한국전산구조공학회
1992년도 봄학술발표회 논문집

심해용 해양구조물의 지진하중에 대한 비정상거동해석

Nonstationary Seismic Response Analysis of Offshore Compliant Tower

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요 약

해양 가이드 타워를 대상으로 하여, 지진하중에 대한 심해용구조물의 비정상거동해법에 대하여 연구하였다. 지반운동의 비정상특성은 정상과정성분에 시간포락함수가 곱해진 형태로 모형화하였으며, 구조물의 비정상거동은 시간종속분산함수로 구하였다. 지반가속도에 대한 자기상관함수를 복소지수함수의 형태로 이상화함으로써, 구조물 거동의 시간종속함수가 해석적인 방법으로 쉽게 구할 수 있는 기법을 개발하였다. 지진의 발생시간 동안 예상되는 최대거동을 구하였으며, 이를 구조물 거동을 정상확률과정으로 가정하여 산정한 결과와 비교 분석하였다.

1. INTRODUCTION

The guyed tower is one of the compliant structures developed for deepsea oil production[1]. It is mainly composed of a slender tower and a guyline system as shown in Fig.1a. The guyline system is an array of guylines radiating from the tower and provides the lateral support to the tower.

Recently, many studies have been reported on the seismic analysis of offshore structures[2,3]. However, in most cases, the random characteristics particularly on the nonstationary behaviors are not analyzed rigorously. More rigorous approaches for nonstationary analysis have been reported for several cases of simpler structures[4,5]. By treating the nonstationary excitation as sequences of random pulses, Lin[4] developed a method to compute the response in terms of the long-characteristic functional. Shinozuka et al[5] developed a procedure to obtain the time dependent variance function of the response by modeling the earthquake ground motion as a filtered Poisson process.

In the present paper, a method for nonstationary response analysis of a guyed offshore tower subjected to strong seismic excitations is developed. The equation of motion is constructed for the horizontal motion of the tower using a multi-degree of freedom model. By taking the auto-correlation function of the ground acceleration in terms of complex exponential functions of time, an analytical procedure is developed for the computation of the time varying variances of the tower responses. Expected peak responses are also evaluated and the results are compared with those by the spectral method.

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2. EQUATION OF MOTION

An idealized structure as shown in Fig.1b is used for the dynamic analysis against earthquake loadings. The governing equation of motion for the structural model subjected to earthquake can be written as[2,3,6]

$$[\bar{M}]\{\ddot{X}\} + [C]\{\dot{X}\} + [K]\{X\} + \{1_g\}R(x_m) = -[\bar{M}]\{1\}\ddot{U}_g - [0.5\rho C_d A]\{(\dot{U}_g + \dot{X})|\dot{U}_g + \dot{X}|\} \quad (1)$$

where $\{X\}, \{\dot{X}\}$ and $\{\ddot{X}\}$= vectors of horizontal displacement, velocity and acceleration of the tower relative to the ground, respectively; x_m= horizontal displacement of the guynode; $[\bar{M}] = [M + M_a]$; $[M]$, $[C]$ and $[K]$= matrices of structural mass, damping and stiffness of the tower, respectively; $R(x_m)$= nonlinear restoring force of the guying system; $\dot{U}_g$ and $\ddot{U}_g$ are the horizontal ground velocity and acceleration; $[M_a]$= diagonal added mass matrix($= [\rho(C_m - 1)\nabla]$).

In this study, the nonstationary horizontal ground acceleration, $\ddot{U}_g(t)$, is modeled as a modulated stationary random process as[5,8]

$$\ddot{U}_g(t) = \varphi(t)\, r(t) \quad (2)$$

where $\varphi(t)$ is a deterministic envelope function, and $r(t)$ is a stationary random process having an autocorrelation function $R_{rr}(\tau)$. In the present work, $R_{rr}(\tau)$ is taken as the one corresponding to the filtered Kanai-Tajimi spectrum[8,9] (Figs. 2 and 3):

$$\begin{aligned} R_{rr}(\tau) &= e^{-\omega_g \zeta_g |\tau|}\{a_0 \cos(\omega_{D_g}|\tau|) + b_0 \sin(\omega_{D_g}|\tau|)\} \\ &+ e^{-\omega_1 \zeta_1 |\tau|}\{a_1 \cos(\omega_{D_1}|\tau|) + b_1 \sin(\omega_{D_1}|\tau|)\} \end{aligned} \quad (3)$$

where ω_g and ζ_g are the characteristic ground frequency and ground damping ratio; the constants ω_1 and ζ_1 are selected to obtain the required filter characteristics[3,8]; ω_{D_g} and ω_{D_1} are damped frequencies for ω_g and ω_1, respectively; a_0, b_0, a_1 and b_1 are constants determined from the given parameters S_0(constant power spectral density), ω_g, ζ_g, ω_1 and ζ_1, as in Ref. 12.

Equation 1 includes two nonlinearities. One is the restoring force of guying system and the other is drag term in modified Morison equation[7]. Based on the equivalent linearization technique[2,3,6] the linearized equation of motion against Eq. 1 can be obtained as

$$[\bar{M}]\{\ddot{X}\} + [\bar{C}]\{\dot{X}\} + [\bar{K}]\{X\} = -[\bar{M}]\{1\}\ddot{U}_g \quad (4)$$

where $[\bar{C}]$ are consists of the structural damping $[C]$ and the linearized hydrodamping; $[\bar{K}]$ includes the equivalent stiffness of the guyline system in addition to $[K]$. The linearized forcing term related to the ground velocity has been neglected in this study, since the effect is considered to be insignificant.

By using the natural mode superposition, $\{X\} = [\Phi]\{Y\}$, the uncoupled equations are ontained as:

$$\ddot{Y}_n + 2\zeta_n\omega_n\dot{Y}_n + \omega_n^2 Y_n = -\Gamma_n \ddot{U}_g(t) \quad (5)$$

where ω_n and ζ_n are the n-th mode frequency and damping ratio, respectively, and Γ_n is the n-th mode participation factor. The time dependent covariances of the modal coordinates Y and its time derivatives $\dot{Y}$, can be computed by using convolution integral as follows:

$$E[Y_m(t)Y_n(t)] = \Gamma_m\Gamma_n \int_0^t\int_0^t h_m(t-\tau_1)h_n(t-\tau_2)\varphi(\tau_1)\varphi(\tau_2)R_{rr}(\tau_1-\tau_2)d\tau_1 d\tau_2 \quad (6)$$

$$E[\dot{Y}_m(t)\dot{Y}_n(t)] = \Gamma_m\Gamma_n \int_0^t\int_0^t h'_m(t-\tau_1)h'_n(t-\tau_2)\varphi(\tau_1)\varphi(\tau_2)R_{rr}(\tau_1-\tau_2)d\tau_1 d\tau_2 \quad (7)$$

where $h_n(t)$ is the impulse response function for the n-th modal equation and $h'_n(\cdot)$ denotes the time derivative of $h_n(\cdot)$.

The deterministic envelope function is assumed to be expressed in terms of exponential functions[5] as

$$\varphi(t) = \frac{e^{-at} - e^{-bt}}{c} \quad b > a > 0 \tag{8}$$

where a and b are the constants which may be determined based on the measured earthquake records, and c is a constant adjusting the envelope function to have a peak of unity. Then, the double integrations in Eqs. 6 and 7 can be efficiently carried out, and analytical expressions of the covariance functions can be obtained as in Ref. 13.

For evaluation of the expected maximum response, the method proposed by Davenport[10] based on stationary processes have been widely used. In the present work, however, the expected maximum response for the nonstationary process is evaluated by using the approach by Shinozuka and Yang[11].

3. EXAMPLE ANALYSIS

Example analyses have been carried out for a hypothetical guyed tower that is assumed to be located at a site in 1000 ft water in the Gulf of Mexico (Fig.1). Structural properties for the tower and the restoring force of guying system are similar to those described in Reference 6. Dynamic analysis has been performed using the first four vibration modes of the tower. The corresponding natural frequencies were obtained as 0.234, 2.04, 6.28, and 13.7 rad/s (i.e., natural periods = 25.9, 3.08, 1.0, 0.46 seconds, respectively). The structural damping is assumed to be 5 percent for each mode. Dynamic responses are obtained for the top displacement, bottom shear force and bending moment at node 8.

The parameters for the earthquake spectrum were ω_g= 15.7 rad/s and ζ_g= 0.6, as suggested by Kanai[8] for a firm ground condition; and ω_1= 0.4 rad/s, ζ_1= 0.9 and S_0= 0.0459 $ft^2/s^3 \cdot rad$, which correspond to a class of earthquakes[3] having an average intensity similar to the N-S component of the 1940 El Centro earthquake. The duration is assumed to be 25 sec. The parameters of the modulating envelope functions for a given earthquake intensity are approximately evaluated based on two sets of earthquakes records. One is a=0.083, b=1.166 for Case 1 and the other is a=0.083, b=1.166 for Case 2 (Fig. 4).

Figures 5, 6, and 7 show the time varying variances of the tower responses for two different envelope functions with the same PSD. The limite case by setting the parameters for the envelope function as a=0 and b=1 has been investigated to demonstrate the validity of the nonstationary response analysis procedure in this study.

Table 1 summarizes the comparisons of the expected maximum responses obtained by two different methods, one is the frequency domain spectral method and the other is the time domain nonstationary method that has been developed in this work. It can be seen in general that the maximum responses using the nonstationary method are less than those by the ordinary spectral method approximately by 30 percents.

4. CONCLUSIONS

An efficient method for the nonstationary response analysis of offshore compliant structures under earthquake loading has been presented in this study. It is a time domain approach on the basis of convolution integral. By taking the auto-correlation function of the ground acceleration in terms of complex exponential functions of time, an analytical procedure is developed for the computation of the time varying variances of the tower responses. The expected maximum responses are also evaluated. It has been found that the maximum responses obtained by including the nonstationary effect, are significantly less than those estimated by the conventional spectral method based on the stationary assumption.

REFERENCES

1. Mangiavacchi,A., et al., Design criteria of a pile founded guyed tower, Proc. Offshore Technology Conference, Paper OTC3882, Houston, (1980).
2. Penzien,J., Kaul,M.K. and Berge,B., Stochastic response of offshore towers to random sea waves and strong motion earthquake, Computers and Structures 2, 733-756(1972).
3. Brynjolfsson,B. and Leonard,J.W., Response of guyed offshore towers to stochastic loads: time domain vs frequency domain, Engineering Structures 10, 106-116(1988).
4. Lin,Y.K., Nonstationary excitation and response in linear systems treated as sequences of random pulses, J. of Acoustic Society of America 38(3), 453-460(1965).
5. Shinozuka,M., Itagaki,H. and Hakuno,M., Dynamic safety analysis of multistory buildings, J. of Structural Div., ASCE 94(1), 303-330(1968).
6. Ryu,C-S. and Yun,C-B., Nonlinear stochastic analysis of guyed towers, Proc. 6th International OMAE Symposium, ASME, 53-59(1987).
7. American Petroleum Institute, Planning, designing and constructing fixed offshore platforms, API.RP2A, 15th ed.(1984).
8. Clough,R.W. and Penzien,J., Dynamics of structures, McGraw-Hill, 613-616(1975)
9. Kanai,K., Semi-empirical formula for seismic characteristics of the ground, Univ. Tokyo Bulletin Earthquake Research Institute 35, 309-325(1957).
10. Davenport,A.G., Note on the distribution of the largest value of a random function with application to gust loading, Proc. Inst. Civil Engineering 28, 187-196(1964).
11. Shinozuka,M. and Yang,J-N., Peak structural response to nonstationary random excitations, J. of Sound and Vibration 16(4), 505-517(1971).
12. Ryu,C-S., Dynamic response and reliability analysis of offshore guyed tower for wave and earthquake loads, Ph.D. Thesis, Dept. of Civil Engineering, Korea Advanced Institute of Science and Technology (1991).
13. Ryu,C-S. and Yun,C-B., Nonstationary response analysis of offshore guyed tower subjected to earthquake loadings, Proc. of International Society of Offshore and Polar Engineering, San Francisco, USA (1992).

Table 1. Expected maximum responses calculated by different methods

S_0 (ft^2/sec^3)	Methods	Deck Displ. (ft)	Base Shear ($10^3 kips$)	Moment at No.8 ($10^5 kips \cdot ft$)
0.0459	Nonst.-Case 1	2.920	2.11	5.29
	Nonst.-Case 2	3.196	2.37	6.00
	Spectral	4.821	3.23	8.38
0.1033	Nonst.-Case 1	4.078	3.09	7.71
	Nonst.-Case 2	4.492	3.48	8.76
	Spectral	6.616	4.72	12.2

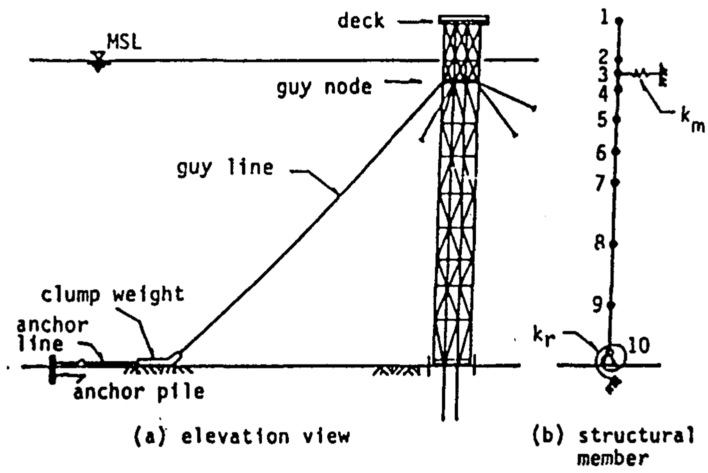

Fig. 1 Structural Configuration of Offshore Guyed Tower

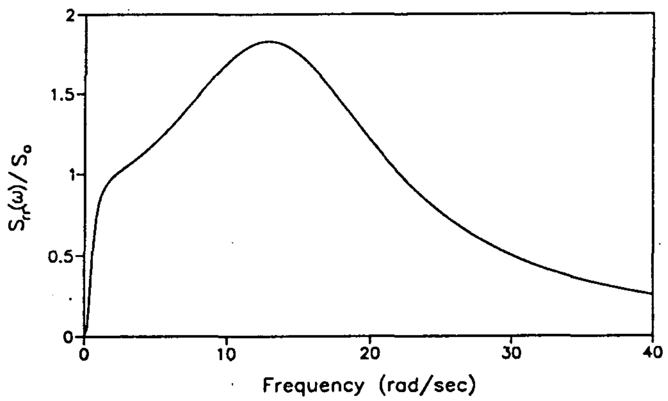

Fig. 2 Filtered Kanai-Tajimi Spectrum for Ground Acceleration

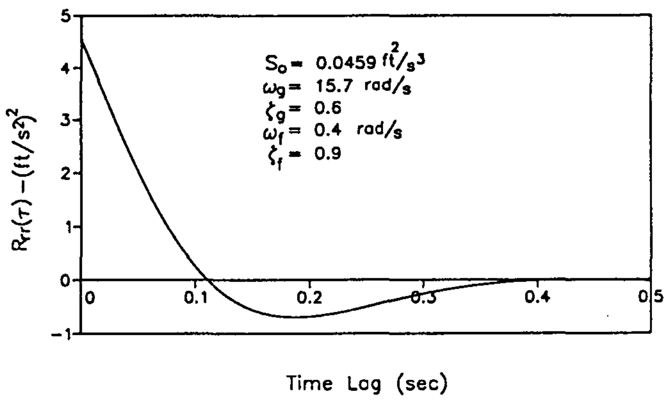

Fig. 3 Auto-correlation Function Corresponding to Filtered Kanai-Tajimi Spectrum

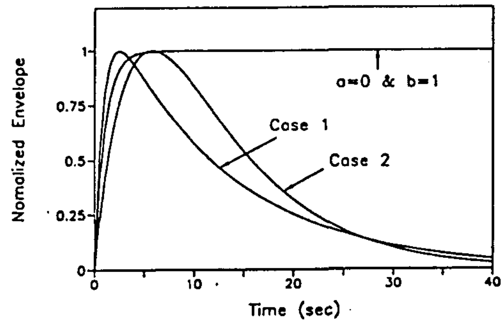

Fig. 4 Envelope Functions

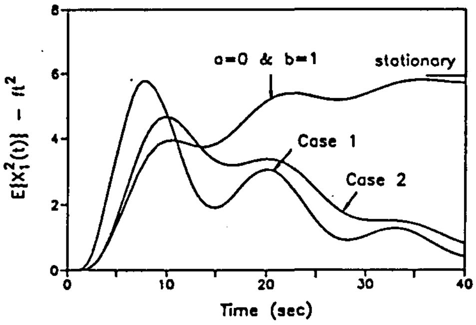

Fig. 5 Variance Functions for Deck Displacement

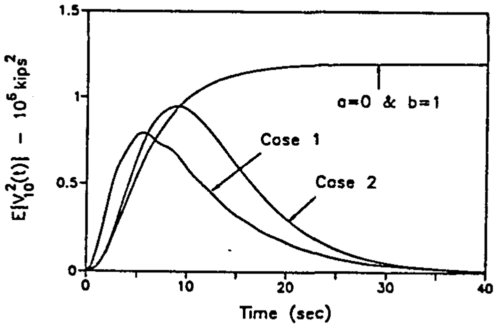

Fig. 6 Variance Functions for Base Shear

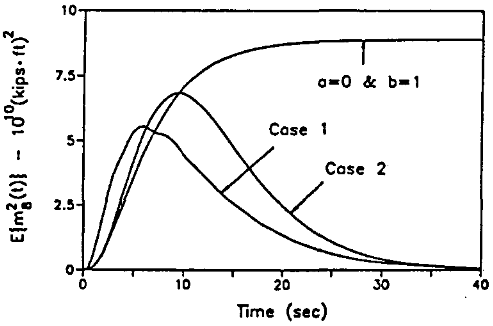

Fig. 7 Variance Functions for Base Moment