Bmatrix}\{\dot{q}\}\\ \{\dot{u}_p\}\end{Bmatrix}+\begin{bmatrix}[\omega_{fj}^2] & 0\\ 0 & [\tilde{K}_p]\end{bmatrix}\begin{Bmatrix}\{q\}\\ \{u_p\}\end{Bmatrix}=\begin{bmatrix}[P_a]\\ [P_b]\end{bmatrix}\begin{Bmatrix}\{\dot{V}_{oa}\}\\ \{V_{oa}\}\end{Bmatrix} \quad (6)$$

where

$$\begin{bmatrix}[P_a]\\ [P_d]\end{bmatrix}=\begin{bmatrix}[\Phi]^T[C_M] & [\Phi]^T[C_D]\\ [G]^T[L]^T[C_M] & [G]^T[L]^T[C_D]\end{bmatrix},$$

$\{u_a^c\}=[\Phi]\{q\}$, $[\tilde{C}_{pa}]=[G]^T[L]^T[\tilde{C}_{aa}][\Phi]$

$[\tilde{M}_{ap}]=[\Phi]^T[\tilde{M}_{aa}][L][G]$, $[\tilde{C}_p]=[G]^T[[C_{bb}]+[L]^T[C_D][L]][G]$

$[\tilde{M}_{pa}]=[G]^T[L]^T[\tilde{M}_{aa}][\Phi]$, $[\tilde{K}_p]=[K]_p+[G]^T[K_{bb}][G]$

$[\tilde{M}_p]=[M_p]+[G]^T[[L]^T[\tilde{M}_{aa}][L]+[M_{bb}]][G]$, $[\tilde{M}_{aa}]=[M_{aa}]+[C_A]$

$[\tilde{C}_{ap}]=[\Phi]^T[C_D][L][G]$, $[\tilde{C}_{aa}]=[C_{aa}]+[C_D]$

in which $[I]$ is the unit matrix, $[L]$ is the quasi-static transformation matrix, $[\Phi]$ is the undamped eigenvector (mode shape), $[\omega_{fj}^2]$ is the corresponding eigenvalue (square of the circular frequency) for the jth mode of the structure with rigidly supported base condition, β_{fj} is the damping ratio which includes both the structural damping and the hydrodynamic damping, $\{u_a^c\}$ is the dynamic displacement of the structure for the rigidly supported condition, $\{q\}$ is the modal displacement vector for dynamic displacements and the superscript T denotes the transpose of a matrix. Eq. (6) contains nonproportional damping matrix and hence complex eigenvalue analysis must be carried out for exact response analysis. However, when the natural frequencies of the structure are well-separated, it has been observed[10)] that the modal coupling effects due to off-diagonal terms are negligible. The present structure-pile-soil system has well-separated natural frequencies for first few vibration modes, and simplifying approximation can be made by considering only the diagonal terms of the damping matrix.

The response quantities in Eq. (6) can be expressed as a linear combination of generalized coordinates

$\{y\}$ using a new eigenvector $[\Psi]$ for the structure-pile-soil system as

$$\begin{Bmatrix} \{q\} \\ \{u_p\} \end{Bmatrix} = [\Psi]\{y\} \quad \cdots\cdots (7)$$

Therefore Eq. (6) can be rearranged as

$$\{\ddot{y}\} + [2\beta_j\omega_j]\{\dot{y}\} + [\omega_j^2]\{y\} = [\Psi]^T \begin{bmatrix} [P_a] \\ [P_b] \end{bmatrix} \begin{Bmatrix} \{\dot{V}_{od}\} \\ \{V_{od}\} \end{Bmatrix} \quad \cdots\cdots (8)$$

in which $[\omega_j^2]$ is the eigenvalue for jth vibration mode of the structure-pile-soil system, β_j=corresponding damping ratio which includes the structural damping and the hydrodynamic damping.

On the other hand, the water particle velocity and acceleration must be known for the response analysis of Eq. (8). But, these variables take random values in a real sea and require the application of random vibration approach. Their cross spectral density terms derived by Malhotra and Penzien[2] are used in this paper. Bretschneider's one-dimensional wave spectrum[11], which is a function of statistically known mean wave height $\overline{H}$ and mean wave period $\overline{T}$, is adopted to describe the random sea state. The power spectrum $[S_{FF}(\omega)]$ of generalized modal forces can be expressed as

$$[S_{FF}(\omega)] = [\Psi]^T \begin{bmatrix} [P_a] \\ [P_b] \end{bmatrix} \begin{bmatrix} [S_{\dot{V}_0\dot{V}_0}(\omega)] & [S_{\dot{V}_0 V_0}(\omega)] \\ [S_{V_0\dot{V}_0}(\omega)] & [S_{V_0V_0}(\omega)] \end{bmatrix} \begin{bmatrix} [P_a] \\ [P_b] \end{bmatrix}^T [\Psi] \quad \cdots\cdots (9)$$

where ω is the circular wave frequency, $S_{\dot{V}_0\dot{V}_0}(\omega)$, $S_{\dot{V}_0V_0}(\omega)$, $S_{V_0\dot{V}_0}(\omega)$ and $S_{V_0V_0}(\omega)$ are the cross spectral densities of water particle velocities and accelerations.

The power spectrum of the modal responses is

$$[S_{yy}(\omega)] = [H(\omega)][S_{FF}(\omega)][H(\omega)^*] \quad \cdots\cdots (10)$$

where

$$[H(\omega)] = [\omega_j^2 - \omega^2 + i\,2\,\omega\omega_j\beta_j]^{-1}$$

in which $[H(\omega)]$ is the complex frequency response function and $[H(\omega)^*]$ is its conjugate. The variance of the modal responses is

$$E\left[\begin{Bmatrix} \{q\} \\ \{u_p\} \end{Bmatrix} \begin{Bmatrix} \{q\} \\ \{u_p\} \end{Bmatrix}^T\right] = \int_{-\infty}^{\infty} [\Psi][S_{yy}(\omega)][\Psi]^T d\omega \quad \cdots\cdots (11)$$

Finally, the relative responses at each nodal point are also computed by transforming the modal responses due to the modal matrix of the superstructure. The variance of the responses is given as

$$E[\{u_a^c\}\{u_a^c\}^T] = [\Phi]^T E[\{q\}\{q\}^T][\Phi] \quad \cdots\cdots (12)$$

4. DYNAMIC RESPONSE TO EARTHQUAKES

The equilibrium condition of forces at the base nodal points due to earthquake loadings is the same as that given by Eq. (5). The compatibility condition of displacements is

$$\{u_b\} = [G]\{u_p\} + \{u_g\}] \quad \cdots\cdots (13)$$

where $\ddot{u}_g$ is the ground acceleration. In the present study, the ground acceleration is assumed to be horizontal and to be acting along the x-direction. If sea wave excitation is not considered, the hydrodynamic forces are due to response vibration of the structure in still water and are given as

$$\{F_a\} = -[C_A]\{\ddot{u}_a\} - [C_D]\{\dot{u}_a\} \quad \cdots\cdots (14)$$

where

$$[C_A] = [\diagdown\ \rho(C_m - 1)V\ \diagdown],\quad [C_D] = [\diagdown\ \rho C_d A\sqrt{\frac{8}{\pi}}\,\sigma_{\dot{u}_a}\ \diagdown]$$

$\sigma_{\dot{u}_a}$ is computed by an iterative procedure.

By the application of the substructure method, the governing equation of motion for the earthquake loading can now be expressed as

$$\begin{bmatrix} [I] & [\tilde{M}_{ap}] \\ [\tilde{M}_{pa}] & [\tilde{M}_p] \end{bmatrix} \begin{Bmatrix} \{\ddot{q}\} \\ \{\ddot{u}_p\} \end{Bmatrix} + \begin{bmatrix} [2\beta_{fj}\omega_{fj}] & [\tilde{C}_{ap}] \\ [\tilde{C}_{pa}] & [\tilde{C}_p] \end{bmatrix} \begin{Bmatrix} \{\dot{q}\} \\ \{\dot{u}_p\} \end{Bmatrix} + \begin{bmatrix} [\omega_{fj}^2] & 0 \\ 0 & [\tilde{K}_p] \end{bmatrix} \begin{Bmatrix} \{q\} \\ \{u_p\} \end{Bmatrix} = -\begin{bmatrix} [P_a] \\ [P_b] \end{bmatrix} \{\ddot{u}_g\} \quad \cdots\cdots (15)$$

where

$[P_a]=[\Phi]^T[\tilde{M}_{aa}][L][G]$, $[P_b]=[G]^T[[L]^T[\tilde{M}_{aa}][L]+[M_{bb}]][G]$

All the matrices on the left-hand side of Eq. (15), have the same expressions as those given earlier for Eq. (6). However, the matrix for the hydrodynamic coefficients $[C_D]$ has the form as defined in Eq. (14) since hydrodynamic damping by sea waves is not considered. Further, Eq. (15) contains nonproportional damping matrix, and hence complex eigenvalue analysis must be carried out for exact response analysis. For the same reason as stated previously, modal coupling effects are neglected and simplifying approximation is made by considering only the diagonal terms.

For the response analysis using Eq. (15), the ground accelerations must be known. Since earthquake ground motions are random processes, their characteristics are modelled stochastically using power spectral density functions. In this study, the ground accelerations are represented by the Tajimi-Kanai's[12] expression for the stationary filtered white noise. The power spectrum of the generalized modal forces is

$$[S_{FF}(\omega)]=[\Psi]^T\begin{bmatrix}[P_a]\\ \\ [P_b]\end{bmatrix}\begin{bmatrix} & & \\ & S_{\ddot{u}_g\ddot{u}_g}(\omega) & \\ & & \end{bmatrix}\begin{bmatrix}[P_a]\\ \\ [P_b]\end{bmatrix}^T[\Psi] \quad\cdots\cdots (16)$$

where $S_{\ddot{u}_g\ddot{u}_g}(\omega)$ is the power spectrum of the ground acceleration $\ddot{u}_g$, and the eigenvector $[\Psi]$ is defined by Eq. (7). Once $S_{FF}(\omega)$ is known, the variance of the modal responses and the relative responses at each nodal point are similarly computed using Eqs. (11) and (12).

5. COMPARISON OF SEA WAVE RESPONSE AND EARTHQUAKE RESPONSE

After computing the responses, the next step would be to clarify the reliability of offshore structures. A critical reliability analysis must include a systematic analysis of uncertainties associated with the offshore environment loadings, geotechnical conditions and performance of structures. However, in this study, only a preliminary reliability analysis is carried out to examine the relative importance of wave and seismic responses. The probability distribution of time to first passage across specific barriers is determined.

For a zero-mean Gaussian process $x(t)$ with spectral density function $G_x(\omega)$ about its central frequency ω_0 the degree of dispersion is expressed using a bandwidth parameter q_x[13),14)] as

$$q_x=\left(1-\frac{\alpha_1^2}{\alpha_0\alpha_2}\right)^{1/2} \qquad 0\leq q_x\leq 1 \quad\cdots\cdots (17)$$

where

$$\alpha_i=\int_0^\infty \omega^i G_x(\omega)d\omega \qquad (i=0,\ 1,\ 2)$$

The reliability $L(\lambda)$ of the structure corresponds to the probability of $x(t)$ not exceeding the critical barrier λ and is given as

$$L(\lambda)=\exp\left[-\frac{1}{\pi}\sqrt{\frac{\alpha_2}{\alpha_0}}\ t_0\exp\left(-\frac{\gamma^2}{2}\right)C_1\right] \quad\cdots\cdots (18)$$

where

$$C_1=\frac{1-\exp\left[-\sqrt{\frac{\pi}{2}}\ q_x\gamma\right]}{1-\exp\left(-\frac{\gamma^2}{2}\right)}$$

in which t_0 is the duration of the input excitation and $\gamma=\lambda/\sqrt{\alpha_0}$ is the reduced level.

6. RESULTS AND DISCUSSIONS

The dynamic response analysis is carried out for the offshore tower model shown in Fig. 1. The tower is 120 m high and the depth of water is 110 m from mean sea level. The main members have an outer diameter of 2.8 m and a thickness of 27 mm. The structural members as well as the piles are made of steel. The shear velocity V_s in the soil is assumed to be 100 m/sec. The natural frequencies and vibrational mode shapes are computed by eigenvalue analysis, firstly for the rigidly supported base condition (case I) and then for the soil-structure interaction condition (case II). C_m is the inertia coefficient and C_d is the drag coefficient, 2.0 and 1.0 are used in this analysis, respectively. The corresponding values of natural periods for upto third mode of vibration are shown in Table 1. Since the soil-pile-foundation system is relatively stiff, there is not much difference between the natural periods of both cases.

(1) Dynamic response to random sea waves

Fig. 3 shows the examples of Bretschneider spectra for mean wave height $\overline{H}=7$ m, and mean wave period $\overline{T}=8$ sec, 10 sec and 12 sec. Higher frequency components have smaller energy while lower frequency components have greater energy. Fig. 4 shows examples of modal wave response spectra for the structure. Each spectrum has two peaks : one at the mean wave frequency corresponding to the wave force (peak 1) and the other at the first natural frequency of the structure-pile-soil system (peak 2) corresponding to the predominant frequency response function. The response curves are significantly affected by the input wave loading conditions (peak 1). On the other hand, when the mean wave period is larger, the peak values of the modal response curves (peak 2) are smaller due to increase in the hydrodynamic damping effects. This is due to the characteristics of the linearized damping forces. The equivalent damping coefficient C_D is a function of the relative velocity. When the mean wave period is larger, the wave energy, wave forces, relative velocity are also larger ; and therefore damping forces increase.

The rms responses are computed for mean wave periods ranging from 5 to 15 sec and for mean wave heights, $\overline{H}=3$ m (strong winds) and $\overline{H}=7$ m (severe storms). Fig. 5 shows the rms response displacement at node 1 (top node) and Fig. 6 shows the rms response bending moment at node 9 (base node) for different sea states. The responses considering the soil-structure interaction are compared with those for the rigidly supported base condition, and the first three vibrational modes are used for the analysis. It is shown that the responses are generally larger when the interaction effects are included. However, there are few differences when the mean wave period is longer, that is, the responses are little affected by the foundation type. Further, as the mean wave period approaches the predominant period of the structure,

Table 1 Natural periods of the tower (sec).

Vibration mode	Rigidly supported base	Soil-structure interaction
First	3.32	3.36
Second	0.54	0.54
Third	0.48	0.50

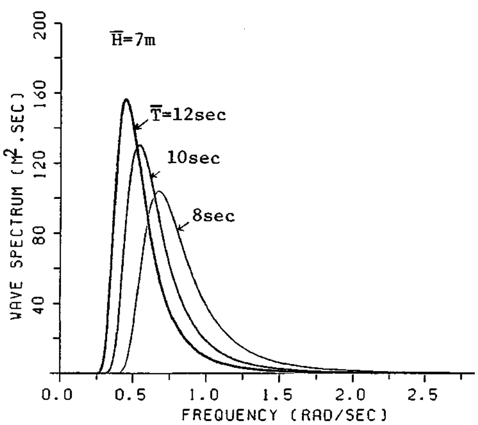

Fig. 3 Bretschneider's wave energy spectrum.

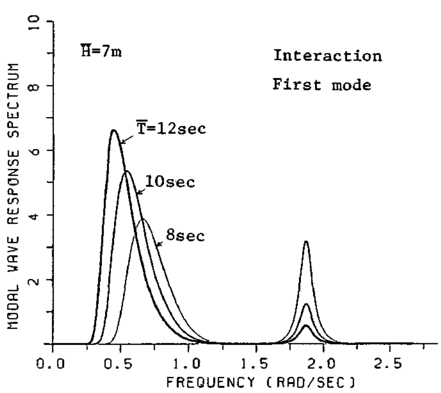

Fig. 4 Modal wave response spectrum.

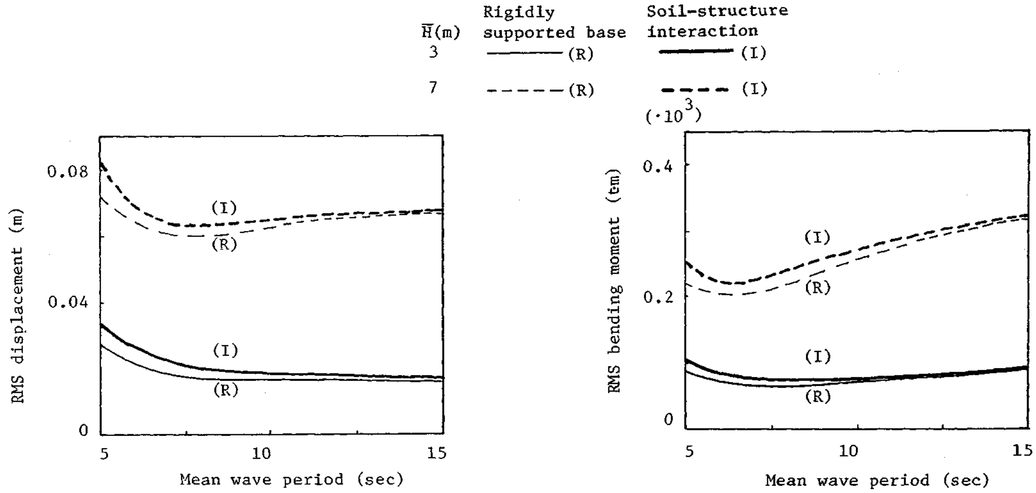

Fig. 5 RMS displacement at node 1.

Fig. 6 RMS bending moment at node 9.

the response values rapidly increase because of the resonance phenomenon. Similar results are likewise observed for rms response bending moment. Therefore, it is very important to determine the predominant natural frequency of the offshore structure accurately, in order to include the vibrational mode effects on the dynamic soil-structure interaction.

(2) Dynamic response to earthquakes

The size of an earthquake is expressed quantitatively as 'magnitude', which is a measure of the amount of strain energy at the source. For earthquakes of magnitude more than 4 (Japanese scale), ground motions are severe and structural damages are anticipated. Fig. 7 is a plot of Kanai's ground acceleration spectrum for magnitude 4 (rms ground acceleration, $\sigma_{\ddot{u}_g}=30$ gal) and magnitude 5 ($\sigma_{\ddot{u}_g}=100$ gal). Marine soil is usually soft and therefore a characteristic ground frequency of 10 rad/sec is assumed. An example of modal seismic response spectrum is shown in Fig. 8. Unlike wave response spectrum, this spectrum has only one peak which is very sharp at the first natural frequency of the structure. This is due to the characteristics of the frequency response function. Since the peak frequency of the power spectrum of the ground motion is very far from the predominating frequency of the structure, it is shown that the corresponding frequency response function has significant effects on the response.

Fig. 9 shows the rms displacement and rms bending moment at node 1 and node 9, respectively. The rms responses increase with the increase in the ground acceleration. The dynamic response for the

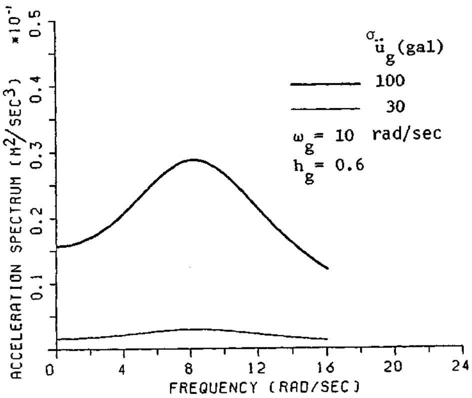

Fig. 7 Kanai's ground acceleration spectrum.

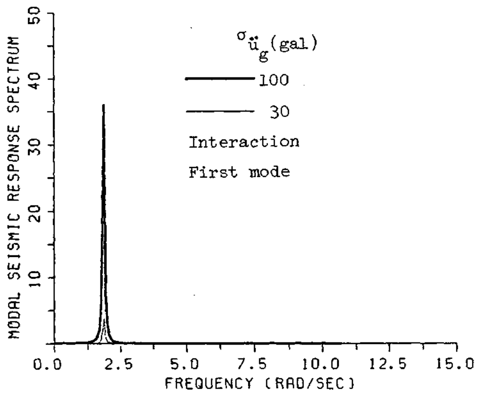

Fig. 8 Modal seismic response spectrum.

soil-structure interaction condition is generally smaller than that for the rigidly supported base condition because of the energy dissipation of the pile-soil foundation system. This phenomenon has different properties from the response for sea wave loading. Since the response velocity of the structure is small regardless of the intensity of input motion, the nonlinear drag force has few contributions on the response.

(3) Comparison of sea wave response and earthquake response

Since the wave response of the structure varies with wave field parameters namely mean wave height $\overline{H}$ and mean wave period $\overline{T}$, and the seismic response is a function of the ground vibration, expressed quantitatively as the rms ground acceleration, it is not possible to compare those responses directly. But, by employing the principles of first-passage probabilities, it is possible to estimate the relative significance of these responses. The wave loading condition, for which the mean wave height is 7 m and the mean wave period is 10 sec, is assumed. This sea state has a return period of approximately a few decades. The earthquake loading conditions are represented using rms ground accelerations, varying from 20 gal to 150 gal, which correspond to design ground motions. The reliabilities on the first passage are presented in Fig. 10 for the rigidly supported base condition and in Fig. 11 for the soil-structure interaction condition. The barrier level λ is expressed

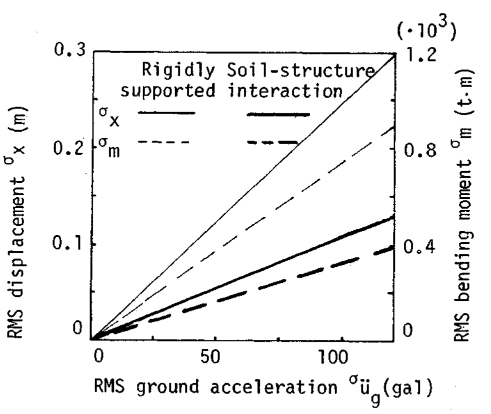

Fig. 9 RMS displacement at node 1 and RMS bending moment at node 9.

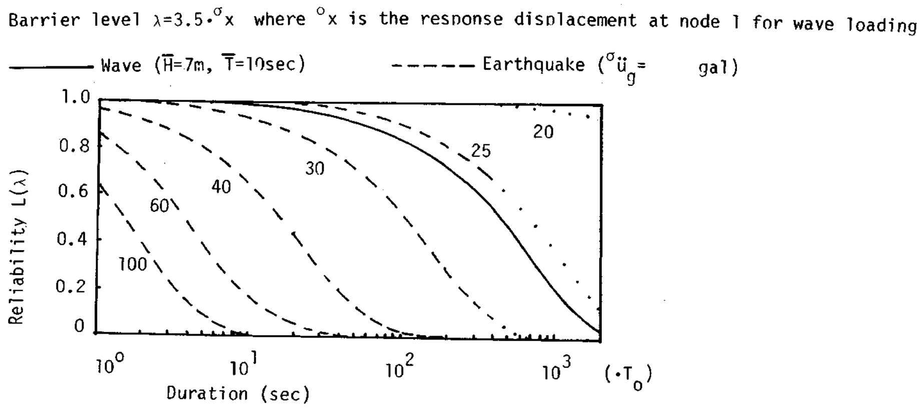

Fig. 10 Reliability of wave and earthquake loadings (Rigidly supported base).

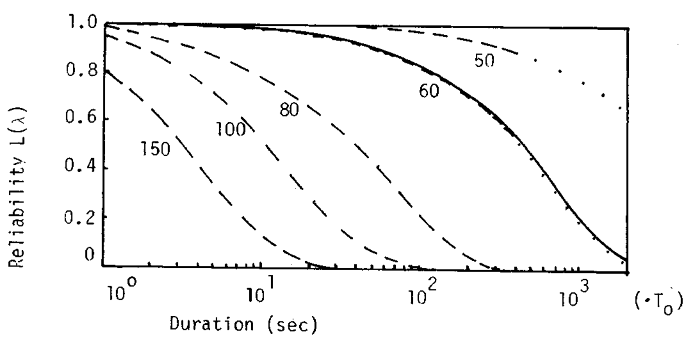

Fig. 11 Reliability of wave and earthquake loadings (Soil-structure interaction).

in terms of the rms response displacement of node 1 for the wave loading. The duration time t_0 is expressed in terms of the first natural period T_0 of the structure-pile-soil system. Since, the wave motion has the duration time of a few hours, reliable displacement can be evaluated for a large value of peak factor. On the other hand, the corresponding displacement may be caused by very severe earthquake ground motion because earthquake ground motion has comparitively short duration time of less than a few minutes. Therefore, studies on the first passage probabilities indicate that wave loadings have more significant effects on response evaluations because of the longer duration time for the numerical examples of this study.

7. CONCLUSIONS

The principal conclusions of this study are as follows :

(1) The response of offshore structures mainly depends on the first few vibration modes. Therefore it is important to determine accurately these vibration modes and the natural frequencies.

(2) The wave response normally increases with increasing mean wave height and mean wave period. The response values are strongly influenced by the drag forces.

(3) The wave response for the soil-structure interaction condition is higher than that for the rigidly supported base condition. However, for longer wave periods there are few differences in the response values.

(4) The seismic response varies with the input ground acceleration and the first natural frequency. The response values are lower for the soil-structure interaction condition than for the rigidly supported base condition due to the energy dissipation of the pile-soil foundation system.

(5) The effects of drag force provide small contributions on the seismic response including the dynamic soil-structure interaction.

(6) The reliability studies on the first passage probabilities for sea waves and earthquake loadings show that wave loading is the dominating design criterion because of the longer duration time.

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REFERENCES

1) Foster Jr., E. T. : Model for nonlinear dynamics of offshore towers, Journal of the Engineering Mechanics Division, ASCE, Vol. 96, No. EM 1, pp. 41~67, 1970.

2) Malhotra, A. K. and Penzien, J. : Nondeterministic analysis of offshore structures, Journal of the Engineering Mechanics Division, ASCE, Vol. 96, No. EM 6, pp. 985~1003, 1970.

3) Clough, R. W. and Penzien, J. : Dynamics of Structures, 1st ed., McGraw-Hill Kogakusha, Ltd., pp. 611~628, Tokyo, 1975.

4) Singh, M. P. and Chu, S. L. : Stochastic considerations in seismic analysis of structures, Earthquake Engineering and Structural Dynamics, Vol. 4, pp. 295~307, 1976.

5) Penzien, J., Kaul, M. K. and Berge, B. : Stochastic response of offshore towers to random sea waves and strong motion earthquakes, Computers and Structures, Vol. 2, pp. 733~756, 1972.

6) Takemiya, H., et al. : Soil-structure interaction of gravity-type steel offshore structure for earthquakes, Presented at the 1986 International Conference on Offshore Mechanics and Arctic Engineering held at Tokyo, Japan, pp. 663~670.

7) Kawano, K., Yamada, Y. and Venkataramana, K. : Dynamic response analysis of soil-offshore structure systems, Proceedings of the 3rd International Conference on Civil and Structural Engineering Computing held at London, Vol. 1, pp. 315~319, 1987.

8) Bea, R. G. : Dynamic Response of Piles in Offshore Platforms, Dynamic Response of Pile Foundations : Analytical Aspects, ASCE, pp. 80~109, 1980.

9) Dao, B. V. and Penzien, J. : Comparison of treatments of non-linear drag forces acting on fixed offshore platforms, Applied Ocean Research, Vol. 4, No. 2, pp. 66~72, 1982.

10) Yamada, Y. and Kawano, K. : Seismic response analysis of nonproportional damping system due to response spectrum method, Journal of Structural Engineering/Earthquake Engineering, JSCE, Vol. 4, No. 1, pp. 213~222, 1987.

11) Horikawa, K. : Coastal Engineering, 1st ed., University of Tokyo Press, Tokyo, pp. 61~67, 1978.

12) Tajimi, H. : A Statistical Method of determining the Maximum Response of a Building Structure During an Earthquake, Proceedings of the 2nd World Conference on Earthquake Engineering, International Association for Earthquake Engineering, Vol. 2, pp. 781~793, 1960.

13) Vanmarcke, E. H. : On the Distribution of the First-Passage Time for Normal Stationary Random Processes, Journal of Applied Mechanics, ASME, Vol. 42, pp. 215~220, 1975.

14) Kiureghian, A. D. : Structural Response to Stationary Excitation, Journal of the Engineering Mechanics Division, ASCE, Vol. 106, No. EM 6, pp. 1195~1213, 1980.

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