(114) WAVE AND EARTHQUAKE INDUCED VIBRATION OF OFFSHORE STRUCTURES

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1. INTRODUCTION

The exploitation of offshore areas of seas and oceans for constructing the civil engineering structures has necessitated the reassessment of the methods of design. For a typical offshore structure located in a seismically active region, the wave loading and the earthquake loading are the two main design factors. Numerous researches have been carried out in the past on the wave response as well as the earthquake response characteristics of offshore structures, but few examples are found the correlation of those results.

In this paper, the authors have presented a comparitive study of the dynamic responses of an offshore tower subjected to wave and earthquake loadings. The elevation of the tower model is shown in Fig.1. The main members of the tower are made of steel and each member has an outer diameter of 2.8m. The base of the tower rests on a soil-pile foundation constructed using steel piles.

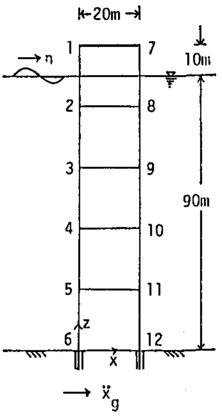

Fig.1 Offshore Tower model

The random waves around the tower are represented by the Bretschneider type one dimensional power spectrum. While formulating the earthquake effects, it is assumed that the movement of the sea bed is horizontal and also that the structure can not produce waves of appreciable amplitude by their motion, so that we can neglect the effects of radiation damping. Tajimi and Kanai's power spectrum is used for the stationary filtered white noise of input ground acceleration. The dynamic stiffness coefficients of the soil-pile foundation are interpreted as a generalised spring dash-pot system. The analysis is carried out in the frequency domain using the mode superposition principle.

2. FORMULATION OF EQUATION OF MOTION

The structure is discretized by the lumping of mass at selected nodal points. The dynamic equation of motion will have the form

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

in which suffix a and b denote the unrestrained nodal point of the superstructure and the fixed nodal point at the base respectively.

The displacement $\{x_a\}$ of the superstructure is the sum of the dynamic displacement $\{x_a^c\}$ due to inertia forces and the quasi-static displacement $\{x_b\}$ due to the vibration of the foundation. Therefore,

$$\begin{Bmatrix} \{x_a\} \\ \{x_b\} \end{Bmatrix} = \begin{bmatrix} [I] & [L] \\ [0] & [I] \end{bmatrix} \begin{Bmatrix} \{x_a^c\} \\ \{x_b\} \end{Bmatrix} \quad (2)$$

in which [I] is the unit matrix and $[L]=-[K_{aa}][K_{ab}]^{-1}$.

The force vector $\{F_a\}$ which takes into account the fluid-structure interaction is expressed by the Morison equation considering the relative motion between the waves and the structure. The nonlinear drag term is replaced by a linearised drag term based on the assumption that the relative velocity is a zero-mean Gaussian process. While computing the earthquake response, the wave motion is neglected, and the force vector $\{F_a\}$ is

$$\{F_a\} = [C_M]\{\ddot{x}_a\} + [C_D]\{\dot{x}_a|\dot{x}_a|\} \quad (3)$$

The equation of motion of the soil-pile-foundation system is expressed by

$$[M_p]\{\ddot{x}_p\} + [C_p]\{\dot{x}_p\} + [K_p]\{x_p\} = \{R_s\} \quad (4)$$

The displacement vector $\{x_p\}$of the pile head consists of sliding and rocking motions. $\{R_s\}$ is the reaction force caused by the interactions between the superstructure and the foundation system.

The compatibility condition of the displacements and the equilibrium equation of the interacting forces are respectively,

$$\{x_b\} = [G](\{x_p\}+\{x_g\}) \quad (5) \qquad \{F_b\} + \{R_s\} = \{0\} \quad (6)$$

in which [G] matrix connects the displacements of the base of the tower to those of the pile head. $\{\ddot{x}_g\}$ is the ground acceleration at the nodal points of the pile head.

3. DYNAMIC RESPONSE ANALYSIS METHOD

The modal displacements of the structure are

$$\{x_a^c\} = [\Phi]\{q\} \quad (7) \qquad \begin{Bmatrix}\{q\}\\ \{x_b\}\end{Bmatrix} = [\Psi]\{y\} \quad (8)$$

where $[\Phi]$ is the modal matrix for the superstructure and $[\Psi]$ is the modal matrix for the structure including the foundation. These matrices are computed by the eigen value analysis of the governing equation of motion for undamped free vibrations.

The modal response spectrum of the tower is

$$[S_{yy}(\omega)] = [H(\omega)][\Psi][S_{FF}(\omega)][\Psi]^T[H(\omega)^*] \quad (9)$$

where $S_{FF}(\omega)$ is the force spectrum, $H(\omega)$ is the frequency response function and $H(\omega)^*$ is its conjugate. The auto correlation function of the modal response is obtained by the Fourier transformation of Eq.(9) and subsequently the r.m.s. responses are determined.

4. NUMERICAL EXAMPLE

Fig.2 shows the Bretschneider type wave energy spectrum in which $\bar{H}$ is the mean wave height and $\bar{T}$ is the mean wave period. Higher frequency component spectra have smaller energy and lower frequency component spectra have greater energy. The wave force spectrum at node 2 of the tower is calculated by the Morison equation and is plotted in Fig.3. C_m is the inertia coefficient and C_d is the drag

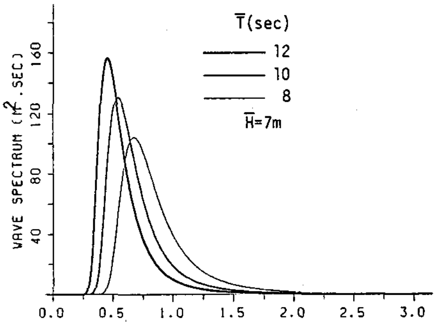

Fig.2 Bretschneider's Wave Energy Spectrum

coefficient. The ground motion is represented by the Tajimi-Kanai's stationary filtered white noise. Its shape is shown in Fig.6 where s is the intensity of ground acceleration.

The dynamic response is computed for three types of flexibilities of the tower. Two cases have been examined: Firstly, treating the bottom of the tower as fixed, and secondly considering the inter-action effects of the soil-pile system. The natural frequencies and the vibrational mode shapes are computed by eigenvalue analysis. The natural periods are shown in Table 1 for first mode.The examples of frequency response functions for wave re-sponse analysis are shown in Fig.4 and for seismic response analy-sis in Fig.7. The dis-crepancies in their values are due to the characteristics of the linearised wave force equation. The values are higher for seismic loading case bacause the hydrodynamic damping due to wave motion is not taken into account.

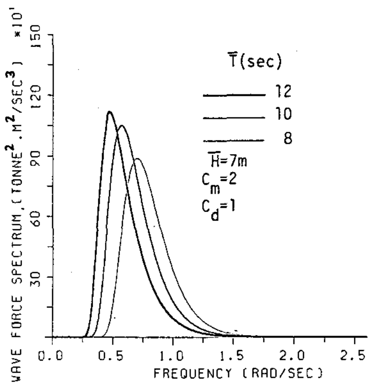

Fig.3 Wave Force Spectrum

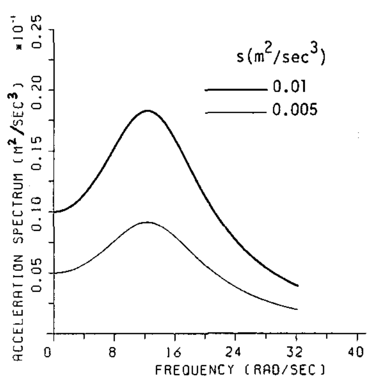

Fig.6 Tajimi and Kanai's Ground Acceleration Spectrum

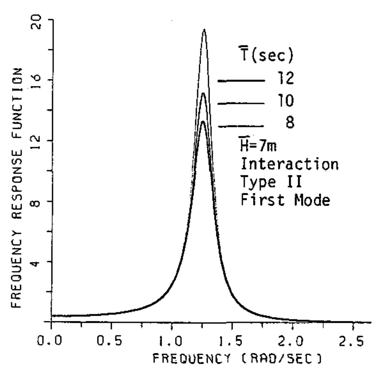

Fig.4 Frequency Response Function

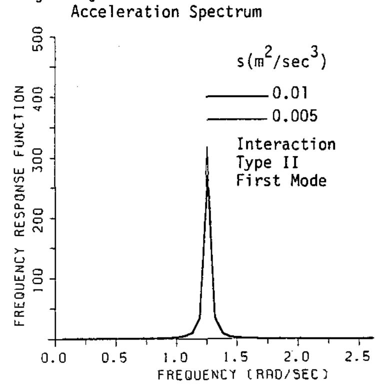

Fig.7 Frequency Response Function

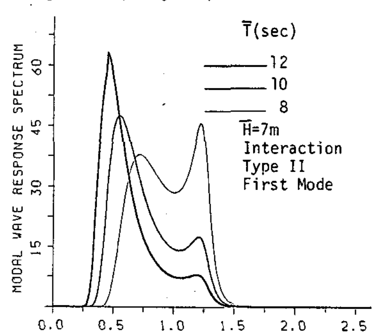

Fig.5 Modal Wave Response Spectrum

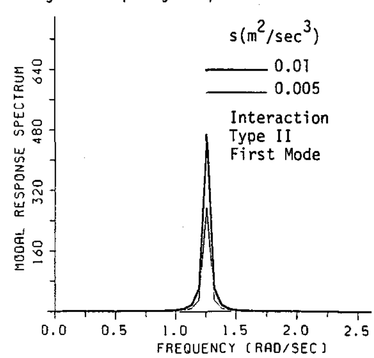

Fig.8 Modal Response Spectrum (Seismic)

Table 1. Natural Periods of the Offshore Tower (:sec)

Type	Fixed	Interaction
I	5.58	7.06
II	3.72	5.00
III	2.20	3.09

The wave response is computed for mean wave periods ranging from 4 to 15sec and mean wave heights ranging from 5 to 9m. The earthquake response is computed for intensities 0.01 and $0.005m^2/sec^3$ which correspond to r.m.s. ground accelerations of about 70 and 96gal respectively. Fig.5 shows typical modal displacement spectra for first mode of the type II tower subjected to wave loading and Fig.8 shows the modal displacement spectra for the same tower subjected to ground motion input.

The r.m.s displacements of node 1 are shown in Fig.9 for the interaction case and in Fig.10 for the fixed case. The maximum wave response is obtained when the wave period is nearer to the natural period of the structure. The response displacement is higher when the soil-pile interaction effects are considered.

The seismic response and the wave response are of the same order for the interaction case. But for the bottom-fixed tower, response to wave action is more significant.

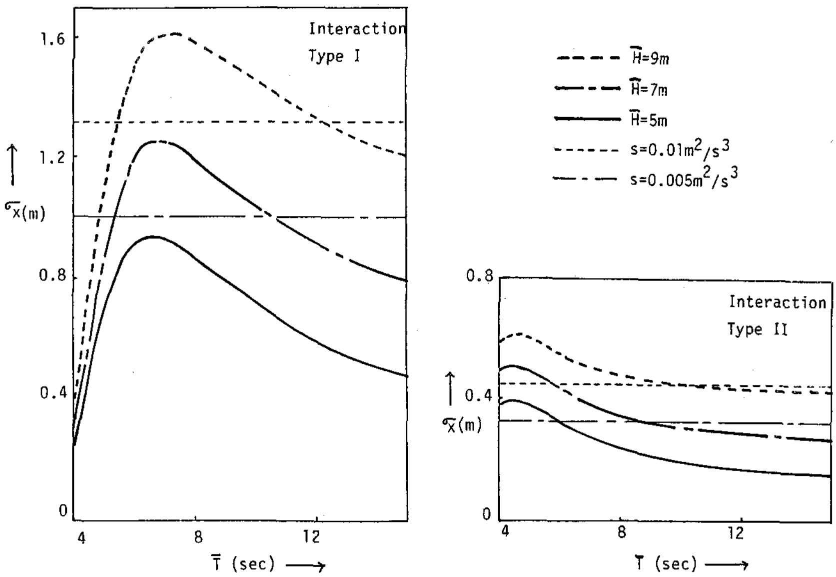

Fig.9 R.M.S.Displacement of Node 1 (Base Resting on Piled Foundation)

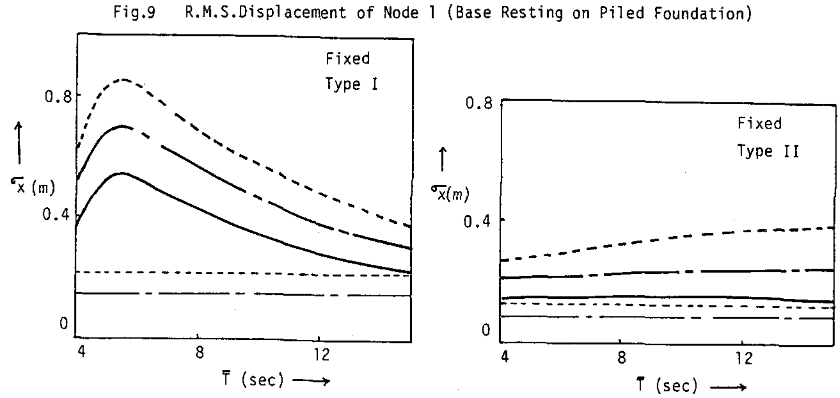

Fig.10 R.M.S.Displacement of Node 1 (Base Rigidly Fixed to Ocean Floor)

5. CONCLUSIONS

The dynamic response of an offshore tower subjected to wave and earthquake loadings is investigated. The results are expressed using mean wave height, mean wave period and the intensity of ground acceleration. The principal results and conclusions of this study can be summarised as follows:

(1) The wave responses are amplified when the natural periods are nearer to the mean wave periods. Hence it is advisible to design the structure in such a way that its natural period does not fall within the spectrum of the incident wave periods.

(2) The seismic loadings have significant effects on the response of offshore towers resting on piled foundations. However, by decreasing the fundamental period or by rigidly fixing the base to the ocean floor, the magnitude of the response can be controlled.

(3) The magnitudes of the seismic response and the wave response may be comparable under certain field conditions as illustrated in this paper.

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