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Dynamic response of pile-cap structure under random sea wave action

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Abstract

Based on Pierson-Moscowitz (P–M) sea wave spectrum and linear wave theory, this paper presents the techniques for simulating random wave action on slender piles of pile-cap structure in coastal engineering, and discusses the dynamic response of pile-cap structure under random wave action using Finite Element (FE) method. In this study, a full FE model of a realistic pile-cap structure of a sea platform is established. The dynamic time-history analysis of the structure under the random sea wave action is carried out in consideration of two different structural damping types. As contrast, the static and dynamic analyses of the structure under the characteristic wave are also conducted. By comparing the displacement and internal force induced in the structure based on the different approaches considered, it is found that the dynamic response under the random sea wave is largest, which could have serious implications for design of structures of this type.

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Keywords: random wave action; FE method; dynamic time-history analysis; damping

1. Introduction

Pile-cap structure is a common coastal structure type, being widely used in harbors. For the structure, sea wave action is the main environmental load. Due to the complexity of motion mechanism of the wave, its action on structure shows randomness, as mentioned by Wen et al. [1]. Usually, the response of coastal structures under wave

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action can be calculated by static analysis method, spectral analysis method, and dynamic time-history analysis method. Borgman [2] conducted spectral analysis of ocean wave forces on piles, and Ou [3] also reported the method. However, with the progress of computer technology, dynamic time-history analysis is increasingly applied, on account of its more true reflection of structural dynamic response and the ability to simulate nonlinearity if necessary. In practical engineering, the characteristic wave is usually adopted to perform static calculation and make structural design, neglecting the randomness of wave and dynamic effect of the structure, which reduce the structure reliability. The paper will present the technique for simulating random wave action on slender piles of pile-cap structure in coastal engineering, and carry out dynamic or static analysis for a realistic pile-cap structure under the random wave or the characteristic wave by Finite Element (FE) method to conduct comparison among different approaches.

2. Simulation of the random sea wave

It is difficult to accurately express the motion of sea wave because of its complexity. In practice, wave energy spectrum is often utilized to the description of the wave. The spectrum is the wave energy distribution relative to its frequencies. At present, several wave spectrums have been obtained by measurement and the statistical analysis, such as Pierson-Moscowitz (P-M) spectrum, JONSWAP spectrum,. In the paper, P-M spectrum will be applied.

2.1. P–M spectrum

The P–M spectrum (single parameter spectrum) can be expressed as

$$S(\omega) = (0.78\omega^5)\mathrm{e}^{-3.11/(\omega^4 H_{1/3}^2)} \tag{1}$$

where ω is circular frequency, and $H_{1/3}$ is the average height of highest one-third wave. Suppose $H_{1/3}$ is assigned the value as the case in the upcoming section 4.1, and then the spectrum can be given as Fig. 1.

2.2. Simulation of sea wave surface elevation

The variation of wave surface's elevation is usually considered as a stationary random process. According to linear wave theory, namely Airy wave theory, the wave is made up of infinite number of cosine waves (usually take sufficient N waves into account) with different amplitudes, frequencies, and random phases (Borgman [4] and Yeh [5]), giving that

$$\eta(y,t) = \sum_{i=1}^{N} a_i \cos(k_i y + \omega_i t - \varepsilon_i) = \sum_{i=1}^{N} \eta_i(t) \tag{2}$$

where $\eta(y,t)$ is the instantaneous elevation of wave surface relative to still-water level for position y at time t ; a_i is the amplitude of the i-th harmonic wave; ε_i is the random initial phase conforming to the uniform distribution within the scope (0~2π); ω_i and k_i are the circular frequency and wave number of the i-th harmonic wave respectively, and they meet the dispersion relation, $\omega_i^2 = g k_i \tanh(k_i d)$, where g represents gravitational acceleration and d represents the distance between the still-water level and sea bottom.

In the simulation, the energy of the target spectrum is supposed to distribute within the scope $\omega_L \sim \omega_H$ (ω_L is lower limit of frequency and ω_H is the upper limit), and ignore the rest. Divide the frequency range into N equal sections with the length $\Delta\omega$ for each, and define the frequency of the midpoint of each interval as $\omega_i (i = 1,2,...,N)$. According to the energy relationship between harmonic waves and the wave spectrum, the amplitude of the i-th ($i = 1,2,...,N$) harmonic wave is obtained as

$$a_i = \sqrt{2S(\omega_i)\Delta\omega} \qquad \omega_i \in [\omega_L, \omega_H] \tag{3}$$

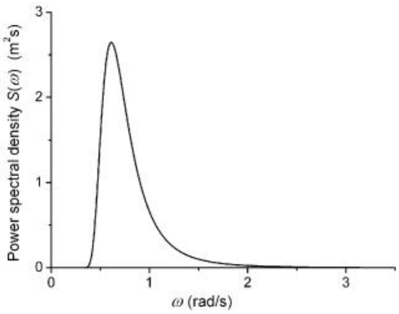


Fig. 1. P–M sea wave spectrum.

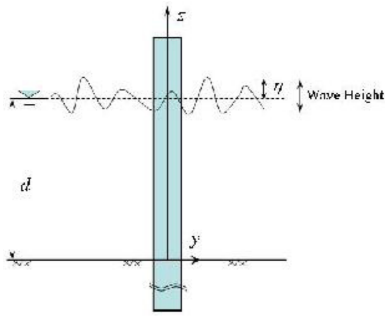


Fig. 2. Schematic drawing of the wave simulation.

2.3. Simulation of the random wave action on pile

Piles of pile-cap structure in coastal engineering generally can be classified as slender member (the ratio of pile's diameter to the wave length less than 0.2), so that the formula present by Morison [6] is suitable to calculate the wave force on piles. The Morison formula can be given in the form

$$f_{w1}(z,t) = f_D(z,t) + f_I(z,t) = \frac{1}{2}\rho_w C_D D u(z,t)\left|u(z,t)\right| + \frac{\pi}{4}\rho_w C_M D^2 a(z,t) \tag{4}$$

where $f_D(t)$ and $f_I(t)$ represents the resistance and inertial force respectively, C_M is the mass coefficient, C_D is the drag coefficient, D is the pile diameter, ρ_w is the density of water, $u(z,t)$ is the velocity of water particle, and $a(z,t)$ is the acceleration of water particle. Scholars have carried out a large number of theoretical and experimental researches to evaluate the coefficients C_M and C_D, and reliable values are provided based on several wave theories [7]. In condition of linear wave, set C_D =1.2 and C_M =2.0 for circular section pile. The water particle's velocity $u(z,t)$ and acceleration $a(z,t)$ can be expressed as

$$u(z,t) = \sum_{i=1}^{N} \frac{k_i g}{\omega_i} \frac{\cosh k_i z}{\sinh k_i d} \eta_i(t)\ ;\ a(z,t) = \frac{\partial u(z,t)}{\partial t} = \sum_{i=1}^{N} k_i g \frac{\cosh k_i z}{\sinh k_i d} \eta_i(t) \tag{5}$$

where $\eta_i(t) = a_i \cos(\omega_i t - \varepsilon_i)$, representing the i-th harmonic wave at a fixed point (y =0 in Eq. (2)); z is the distance from the point to the sea bottom (Fig. 2), and the other symbols have the same meaning as in Eq. (2).

Hence, Eq. (5) can be made use of to simulate the velocity and acceleration of water particle, however it is important to note that each harmonic wave should adopt the same random initial phase as in corresponding wave surface elevation simulation, and then the Morison formula can be applied to obtain the random wave force.

3. Dynamic analysis of the structure

3.1. Dynamic equilibrium equation for structure

Due to the sea wave action, pile elements submerged in water or not have different equilibrium equations. For the pile element above water, the static equilibrium equation is given as (Qu [8] and Clough [9])

$$EI(y'')'' + C_1\dot{y} + \bar{m}_1\ddot{y} = 0 \tag{6}$$

where y represents the horizontal deflection of the pile, $(y'')'' = \partial^4 y/\partial z^4$, $\dot{y} = \partial y/\partial t$, $\ddot{y} = \partial^2 y/\partial t^2$, EI represents the flexural stiffness of the pile's cross section, C_1 represents the damping coefficient of the structure, and $\bar{m}_1$ represents mass of the pile per unit length.

Additionally, the dynamic equilibrium equation of the submerged pile element is

$$(EIy'')'' + C_1\dot{y} + \bar{m}_1\ddot{y} = f_{w2}(t) \tag{7}$$

where $f_{w2}(t)$ is the Morison force imposed on the pile element, given as

$$f_{w2}(t) = \frac{1}{2}C_D\rho_w D\left|u-\dot{y}\right|(u-\dot{y}) + C_M\rho_w\frac{1}{4}\pi D^2(\dot{u}-\ddot{y}) \tag{8}$$

where $\dot{u}$ is the water particle's acceleration. Regardless of the coupling term of vibration velocity $\dot{y}$ and the water particle's velocity u in Eq. (8) [10], substitute it into Eq. (7) and then gives

$$(EIy'')'' + C_1\dot{y} + \frac{1}{2}\rho_w C_D D\left|\dot{y}\right|\dot{y} + \left[\bar{m}_1 + \frac{1}{4}\rho_w\pi D^2(C_M-1)\right]\ddot{y} = f_{w1}(t) \tag{9}$$

where the expression of $f_{w1}(t)$ refers to Eq. (4). The hydrodynamic damping in Eq. (9) is nonlinear, which lead to the difficulty to solve the equations, therefore, the approximate linearization method is normally adopted. Considering comprehensive damping effect here, define $C_1\dot{y} + (1/2)\rho_w C_D D\left|\dot{y}\right|\dot{y} = C\dot{y}$ and $\bar{m} = \bar{m}_1 + (1/4)\rho_w\pi D^2(C_M-1)$, where C is called as comprehensive damping, and then Eq. (9) can be rearranged into the linearization form

$$(EIy'')'' + C\dot{y} + \bar{m}\ddot{y} = f_{w1}(t) \tag{10}$$

Since most of the pile shaft is submerged in the water or influenced by the sea wave, the same form of the equilibrium equation is adopted for the whole pile shaft to avoid the non-proportional damping, and the structural damping is given referring to specification derived from experience with a combination of all factors.

3.2. Dynamic analysis method

This paper adopts two different methods to perform dynamic time-history analysis, with different damping forms and transient solving approaches, specific as follows:

(A) Consider added mass caused by hydrodynamic effect (Eq. (9)). Use Rayleigh damping and adopt the same damping ratio for different vibration modes. The structural transient analysis is performed by the full method.

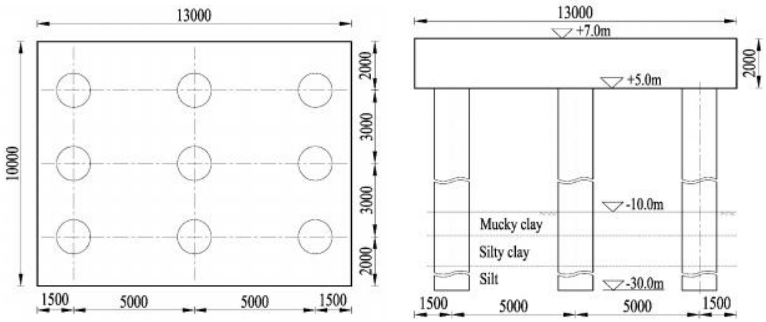
(B) Consider added mass of the structure. Use the constant damping ratio for the whole structure. The transient analysis is performed by the modal superposition method.

In the paper, ANSYS 13.0 is utilized to perform FE analysis. In the calculation, Newmark-β method is used to make numerical integral. In order to avoid the numerical noise caused by the numerical error in displacement solution of the high frequency part of the structure, the numerical damping is introduced, specific as β=0.2525 and γ=0.505, and also $\gamma = 0.5+\delta$, where δ is the attenuation factor and here δ=0.005.

4. Case study

4.1. General situation of the engineering example

It is a coastal operating platform using pile-cap structure, with planning size 13 m by 10 m, supported by steel tubular piles with the diameter 1400 mm. The structure's details referring to Fig. 3 below. The geologic condition of the engineering site and indicators of soil physical properties are given in Table 1. For the structure, m-method is used to simulate pile-soil interaction. Since m-value has big effect on piles' deformation, it should be given as accurate as possible according to soil parameters by the Chinese standard [11], specific as Table 1. The structural damping ratio is set as 0.05 recommend by the standard just mentioned, with additional hydrodynamic effect considered. To establish the FE model, the cap and piles are built by Shell 181 element and Beam 188 element respectively, the soil spring by Combin 14 element. The FE model is shown in Fig. 4.



(a) Planning (b) Front view

Fig. 3. Dimension of the pile-cap structure (Unit: mm).

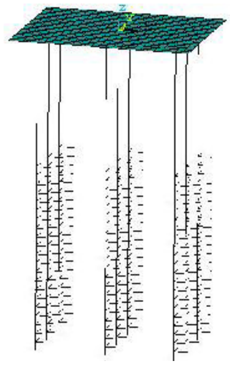


Fig. 4. FE model of the pile-cap.

Table 1. Stratigraphic distribution and indicators of soil physical properties.

Soil horizon	Bottom level (m)	Void ratio e	Liquidity index I_L	m-value (kN/m^4)
Mucky clay	-13	1.943	1.50	3000
Silty clay	-23	0.755	0.72	5500
Silt	-38	0.738	0.60	8000

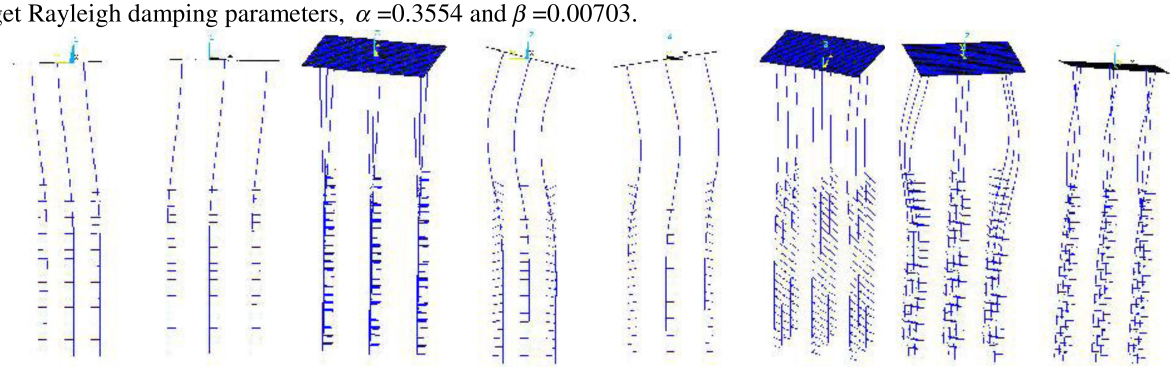
The static water level is 0.0m for the sea area where the project located in. Due to the high position of the cap, wave action can be neglected for it. For 50 years return period sea wave, key parameters are given in Table 2.

Table 2. Essential factors of the design wave.

Wave period T (s)	Wave length L (m)	Mean wave height H_m (m)	$H_{1/3}$ (m)	$H_{1\%}$ (m)
7.78	73.9	2.85	4.28	5.82

4.2. Dynamic property of the structure

Using Block Lanczos method to perform the modal analysis, with added mass considered, first 8 order natural vibration frequencies and vibration modes are obtained as in Fig. 5. Use the first two order natural frequencies to get Rayleigh damping parameters, α =0.3554 and β =0.00703.



(a) 1st order f_1=1.104 HZ (b) 2nd order f_2=1.160 HZ (c) 3rd order f_3=1.199 HZ (d) 4th order f_4=8.956 HZ (e) 5th order f_5=10.385 HZ (f) 6th order f_6=10.953 HZ (g) 7th order f_7=11.333 HZ (h) 8th order f_8=11.358 HZ

Fig. 5. First 8 order vibration modes.

4.3. Dynamic analysis based on random wave

The structure dimension is relatively small compared with the wavelength, so here neglect the space difference of wave action on different piles. The P-M wave spectrum curve is shown in Fig. 1 in the former section. In the simulation, the up-limit frequency spectrum is set as 3.14 rad/s (1.0 HZ), frequency interval equally divided into 1024 pieces, wave duration time set as 500 s, and the time interval set as 0.1 s. Time history of the random wave surface elevation simulated is shown in Fig. 6, and that of the Morison force is shown in Fig. 7 ($z = 0$ m, $z = -5$ m).

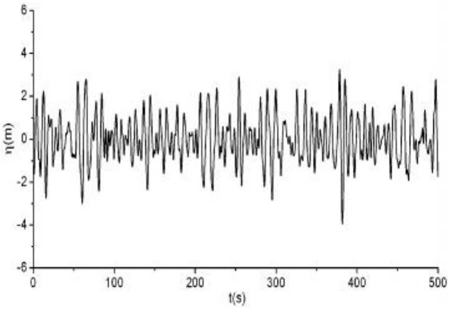
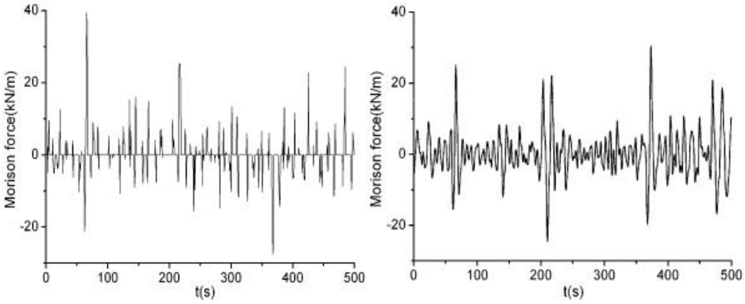


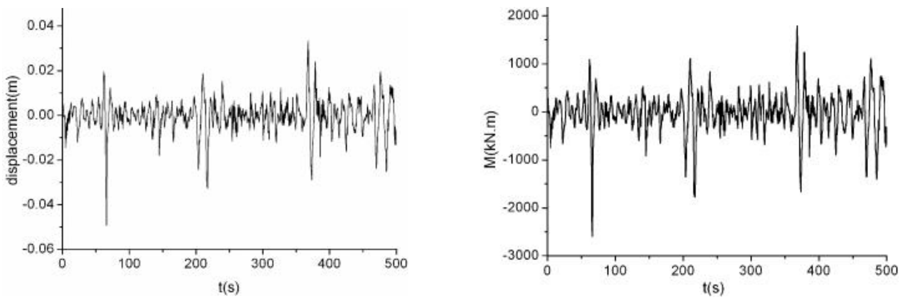
Fig. 6. Time history of the random wave surface elevation.



(a) z=0.0 m (b) z=-5.0 m

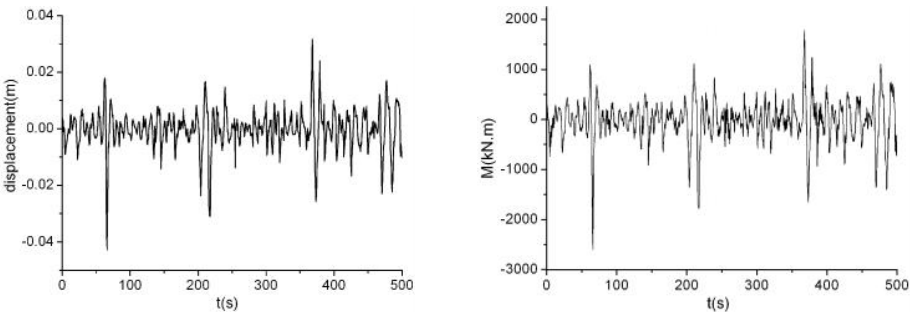
Fig. 7. Time history of the random Morison force.

Using simulated random wave load, transient analysis of the structure is performed, to give the dynamic response of the structure as shown in Fig. 8 (Rayleigh damping) and Fig. 9 (constant damping ratio).



(a) Horizontal displacement of the cap (b) Moment at the pile top

Fig. 8. Dynamic response of the structure under random wave (Rayleigh damping).



(a) Horizontal displacement of the cap (b) Moment at the pile top

Fig. 9. Dynamic response of the structure under random wave (constant damping ratio).

4.4. Dynamic analysis based on characteristic wave

In design, the characteristic wave is defined by 50 years return period wave with 1% cumulative probability. Here, design wave height $H_{1\%}$ =5.82 m, effective period of wave t =7.78 s. Time history of the characteristic wave surface elevation is shown in Fig. 10, and that of the Morison force shown in Fig. 11. The added mass is the same as in random wave, and Rayleigh damping is used in calculation. Dynamic responses are obtained as in Fig. 12.

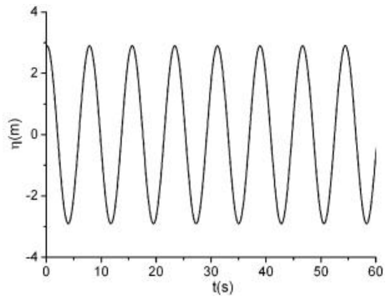
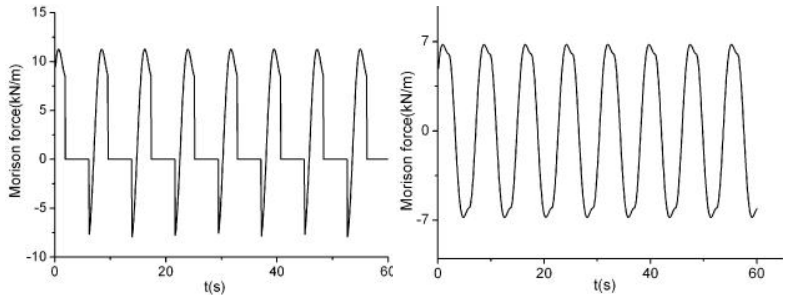
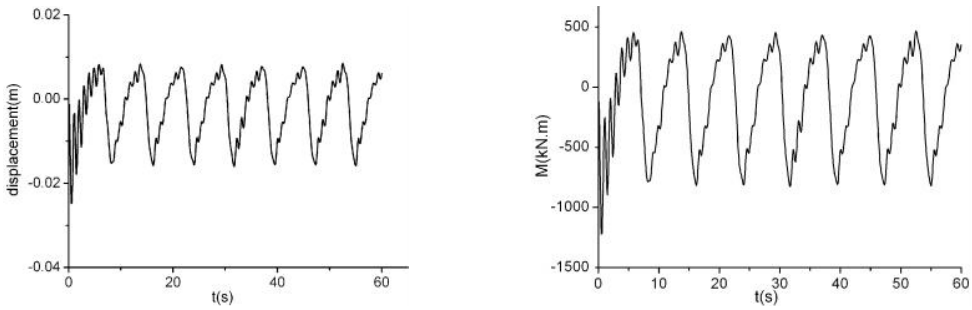


Fig. 10. Time history of the characteristic wave surface elevation.



(a) z=0.0m (b) z=-5.0m

Fig. 11. Time history of Morison force under the characteristic wave.

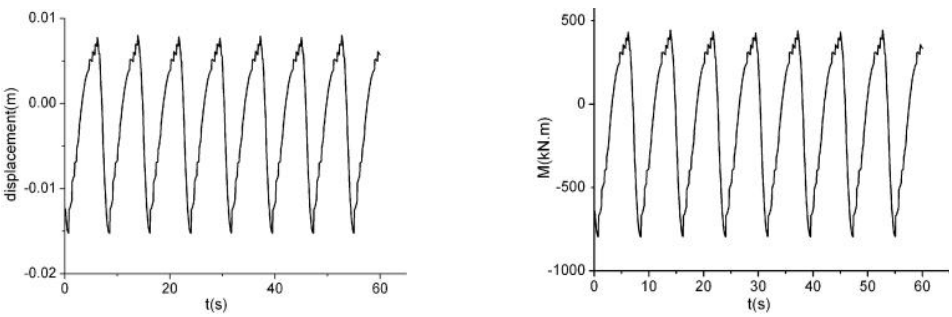


(a) Horizontal displacement of the cap (b) Moment at the pile top

Fig. 12. Dynamic response of the structure under the characteristic wave.

4.5. Static analysis

For static analysis, wave parameters are the same with the dynamic analysis based on the characteristic wave above. By static calculation, the results are given as Fig. 13.



(a) Horizontal displacement of the cap (b) Moment at the pile top

Fig. 13. Static response of the structure under the characteristic wave.

4.6. Calculation results summary

The main calculation results are summarized in Table 3. It can be concluded from the calculation results that:

(A) The displacement and internal stress obtained from static analysis result are the smallest, and that obtained from dynamic analysis based on characteristic wave are medium (1.7 times as the static result), and the biggest result come from the dynamic analysis based on random wave (about 3.0 times as the static result).

(B) As to dynamic analysis based on random wave, the calculation with the Rayleigh damping employed produces larger response than that with constant damping ratio, but the difference is little.

(C) As to dynamic analysis based on characteristic wave, the structural response is bigger when at the transient process stage, but only a little bigger than the static results when at stable stage process.

Table 3. Summary of the calculation.

Items	Static analysis	Dynamic analysis based on characteristic wave	Dynamic analysis based on random wave	
			Rayleigh damping	Constant damping ratio
Displacement (m)	0.015	0.025	0.049	0.043
Max. moment (kN.m)	795.6	1221.8	2592.2	2175.0
Max. stress (MPa)	25.8	39.5	83.9	70.4

5. Conclusion

The paper presents the technique to simulate the random wave action on slender piles of the pile-cap structure most common in coastal engineering, and studies the dynamic response of a realistic pile-cap structure under the random wave and the characteristic wave, which are also compared with static analysis results normally used in current structural design in China. Through the study, we get the following conclusions:

(1) Dynamic response of the cap-pile structure based on random wave is larger than that from the characteristic wave, and the response obtained from the static analysis is the smallest.

(2) Presently, static analysis method is mostly applied in design, without wave randomness and structural dynamic effect considered, which decrease the reliability and safety of structures. Especially in some cases when the dominant period of wave is close to the natural period of the structure, large dynamic response may generate. Therefore great importance must be attached to the problem in the structural design.

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