COMPDYN 2013
4th ECCOMAS Thematic Conference on
Computational Methods in Structural Dynamics and Earthquake Engineering
M. Papadrakakis, V. Papadopoulos, V. Plevris (eds.)
Kos Island, Greece, 12–14 June 2013

DYNAMICS OF MARINE STATIONARY PLATFORM UNDER ACTION OF HORIZONTAL SEISMIC LOADING

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Keywords: earthquake, horizontal seismic loading, marine stationary platform, deterministic and random motion, strength of piles

Abstract. *Dynamics of a fixed sea-based offshore platform under action of the horizontal seismic loading is investigated. The structure consists of a tube-like ferroconcrete rod, of an upper platform, and of a pile foundation. To describe the structure motion the beam-like model is used. Some deterministic and random models of the seismic loading are proposed. The depth of water is of the order 250 meters, the attached mass of water and its resistance is taken into account, the surface waves influence on the motion is not studied. The system of partial differential equation is reduced to the system with 3 degrees of freedom. The main attention is paid to the piles deformations and to their strength. Some numerical examples are studied.*

1 INTRODUCTION

Dynamics of a marine stationary platform under action of the horizontal seismic loading is investigated. The structure consists of a tube-like ferroconcrete rod. At its upper end the platform is attached. The similar structure is proposed in [1], where the foundation is modeled as a rigid body. Here a pile foundation is used to attach the rod with the ground. The depth of water is of the order 250 meters, the attached mass of water and its resistance is taken into account. Only the seismic loading is included at design in assumption, the surface gravity waves, the wind acting on the basic platform, the current, the ice loading, is not studied. The seismic loading is investigated also in [2], [3] but in this paper the more detailed analysis of foundation is proposed. The foundation stiffness is found as a result of the piles deformations. In contrast to [2], [3], where the concrete failure is assumed as the main reason of the structure destruction, here the strength of the entire structure is connected with the strength of piles. The deterministic and random models of seismic acceleration are used. Some numerical examples are studied.

2 MATHEMATICAL MODEL OF STRUCTURE

The structure consists of a tube-like iron-concrete rod *1* (see Fig. 1). At the upper end of rod a basis platform *2* is attached. A pile foundation *3* is used to attach the rod with the ground.

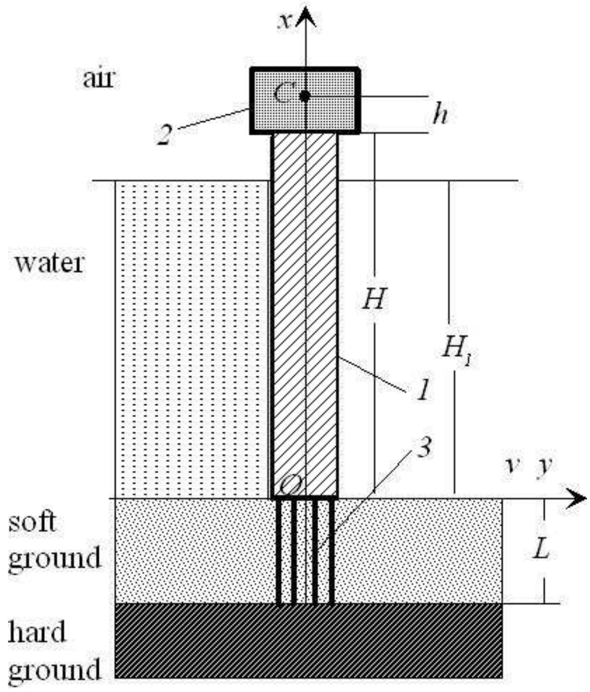

Fig. 1. The sketch of structure.

It is assumed that in limits of the foundation the ground acceleration does not depend on the space co-ordinates. The movable co-ordinate system, which moves with the given seismic acceleration $a(t)$ is accepted. To describe the rod bending a simple linear beam-like model is used

$$J\frac{\partial^4 v}{\partial x^4} + \frac{\partial}{\partial x}\left(P(x)\frac{\partial v}{\partial x}\right) + \rho(x)\left(\frac{\partial^2 v}{\partial t^2} + a\right) = f_v, \tag{1}$$

Here x $(0 \leq x \leq H)$ is the vertical coordinate; $v(x,t)$ is the horizontal deflection of the beam axis, t is the time; J is the bending beam stiffness; $P(x) = P_0 + \rho_0(H - x)g$ is the axial compression, $P_0 = m_0 g$; $g = 9.81$ is the gravity acceleration, m_0 is the mass of the basic platform, ρ_0 is the beam mass density per unit length; $\rho(x) = \rho_0 + \rho_1(x)$ is the mass density with the attached mass of water,

$$\rho_1 = \pi R^2 \gamma_w g, \quad 0 \le x \le H_1; \quad \rho_1 = 0, \quad H_1 < x \le H, \tag{2}$$

R is the beam radius, γ_w is the water mass density, dimensions H and H_1 are shown in Fig. 1; f_v is the resistance of water proportional to the square of relative water velocity V,

$$f_v = -C_v \gamma_w R V|V|, \quad V = \frac{\partial v}{\partial t} + V_0, \quad V_0(t) = \int_0^t a(t)\, dt, \tag{3}$$

C_v is the resistance coefficient.

The boundary conditions at the upper end $x = H$ of the beam are the transversal and rotational motions of the basic platform which is studied as a rigid body with the mass m_0 and the central inertia moment J_0,

$$m_0 \left(\frac{\partial^2 v_c}{\partial t^2} + a \right) = J \frac{\partial^2 \varphi_H}{\partial x^2} + P_0 \varphi_H, \quad J_0 \frac{\partial^2 \varphi_H}{\partial t^2} + m_0 h \left(\frac{\partial^2 v_c}{\partial t^2} + a \right) = -J \frac{\partial \varphi_H}{\partial x} + P_0 h \varphi_H, \tag{4}$$

where φ_H is the angle of axis inclination at $x = H$, and v_c is the horizontal deflection of the gravity center C (Fig. 1),

$$\varphi_H = \frac{\partial v_H}{\partial x}, \qquad v_c = v_H + h\varphi_H, \qquad v_H = v(H, t). \tag{5}$$

To formulate the boundary conditions at the lower end $x = 0$ we assume that the shear stress-resultant Q_0 and bending moment M_0, appearing at the contact of the beam with the pile foundation, are proportional to the lateral deflection v_0 and to the angle φ_0,

$$Q_0 = -J \frac{\partial^2 \varphi_0}{\partial x^2} - P(0)\varphi = c_{11} v_0 + c_{12} \varphi_0 = 0, \qquad M_0 = J \frac{\partial \varphi_0}{\partial x} = c_{12} v_0 + c_{22} \varphi_0 = 0. \tag{6}$$

The coefficients c_{ij} are to be found after examining the pile deformations.

3 THE BEAM STIFFNESS AND THE PILE DEFORMATIONS

The cross-section of the three layered ferroconcrete beam is shown in Fig. 2a (the steel layers are black and the concrete layers are spotted). The external radii of steel tubes are R and R_1 ($R_1 < R$), and the tube thicknesses are d and d_1. Between steel tubes there is a concrete layer ($R_1 \le r \le R - d$), containing N small steel tubes (in Fig. 2a $N = 8$), the polar co-ordinates r_k, θ_k of their centers are

$$r_k = R_0 = (R - d + R_1)/2, \quad \theta_k = \theta_0 + 2k\pi/N, \quad k = 1, 2, ..., N. \tag{7}$$

The radii and the thicknesses of these tubes are R_2 and d_2.

To calculate the beam bending stiffness J we accept the plane cross-section hypothesis [4] and take into account only the extension of material in the x direction. Then in linear approximation the stiffness J is

$$J = E_s \int\int_{S_s} y^2\, dS + E_c \int\int_{S_c} y^2\, dS,$$

where E_s and E_c are the elastic modules of steel and concrete, and S_s and S_c are the corresponding parts of cross-section occupied by steel and concrete. Calculations give

$$\begin{aligned} J = \ & \frac{\pi E_s}{4}\left(R^4-(R-d)^4+R_1^4-(R_1-d_1)^4\right)+\frac{\pi E_c}{4}\left((R-d)^4-R_1^4\right)+ \\ & \frac{N\pi R_0^2}{2}\left((E_s-E_c)R_2^2-E_s(R_2-d_2)^2\right)+\frac{N\pi}{4}\left((E_s-E_c)R_2^4-E_s(R_2-d_2)^4\right). \end{aligned} \tag{8}$$

The stiffness J does not depend on the angle θ_0 (see (7)).

For this cross-section the mass density ρ_0 is equal

$$\begin{aligned} \rho_0 = \ & \pi\gamma_s\left(R^2-(R-d)^2+R_1^2-(R_1-d_1)^2+N(R_2^2-(R_2-d_2)^2)\right)+ \\ & \pi\gamma_c\left((R-d)^2-R_1^2-NR_2^2\right), \end{aligned} \tag{9}$$

where γ_s and γ_c are the volume densities of steel and concrete.

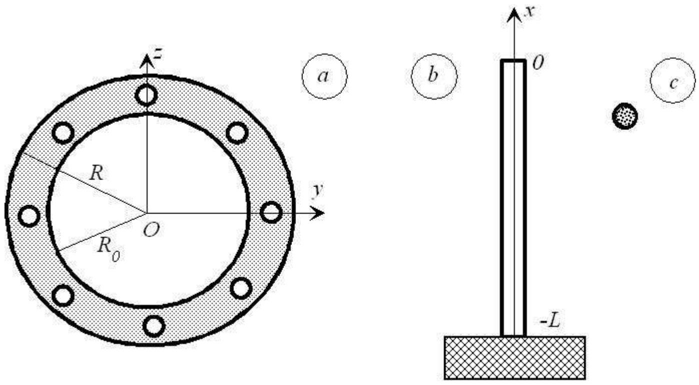

Fig. 2. (a) The cross-section of beam, (b) the pile, (c) the cross-section of pile.

For numerical calculations we take the following parameters given in SI. The dimensions of structure and radii of the tubes (see Fig. 2): $H_1 = 235$, $H = 250$, $R = 6$, $R_1 = 4.95$, $d = d_1 = 0.05$, $R_0 = 5.45$, $R_2 = 0.27$, $d_2 = 0.02$. The densities of water, steel, and concrete: $\gamma_w = 10^3$, $\gamma_s = 7.85 \cdot 10^3$, $\gamma_c = 2.2 \cdot 10^3$. The Young modules of steel and concrete: $E_s = 2.06 \cdot 10^{11}$, $E_c = 0.131E_s$. The parameters of body 2 (see Fig. 1): $h = 3$, $m_0 = 10^7$, $J_0 = 10^8$.

Then by using relations (8) and (9) for $N = 12$ we find $\rho_0 = 10^5$, $J = 1.1 \cdot 10^{13}$.

To find the coefficients c_{ij} in (6) it is necessary to study the piles deformations. The pile foundation is investigated in [5],[6],[7]. We use the simplest linear model in which the inertia of piles and their weight is neglected.

We assume that the pile deformations satisfy to the static boundary value problem

$$\begin{aligned} & J_p\frac{d^4v}{dx^4}+P_1\frac{d^2v}{dx^2}+k(x)v=0, \quad -L\le x\le 0, \\ & v(0)=v_0, \quad \left.\frac{dv}{dx}\right|_{x=0}=\varphi_0; \qquad v(-L)=\left.\frac{dv}{dx}\right|_{x=-L}=0, \end{aligned} \tag{10}$$

where L is the depth of the soft ground, J_p is the pile bending stiffness (see Fig. 2c),

$$J_p = \frac{\pi E_s}{4}\left(R_3^4-(R_3-d_3)^4\right)+\frac{\pi E_c}{4}(R_3-d_3)^4, \tag{11}$$

R_3 and d_3 are the radius of pile and its thickness; $P_1 = (P_0 + \rho_0 Hg)/N$ is the compressing force (we suppose that the force P_1 does not change during the studied time), $k(x)$ is the coefficient describing the lateral ground reaction. We accept that this coefficient linearly depends on x,

$$k(x) = -2R_3 c_g x > 0, \qquad 0.5 \cdot 10^6 \le c_g \le 10^7, \tag{12}$$

where c_g is the coefficient depending on the ground.

To find the numerical solution of the problem (10) we accept the following data: $L = 30$, $R_3 = 0.25$, $d_3 = 0.02$. Then for $N = 12$ we find $J_p = 2.386 \cdot 10^8$, $P_1 = 2.86 \cdot 10^7$, and solve the problem (10) for three values $c_g = 5 \cdot 10^5,\ 2 \cdot 10^6,\ 10^7$.

To formulate the boundary conditions (6) we seek dependence of the stress-resultant Q_p and stress couple M_p at $x = 0$ in the form

$$Q_p = b_{11} v_0 + b_{12} \varphi_0, \quad M_p = b_{12} v_0 + b_{22} \varphi_0. \tag{13}$$

After solution of the problem (10) we find coefficients b_{ij} for three given values of the ground stiffness coefficients c_g.

Table 1. Stiffness coefficients of pile.

c_g	b_{11}	b_{12}	b_{22}
$5 \cdot 10^5$	$0.555 \cdot 10^7$	$-2.004 \cdot 10^7$	$0.960 \cdot 10^8$
$2 \cdot 10^6$	$1.354 \cdot 10^7$	$-3.503 \cdot 10^7$	$1.319 \cdot 10^8$
10^7	$3.690 \cdot 10^7$	$-6.685 \cdot 10^7$	$1.852 \cdot 10^8$

The stretching pile density b_p is equal

$$b_p = \left(\pi E_s (R_3^2 - (R_3 - d_3)^2) + \pi E_c (R_3 - d_3)^2\right)/L, \tag{14}$$

and for the given parameters $h_p = 3.566 \cdot 10^8$.

Now we find the stiffness coefficients c_{ij} for the entire foundation with N piles,

$$c_{11} = N b_{11}, \quad c_{12} = N b_{12}, \quad c_{22} = N b_{22} + \frac{N}{2} R_0^2 b_p. \tag{15}$$

4 THE NATURAL FREQUENCIES AND MODES OF FREE VIBRATIONS

We seek the approximate solution of the boundary value problems (1), (4) and (6) as a partial sum of series of natural modes with the unknown coefficients $q_k(t)$ depending on time

$$v(x,t) = \sum_{k=1}^{K} V_k(x) q_k(t). \tag{16}$$

The natural modes may be found as a solution of the boundary value problem,

$$\begin{aligned}
&J \frac{d^4 V_k}{dx^4} + \frac{d}{dx}\left(P(x)\frac{dV_k}{dx}\right) - \omega_k^2 \rho(x) V_k = 0, && 0 \le x \le H,\\
&Q_k + c_{11} V_k + c_{12}\varphi_k = 0, \qquad -M_k + c_{12} V_k + c_{22}\varphi_k = 0, && x = 0,\\
&Q_k + m_0 \omega_k^2 V_{ck} = 0, \quad -M_k + \omega_k^2 \left(J_0 \varphi_k + m_0 h V_{ck}\right) + Ph\varphi_k = 0, && x = H,
\end{aligned} \tag{17}$$

where

$$\varphi_k = \frac{dV_k}{dx}, \quad V_{ck} = V_k + h\varphi_k, \quad M_k = J\frac{d\varphi_k}{dx}, \quad Q_k = \frac{dM_k}{dx} + P\varphi_k. \tag{18}$$

Here V_{ck} is the horizontal deflection of the mass center C of the body 2 (see Fig. 1).

The eigen functions $V_k(x)$ satisfy to the orthogonality condition ($\omega_k \ne \omega_n$),

$$\int_0^H \rho(x) V_k(x) V_n(x)\,dx + \left(m_0 V_{ck} V_{cn} + J_0 \varphi_k \varphi_n\right)_{x=H} = 0. \tag{19}$$

5 THE APPROXIMATE SOLUTION

The single non-linear term in the equation (1) is the water resistance f_v. We delete it and introduce the linear resistance including the water resistance and the resistance in the beam and in the ground [8].

To deliver equations for the unknown functions $q_k(t)$ in (16) we multiply equation (1) by $V_k(x)$ and integrate the result in the limits $0 \leq x \leq H$. Then by using the condition (17) we get the equations,

$$\frac{d^2q_k}{dt^2} + \omega_k^2 \left(\frac{\eta_k}{\omega} \frac{dq_k}{dt} + q_k \right) + \frac{f_k}{m_k} a(t) = 0, \quad k = 1, 2, , \ldots, K, \tag{20}$$

where

$$\begin{aligned} f_k &= \int_0^H \rho(x) V_k(x) dx + \left(m_0 V_{ck} + J_0 \frac{dV_k}{dx} \right)_{x=H}, \\ m_k &= \int_0^H \rho(x) V_k^2(x) dx + \left(m_0 V_{ck}^2 + J_0 \varphi_k^2 \right)_{x=H}, \end{aligned} \tag{21}$$

and η_k is the dimensionless resistance coefficient (we take $\eta_k = 0.1$), ω is the typical frequency. The term with η_k is included in (20) formally to take the damping into account.

Therefore to solve equations (19) it is necessary to find the natural frequencies ω_k and parameters f_k and m_k by solving the problem (17). We nornalize the eigen functions $V_k(x)$ by condition $m_k = 1$. Then only parameter f_k which characterize the level of excitation of the k-th mode remains.

For three values c_g and for the given parameters of platform from system (17) three first natural frequencies ω_k are found. These frequencies and the corresponding values $V_(ck)$ and Δw_k are given in Table 2. Here $\Delta w_k = R_* f_k \varphi_k(0)$, (with $R_* = 5.7$) is the measure of the pile extension corresponding to k-th vibration mode.

Table 2. Parameters of the vibration modes.

c_g	k	ω_k	V_{ck}	Δw_k
	1	1.233	−0.000370	0.0162
$5 \cdot 10^5$	2	3.205	−0.000545	0.00465
	3	6.678	−0.000705	0.000697
	1	1.362	−0.000388	0.0142
$2 \cdot 10^6$	2	3.906	−0.000585	0.00294
	3	7.347	−0.000730	0.000501
	1	1.429	−0.000401	0.0123
10^7	2	4.489	−0.000614	0.00201
	3	8.675	−0.000776	0.000871

As it is seen from Table 2 the values Δw_k quickly decrease with the number k of the vibration mode. The ground stiffness c_g effect on the parameters ω_k, V_{ck}, and Δw_k not very essentially, and we will study later only the intermediate case $c_g = 2 \cdot 10^6$.

6 MODEL OF THE SEISMIC GROUND ACCELERATION

As in [2], [3] we take the test horizontal ground acceleration in the form

$$a(t) = At^2 e^{-\alpha t} \sin(\alpha t + \theta). \tag{22}$$

The parameters in (22) satisfy to a restriction, which follows from the equality

$$\int_0^\infty a(t)dt = 0, \tag{23}$$

because in the opposite case the ground will move when the seismic impulse is finished. From (23) it follows that

$$\alpha(3\nu^2 - \alpha^2)\sin\theta + \nu(\nu^2 - 3\alpha^2)\cos\theta = 0. \tag{24}$$

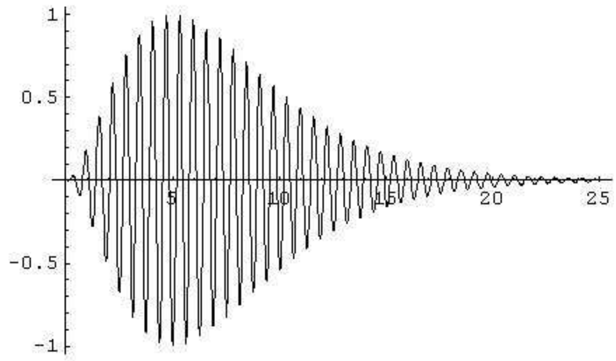

Fig. 3. The seismic impulse.

In Fig. 3 the seismic impulse with parameters $A = 0.295$, $\alpha = 0.4$, $\nu = 10$ is shown, and value θ is found from relation (24). For this impulse $\max_t\{a(t)\} = 1$.

We change impulse (22) by the following

$$a(t) = \tau^2 e^{2(1-\tau)} \sin(\nu t + \theta), \quad \tau = t/T; \quad \tan\theta = \frac{\alpha(3\nu^2 - \alpha^2)}{\nu(3\alpha^2 - \nu^2)}, \quad \alpha = \frac{2}{T}. \tag{25}$$

For impulse (25) $\max_\tau\{\tau^2 e^{2(1-\tau)}\} = 1$ at $\tau = 1$. Therefore, two parameters remain: the time T of amplitude growth, and the typical frequency, ν.

7 Numerical results. Discussion

We solve numerically equations (20) for $k = 1, 2, 3$, $c_g = 2 \cdot 10^6$ and for the various values T and ν, and find solution (16). The results are presented for the maximal ground acceleration $1\, m/c^2$. The problem is linear, and the results for the other acceleration levels may be obtaining by multiplying.

It occurs that the horizontal displacements of the upper body are less than $3 \cdot 10^{-5}\, m$, and details are not given here.

We analyze the strength of piles and find its maximal vertical displacements by relation

$$W = \max_t \left| \sum_{k=1}^{3} \Delta w_k q_k(t) \right|, \tag{26}$$

where Δw_k are given in Table 2, and $q_k(t)$ are the solutions of equations (16) with $f_k = m_k = 1$ and with the initial conditions $q(0) = dq(0)/dt = 0$.

The maximal vertical strains ε of piles are

$$\varepsilon = \frac{\Delta w}{L}. \tag{27}$$

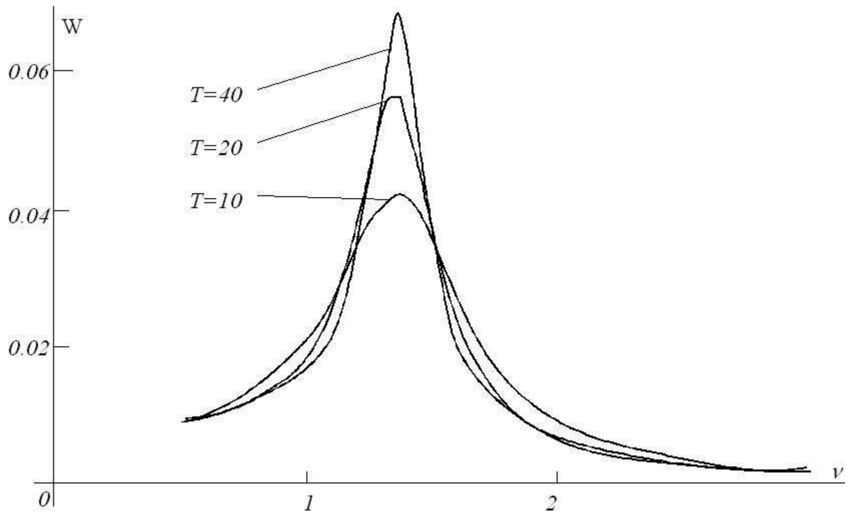

Fig. 4. The piles extension W.

The value ε is to be compared with the limiting values $\varepsilon_s = 0.00134$ and $\varepsilon_c = 1.16\varepsilon_s$ for a steel and a concrete respectively.

In Fig. 4 for three values $T = 10, T = 20$, and $T = 40$ dependences $W(\nu)$, $0.5 \leq \nu \leq 3$, are presented. It is seen the maximal value $W_* = \max_\nu\{W(\nu)\}$, is achieved at the first resonance $\nu \simeq \omega_1 = 1.362$. For example, for $T = 20$ the critical value A_* of the maximal seismic acceleration is

$$A_* = \frac{L\varepsilon_s}{W_*} = 0.7\,(m/c^2). \tag{28}$$

At the second resonance $\omega_2 = 3.906$ the value $\Delta w(\omega_2)$ is ten times smaller than W_*. The value Δw_* grows with T, and the limit,

$$\lim_{T\to\infty}\{W_*\} = \frac{\Delta w_1}{\eta\omega_1^2} = 0.0765, \tag{29}$$

what corresponds to the forced vibrations of system with one degree of freedom under the periodic excitation $\sin(\nu t)$. Indeed, for the system with one degree of freedom at $T \to \infty$ the maximal amplitude $W(\nu)$ is equal

$$W(\nu) = \frac{\Delta w_1}{\sqrt{(\omega_1^2 - \nu^2)^2 + \eta^2\nu^2\omega_1^2}} \tag{30}$$

and for $\nu = \omega_1$ we get relation (29).

8 Remark about the random seismic impulses

Methods of mechanical systems analysis under random excitation are described in [9], [10]. Random processes for earthquake simulation are described in [11]. Analysis of the structure shown in Fig. 1 under action of the random surface waves excitation is contained in [12], [13], [14].

The test seismic impulse (22) may give the upper estimation for the pile extension W because the resonance effect appears for the large T. It is interesting to compare the results of excitation (22) with the random excitation. In the exact statement the seismic acceleration may be studies as a non-stationary random process acting during the finite time $0 \leq t \leq T_0$. For simplicity we assume that the seismic excitation is a random stationary process time with the given spectral density $S_a(\omega)$ acting during the infinite time. Then the motion of structure also will be a random stationary process.

To find approximately the spectral density $S_W(\omega)$ of the pile extension W we study the system (20) with one degree of freedom,

$$\frac{d^2W}{dt^2} + \eta\omega_1 \frac{dW}{dt} + \omega_1^2 W + \Delta w_1 a(t) = 0, \tag{31}$$

with $\omega_1 = 1.362,\ \Delta w_1 = 0.0142$. Then [9], [10]

$$S_W(\omega) = \frac{(\Delta w_1)^2 S_a(\omega)}{(\omega^2-\omega_1^2)^2 + \eta^2\omega_1^2\omega^2}. \tag{32}$$

To compare with the previous results we normalize the process $a(t)$ so that its dispersion,

$$D(a) = \int_{-\infty}^{\infty} S_a(\omega)\, d\omega = \frac{1}{2}, \tag{33}$$

because the average value of function $\sin^2(\omega t)$ is equal to 1/2. Here $D(z)$ is the dispersion of the process $z(t)$,

As a test spectral density $S_a(\omega)$ we take

$$S_a(\omega) = \frac{\eta_0\nu^3}{2\pi((\omega^2-\nu^2)^2 + \eta_0^2\nu^2\omega^2)}. \tag{34}$$

Relation (34) satisfy to equality (32). Parameter ν is the typical frequency of excitation, and parameter η_0 describes the sharpness of curve. In Fig. 5 the curves $S_a(\omega)$ are shown for $\nu = 1$ and for two values of η_0.

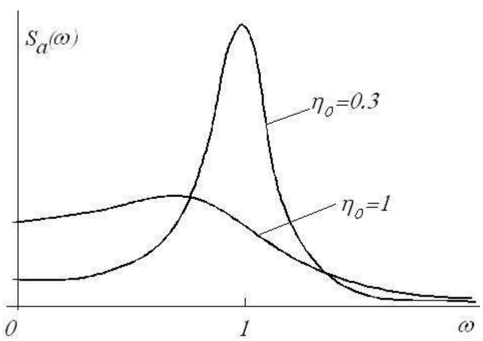

Fig. 5. The spectral density .

Function $S_a(\omega)$ is convenient to calculate the dispersion of the process W,

$$D(W) = \int_{-\infty}^{\infty} \frac{(\Delta w_1)^2 \eta_0 \nu^3\, d\omega}{\pi((\omega^2-\omega_1^2)^2 + \eta^2\omega_1^2\omega^2)((\omega^2-\nu^2)^2 + \eta_0^2\nu^2\omega^2)}. \tag{35}$$

After calculations we get

$$D(W) = \frac{(\Delta w_1)^2(\eta\omega_1^3 + \eta\eta_0\omega_1\nu(\eta\omega_1 + \eta_0\nu) + \eta_0\nu^3)}{2\eta\omega_1^3((\omega_1^2-\nu^2)^2 + \eta\eta_0\omega_1\nu(\omega_1^2+\nu^2) + (\eta^2+\eta_0^2)\omega_1^2\nu^2)}. \tag{36}$$

The measure of the vibration amplitude is a root-mean-square σ_W,

$$\sigma_W = \sqrt{D(W)}. \tag{37}$$

Consider some partial cases. At $\eta_0 \to 0$ the frequencies ω are concentrated near ν (see Fig. 5), and relation (37) gives

$$\sigma_W(\nu) = \frac{\Delta w_1}{\sqrt{2}\sqrt{(\omega_1^2 - \nu^2)^2 + \eta^2\nu^2\omega_1^2}}. \tag{38}$$

This relation almost coincides with relation (30). Factor $\sqrt{2}$ marks a natural difference between a root-mean-square and an amplitude.

For $\omega_1 = \nu$ we obtain the simple relation

$$\sigma_W(\omega_1) = \frac{\Delta w_1}{\omega_1^2}\sqrt{\frac{1+\eta\eta_0}{2\eta(\eta+\eta_0)}}. \tag{39}$$

To estimate the dependence $\sigma_W(\nu)$ on η_0 we compare the cases $\eta_0 = 0$ and $\eta_0 = 1$. The results are shown in Fig. 6.

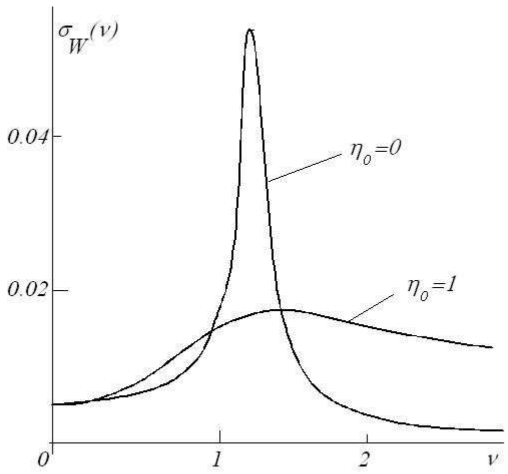

Fig. 6. The root-mean-square $\sigma_W(\nu)$ for $\eta_0 = 0$ and $\eta_0 = 1$.

It is seen in Fig. 6 that in the case $\eta_0 = 1$ the maximal amplitude is essentially smaller than for $\eta_0 = 0$.

9 CONCLUSIONS

Dynamics of a fixed sea-based offshore platform under action of horizontal seismic loading is studied. The main attention is paid to the strength of piles. The approximation with three degrees of freedom is used, but it occurs that only the first vibration mode is essential. Two test models for the ground acceleration are proposed. At the first of them the acceleration is presented as a function (Fig. 3) which allows to analyze the resonance effect. At the second model the seismic excitation is studied as a stationary random process. The presented results allow us to predict the possible behavior of structure under seismic loading

Acknowledgements

The paper is written with the financial support of Russian Foundation of Basic Researches, grants 12.01.92000.HHC-a, and 10-01.00244a.

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