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# SEISMIC RESPONSE OF OFFSHORE STRUCTURES IN RANDOM SEAS

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## SUMMARY

Presented in this paper is a method for dynamic response analysis of offshore structures subjected to simultaneous loadings by earthquake ground motions and random sea waves. Emphasis is placed on the evaluation of nonlinear hydrodynamic damping effects due to sea waves for the seismic response. The governing equation of motion is obtained by the substructure method. Response analysis is carried out using the frequency-domain random vibration approach. It is found that sea waves act as a damping medium and reduce the amplitude of seismic response of offshore structures. The net response displacements of structural nodal points, due to moderate to severe earthquake ground motions in random seas of small wave heights, are less than that due to earthquake motions in still water.

## INTRODUCTION

For any offshore structure located in a seismically active region, random sea waves and earthquake ground motions are two main design loads. Numerous researches (For example, Refs.1,2,3) have been carried out in the past on the analysis of offshore structures subjected to these loads separately. Penzien, et al (Ref.1) presented a stochastic method of analysis of fixed-base offshore towers due to random sea waves and strong motion earthquakes. Bea (Ref.2) developed design criteria for offshore platforms subjected to these loads and compared his results with onshore building structures. Anagnostopoulos (Ref.3) carried out studies on the evaluation of modal solutions of offshore platforms.

Offshore structures will interact not only with the surrounding water and sea waves, but also with the subsoil. Dynamic loads acting on the structures develop high dynamic forces on the foundations. Earthquake excitation is transmitted to the structures through the soil. Takemiya, et al (Ref.4) have shown that the internal forces due to earthquake ground motions are halved if the soil-structure interaction is not incorporated. In their analysis, effects of hydrodynamic forces due to sea waves were not taken into account.

The aim of this paper is to investigate the response of offshore structures, subjected to simultaneous loadings by earthquake ground motions and random sea waves considering the dynamic interactions among wave, structure and foundations. Models considered are jacket structures constructed on pile-soil foundations. Numerical results are presented for various combinations of typical sea states and earthquake ground motions.

Fig.1 shows the elevation of an offshore tower resting on pile-soil foundation. The equation of motion is obtained by substructure method in which the structure-pile-soil system is hypothetically divided into two substructures: the structure and the pile-soil foundation. The net displacement of the structure is expressed as the sum of the dynamic displacement of the structure on a fixed-base, and the quasi-static displacement due to the interactions with the foundation. The dynamic displacements of the fixed-base structure are treated as a linear combination of the first few vibration modes which have significant effects on the response. The quasi-static displacements are expressed as the superposition of a few generalized displacements. The dynamic stiffness coefficients of the pile-soil foundation are interpreted as a generalized spring-dashpot system. While these impedence functions are dependent upon the excitation frequency, a simplified model being independent of the frequency (Ref.5) is assumed in this paper since the first few dominating frequencies of the structure-pile-soil system are within comparitively low frequency range. The equation of motion for the structure-pile-soil system is obtained by combining the equations of motion for the structure subsystem and the pile-soil subsystem and by satisfying the compatibility conditions of displacements and the equilibrium conditions of forces at the base nodal points.

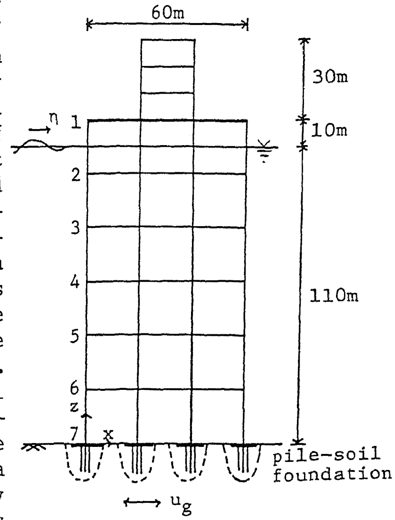

Fig.1 Analytical model of structure-pile-soil system

DYNAMIC RESPONSE ANALYSIS METHOD

The hydrodynamic forces due to sea waves are expressed using the well-known Morison equation. The nonlinear relative-velocity squared drag term in this equation is replaced by an equivalent linearized drag term in a classical manner (Ref.6) assuming Gaussian random process for the relative velocity distribution.

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

$$\{u_b\} = [G][\{u_p\} + \{u_g\}] \tag{1}$$

in which $\{u\}$ is the displacement vector, subscripts b and p denote the nodal points at the structure-foundation interface and at the pile-soil foundation respectively, $\{u_g\}$ is the ground displacement due to earthquake input and matrix $[G]$ connects the deformations at the pile head and at the base nodes. In the present study, ground displacement is assumed to be horizontal and to be acting along the x-direction. The equilibrium equation of forces at each of the base nodal point is

$$\{F_b\} + [G]\{F_p\} = 0 \tag{2}$$

Finally the dynamic equation of motion for the structure-pile-soil system is expressed as

$$\begin{bmatrix} [I] & [\tilde{M}_{ap}] \\ [\tilde{M}_{pa}] & [\tilde{M}_p] \end{bmatrix} \begin{Bmatrix} \{\ddot{q}\} \\ \{\ddot{u}_p\} \end{Bmatrix} + \begin{bmatrix} [2\beta_{fj}\omega_{fj}] & [\tilde{C}_{ap}] \\ [\tilde{C}_{pa}] & [\tilde{C}_p] \end{bmatrix} \begin{Bmatrix} \{\dot{q}\} \\ \{\dot{u}_p\} \end{Bmatrix} + \begin{bmatrix} [\omega_{fj}^2] & 0 \\ 0 & [\tilde{K}_p] \end{bmatrix} \begin{Bmatrix} \{q\} \\ \{u_p\} \end{Bmatrix} = \begin{bmatrix} [P_a] \\ [P_b] \end{bmatrix}_w \begin{Bmatrix} \{\dot{V}_{oa}\} \\ \{V_{oa}\} \end{Bmatrix} - \begin{bmatrix} [P_a] \\ [P_b] \end{bmatrix}_e \{\ddot{u}_g\} \tag{3}$$

where

$$\begin{bmatrix}[P_a]\\ [P_b]\end{bmatrix}_w = \begin{bmatrix}[\Phi]^T[C_M] & [\Phi]^T[C_D]\\ [G]^T[L]^T[C_M] & [G]^T[L]^T[C_D]\end{bmatrix}, \quad \begin{bmatrix}[P_a]\\ [P_b]\end{bmatrix}_e = \begin{bmatrix}[\Phi]^T[\bar{M}_{aa}][L][G]\\ [G]^T\left[[L]^T[\bar{M}_{aa}][L] + [M_{bb}]\right][G]\end{bmatrix}$$

$$[\bar{M}] = [M] + [C_A], \quad [\bar{C}] = [C] + [C_D]$$

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 for jth mode of the fixed-base structure, $[\beta_{fj}]$ is the corresponding damping ratio which includes both the structural damping and the hydrodynamic damping, $\{q\}$ is the modal displacement vector for dynamic displacements, $\{V_{oa}\}$ and $\{\dot{V}_{oa}\}$ are the water particle velocity and acceleration vectors respectively at the undeflected structure coordinate locations, $[C_M]$ is the inertia factor, $[C_D]$ is the drag factor, $[C_A]$ is the added mass factor and superscript T denotes the transpose of a matrix.

Responses may be obtained from Eq.(3) by substituting for water particle velocities, water particle accelerations and ground accelerations. But, these variables take random values and require the application of random vibration approach. In the present study, Bretschneider's one-dimensional wave spectrum (Ref.7) which is a function of statistically known mean wave height $\bar{H}$ and mean wave period $\bar{T}$, is adopted to describe the random sea state. Ground accelerations are represented by the Tajimi-Kanai's expression (Ref.8) for stationary filtered white noise. The power spectrum of generalized modal forces is now expressed as

$$[S_{FF}(\omega)] = [\Psi]^T \begin{bmatrix}[P_a]\\ [P_b]\end{bmatrix}_w \begin{bmatrix}[S_{\dot{V}_o\dot{V}_o}(\omega)] & [S_{\dot{V}_o V_o}(\omega)]\\ [S_{V_o\dot{V}_o}(\omega)] & [S_{V_o V_o}(\omega)]\end{bmatrix} \begin{bmatrix}[P_a]\\ [P_b]\end{bmatrix}_w^T [\Psi] + [\Psi]^T \begin{bmatrix}[P_a]\\ [P_b]\end{bmatrix}_e \begin{bmatrix}S_{\ddot{u}_g\ddot{u}_g}(\omega)\end{bmatrix} \begin{bmatrix}[P_a]\\ [P_b]\end{bmatrix}_e^T [\Psi]$$

where

$$\begin{Bmatrix}\{q\}\\ \{u_p\}\end{Bmatrix} = [\Psi]\ \{y\} \tag{4}$$

in which $\omega$ is the circular wave frequency, $S_{V_oV_o}(\omega)$, $S_{V_o\dot{V}_o}(\omega)$, $S_{\dot{V}_oV_o}(\omega)$, $S_{\dot{V}_o\dot{V}_o}(\omega)$ are the cross spectral densities of water particle velocities and accelerations and $S_{\ddot{u}_g\ddot{u}_g}(\omega)$ is the power spectrum of ground acceleration. Power spectrum of modal responses is obtained by

$$[S_{yy}(\omega)] = [H(\omega)][S_{FF}(\omega)][H(\omega)^*] \qquad \text{where} \qquad [H(\omega)] = [\ \omega_j^2 - \omega^2 + i2\omega\omega_j\beta_j\ ]^{-1} \tag{5}$$

in which $[H(\omega)]$ is the complex frequency response function and $[H(\omega)^*]$ is its conjugate, $[\omega_j^2]$ is the eigenvalue for the jth vibration mode of the structure-pile-soil system, $[\beta_j]$ is the corresponding damping ratio which includes the structural damping and the hydrodynamic damping.

## RESULTS AND DISCUSSIONS

The dynamic response analysis is carried out for the offshore tower model shown in Fig.1. The depth of water is 110m from mean sea level. The main members have an outer diameter of 2.8m and a thickness of 27mm. Each leg of the tower rests on a pile-soil foundation. The structural members as well as the piles are made of steel. The structures are discretized by lumping masses at selected nodal points. Each node, except at the base, has 3 degrees of freedom: horizontal displacement (in x-direction), vertical displacement (in z-direction) and rotation (about y-direction). The base nodes are restrained from vertical movement. The shear velocity $V_s$ in the soil is assumed to be 100m/sec. The natural frequencies and the vibrational mode shapes are computed by eigenvalue analysis, firstly for the rigidly supported base condition and then for the soil-structure interaction condition. The values of natural frequencies are shown in Table 1.

Table 1 Natural frequencies of structure-pile-soil system (:rad/sec)

| Vibration mode | Rigidly supported base | Soil-structure interaction |
|---|---|---|
| First | 2.24 | 1.61 |
| Second | 11.88 | 8.87 |
| Third | 27.49 | 25.14 |

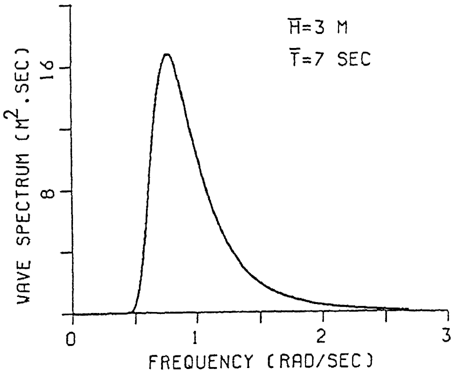


Fig.2 Bretschneider's wave spectrum

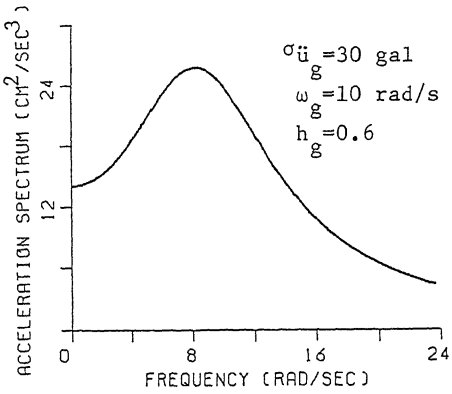


Fig.3 Tajimi-Kanai's ground acceleration spectrum

Fig.2 shows the Bretschneider wave energy spectrum for mean wave height $\bar{H}$=3m, and mean wave period $\bar{T}$=7sec. Fig.3 is a plot of Tajimi-Kanai's ground acceleration spectrum for rms ground acceleration $\sigma_{\ddot{u}_g}$=30gal. Marine soil is usually soft and therefore a characteristic ground frequency of 10rad/sec is assumed. A modal response spectrum for first vibration mode of the soil-offshore structure system, for wave loadings, is shown in Fig.4. This spectrum has two main peaks corresponding to the mean wave frequency and the fundamental frequency of the structure. The other peaks and the zeros are due to the effects of phase difference between wave forces on members horizontally separated. Fig.5 shows the modal response spectrum for vibrations in still water due to earthquakes. This spectrum has only one peak, which is very sharp, near the fundamental frequency of the structure since the characteristic ground frequency and the fundamental frequency of the structure are well separated. Responses decrease rapidly when the excitation frequency moves away from the fundamental frequency of the structure.

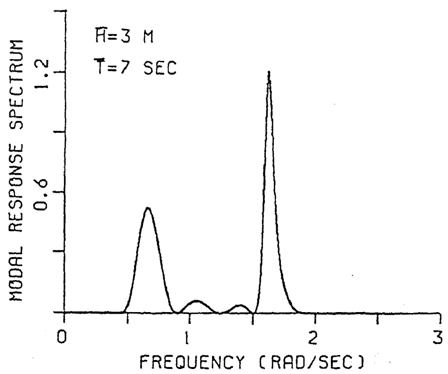


Fig.4 Modal response spectrum (waves only)

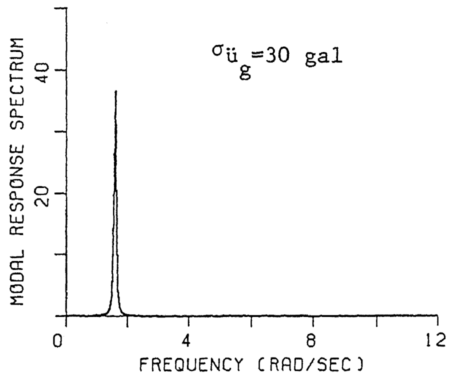


Fig.5 Modal response spectrum (earthquakes only)

Fig.6 shows the example of modal response spectrum for vibrations in random seas due to simultaneous inputs of earthquakes and waves. While the first mode contributes significantly for the modal response due to earthquake ground motions only, there are very different characteristics of modal responses for simultaneous loadings by earthquakes and sea waves. It is observed that the responses are very sensitive to the intensity of earthquake motions and sea wave forces. Further, in the absence of sea waves, hydrodynamic damping forces which are proportional to the structural velocities are relatively small. On the other hand, if sea waves are included, damping forces are now proportional to the relative velocities between the waves and the structure. Since the wave velocities are very much higher than the structural velocities, damping forces become larger than those without waves and the responses are considerably reduced.

Fig.7 shows the rms displacement at node 1 (top node) due to earthquake loadings in still water. The responses increase with the increase in the ground acceleration. Since the hydrodynamic damping for first mode of the interaction system is small and the natural period of the interaction system is longer, dynamic response for the soil-structure interaction condition is generally larger than that for the rigidly supported base condition. Further, the nonlinear damping force has few contributions on the response evaluations because the response velocities of the structure are small.

The rms displacement at node 1 for simultaneous earthquake and wave loadings is plotted against rms ground acceleration in Fig.8. For rms accelerations of earthquake motions over about 30gal, the displacement responses due to simultaneous loadings of earthquake motions and sea wave forces are smaller than that due to earthquake motions in still water. The differences of these responses have increasing tendency for an increase of mean wave period which is due to increase in the nonlinear damping forces for severe loadings of earthquake forces and sea wave forces. However, the probability of simultaneous occurrence of severe earthquake motions and very strong sea waves is very small. In moderate wave conditions, which correspond to mean wave heights under about 3m, the probability, with which the offshore structure is hit by severe earthquake

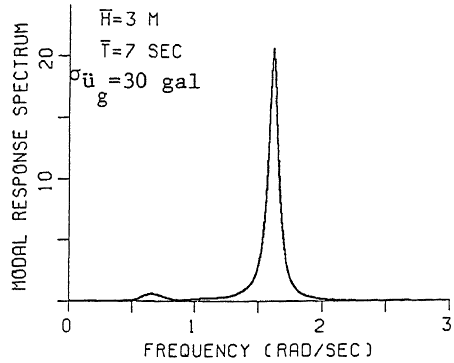


Fig.6 Modal response spectrum (earthquakes and waves)

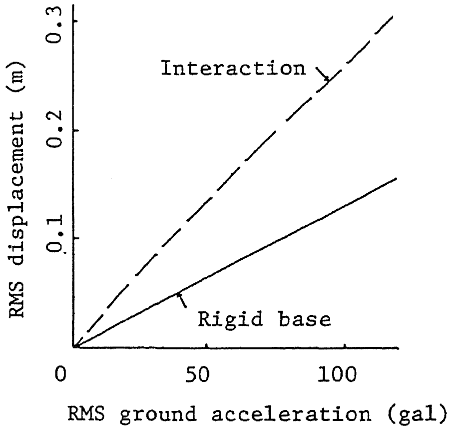


Fig.7 RMS displacement at node 1 (earthquakes only)

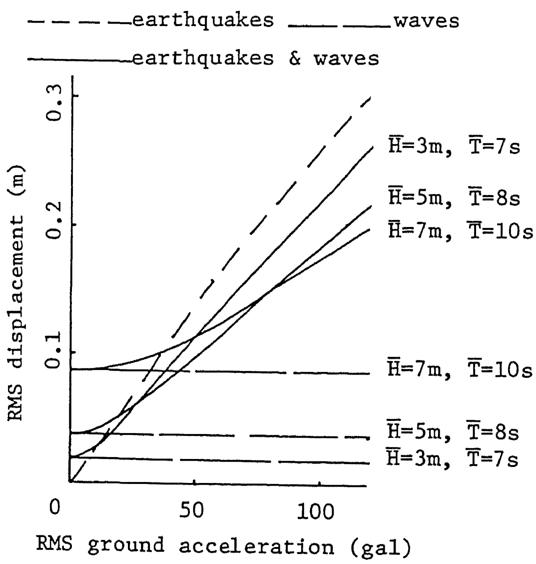


Fig.8 RMS displacement at node 1 (earthquakes and waves)

motions, is considerably large. The response evaluations due to simultaneous loadings of earthquake motions and sea wave forces provide more reliable and conservative values in comparison with the severe earthquake motions only in still water. On the other hand, when the offshore structure is subjected to small earthquake motions, the responses due to simultaneous loadings are generally larger.

Therefore, the seismic responses are mainly controlled by the intensity of earthquake forces, the proximity of the excitation frequency to the fundamental frequency of the offshore structure systems and the nonlinear damping effects of the sea waves.

## CONCLUSIONS

1. The responses of offshore structures mainly depend on the first few vibration modes. Therefore it is important to determine accurately these vibration modes and the corresponding natural frequencies.
2. Responses due to earthquake motions vary with the intensity of the input ground acceleration. The responses are higher for the soil-structure interaction condition than for the rigidly supported base condition. In the absence of sea waves, the effects of nonlinear damping are small.
3. When the earthquakes and sea waves act simultaneously, the nonlinear hydrodynamic damping forces are proportional to the relative velocities between the waves and the structure. Since the wave velocities are very much higher than the structural velocities, damping forces become larger than those without waves. Sea waves act as a damping medium and reduce the amplitude of the seismic response of offshore structures.

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