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DETERMINATION OF LOAD-BEARING CAPACITY OF PILES USED IN STATIONARY OFFSHORE PLATFORMS

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A B S T R A C T

Fastening of hydrotechnical oil-gas mining facilities to seabed soils in Caspian Sea aquatoriums is usually carried out by pile foundations. Sustainability of strength and stability during the design and construction of hydraulic structures requires to solve a number of theoretical and practical problems. Numerous static and dynamic tests (experiments) were carried out in the Caspian Sea aquatorium and in laboratory conditions to solve these issues. The widespread use of pile foundations in the development of offshore oil and gas fields revealed the inconsistency of the domestic scientific methodological and regulatory framework for calculating their load-bearing capacity over the soil. It is established that the wave strikes, acting on pile foundations, interact with the surface design of offshore structures and offshore ground bases. Sea wave banging pile foundations creates vibrations in the pile foundation - topsides offshore structures and moving piles in subgrade. Displacement piles depend on the strength of the reaction between the structure and subgrade, the intensity of the shock wave in the time that passed through piles for offshore soil. At the same time, it takes into account the rheological properties of composite models of shelf soil.

Keywords: shelf; stress; ground; pile foundations; wave; marine structures; soil resistance.

1. Introduction

For the exploration and development of oil and gas fields located on the continental shelves, special hydraulic structures are being built on which the drilling equipment is located and the technological process is organized.

Offshore fixed platforms are mainly used in the drilling of exploration and production wells in the development of oil and gas fields in the sea. At the same time, special sites in the form of separate islands of a pile and large-block structure are being built for drilling oil and gas wells and placing equipment.

The foundations of the pile structure were widely used mainly in the exploration and development of coastal shallow-water offshore oil and gas fields. Their construction completely depends on hydro-meteorological conditions even in the coastal strip, protected from the prevailing storm waves. The main offshore oil fields are located in considerable distance from the coast, from the supply base, in areas with frequent strong winds and waves, due to which the use of island-type structures of the pile structure in the open sea was impossible.

In [1] we have considered the application of large section bored piles with a «hard core» to offshore structures, the influence of wave shocks on these constructions and interaction between a pile and shelf. It was found that the shock wave in constructions of pile foundations forms compression waves that the piles transfer to shelf soil, and this by its influence changes the density, velocity of motion of soil particles, deformation of soil. Compression wave on the pile surface may be of reflective, squeezing, flowing, absorbing and other interaction types and transferred to soil. Compression wave arises in soil when wave shock increases and is reflected during loading. Using the values of elastic and elastic plastic velocities of wave propagation in shelf soils, one can determine pressure increase, effective time of wave compression, and change of pressure in compression phase in time, maximum reflection pressure, and rare faction pressure in thickness of soil layer.

Oscillations of large section bored piles with a «hard core» by the action of a vertical force as a hard punch with a flat circular base, located on an elastic half-space for offshore structural analysis of offshore structures were considered in [2, 3]. It was found that under the influence of the pile loads, as a hard punch can perform harmonic vibrations around its axis and have different values of vertical displacements. The equations of motion of vertical piles have been compiled and solved. The amplitude of harmonic oscillations, a dynamic factor for the displacements at different values of the pile weight and the Poisson ratio shelf soil has been determined.

This article has a following structure: in the Introduction section we present an overview and motivation of our study, Results of research contain the main findings, which were summarized in the Conclusions section.

The works [4-6] cover the increase in reservoir oil yield, traditional technologies of cementing oil and gas wells, new

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achievements in well cementing, and theoretical and practical information about nanocolloids. Article [7] presents studies devoted to the problem of corrosion due to cracking of the cement body of micropiles. In [8], the main factors determining the load-carrying capacity of the anchor device and its structure are analyzed.In [9], a laboratory study is conducted to determine the behavior and variation of pile resistance in clayey soils.The article [10] analyzes the dynamic vertical loads of machine foundations on embedded structures.Article [11] discusses various methods for calculating pile foundations. Modeling of the emerging problem using a three-dimensional finite element model is considered in [12].In article [13], three-dimensional modeling is done using software. In article [14], 3D modeling was performed. The study [15] presents a quantitative analysis of eight commonly used methods and investigates the most appropriate and reliable interpretation method to be used for cast-in-situ piles.In the study [16], a three-dimensional numerical analysis is carried out to study the response of an underexpanded pile in clay to lateral loading. The study [17] conducted a comprehensive study on the optimal dimensions of a truncated cone vessel device.In [18] studies, three-dimensional analysis was carried out using the finite element method; the article published in [19] provides a study of the behavior of secant pile walls. In [20], an analysis is provided for calculating load and resistance coefficients for the design of energy piles; in [21], an analytical approach to predicting buckling taking into account the shape of hanging piles is proposed; and in [22], the prediction of pile displacement is shown. In the article [23], tests of inclined and vertical piles in liquefied soil under earthquake conditions were conducted on a shaking table. In [24], studies were carried out on the fields of temperature and soil humidity, frost heaving, and thawing settlement of the pile-soil system. In [25], the effect of flood load, statistical parameters, sliding pressure, and soil pressure on the anti-slide pile is investigated. A landslide was investigated in [26]. A series of numerical models of the landslide were developed using a three-dimensional finite element method. In [27], a theoretical analysis and a laboratory test of the model were conducted to study the mechanical behavior of the supporting structure of the subway station adjacent to the pile foundation. In [28], a new type of slope test system was developed and studied using simplified laboratory tests, taking into account the reinforcement effect of second-row piles. In the study of [29], pile displacements are calculated and compared with inclinometer device data. In [30], the main soil properties associated with plant growth in areas affected by iron mining activities are evaluated. In [31], experimental and numerical studies are carried out on the deformation response and storage mechanism of piles against sliding in clay conditions. In the study [32], the load-carrying capacity of helical piles in compression due to the effect of torque is studied.

In [33], the dynamic behavior of a group of piles with different loading directions is studied in a combined case, and machine-induced field excitation tests are carried out on small-scale hollow steel piles. In [34], influencing factors such as internal friction angle, pile diameter, hammer distance, and hammer quality are studied through appropriate calculation and analysis. The [35] study uses a two-stage method to analyze the behavior of an existing pile during the driving of adjacent piles. In [36], the effect of curing time on the frictional capacity of smooth concrete pipe pile-cemented soil was investigated through pile-soil interface tests. In [37], a vertical dynamic model of a large-diameter pile in frozen soil was established. The paper [38] investigates and compares the load taken by the shaft and tip for different pile group configurations tested in saturated soil and unsaturated soil developed under two conditions. In the study of [39], the dynamic pile-soil interaction in a liquefied field was investigated through numerical modeling and shaking table tests.

In [40], the influence of the slope on the horizontal bearing capacity of the pile is analyzed. In [41], the load-carrying curves, pile mechanical properties, and pile-soil stress properties of the composite foundation are investigated through field tests and numerical modeling. In [42], pipe piles with soil plugs under dynamic load are modeled. In [43], the influence of nonlinearity on the vertical impedance of the pile group is studied using numerical methods. In [44], a model for optimization evaluation is proposed to evaluate the seismic performance of slopes reinforced with a pile-anchor system. In [45], pile load transfer, foundation settlement, pile-soil stress distribution, and load distribution characteristics are investigated to determine the load-bearing capacity of a composite foundation. In [46], a model test was conducted to investigate the stress distribution of a sliding pile and the evolution of the soil arch during loading. The article [47] numerically investigates the structural forces in the tunnel lining after applying an adjacent loaded pile, considering the condition of the tunnel and soil layer and the different pile tips. In the study of [48], laboratory model tests were conducted to investigate the soil-pile interaction under lateral loads. In [49], the nonlinear response of a laterally loaded rigid pile in sandy soil is investigated. [50] uses finite element modeling based on linear elastic and elasto-plastic models for oil sand materials.

2. Results

The problem of wave interaction of offshore structures with shelf soil base and transmission of vibrations through large section piles with «hard core» is far from its solution in the volume determined by the requirements of offshore structures. At present, there is only a limited number of studies whose result may be used in engineering activity when building offshore structures.

For studying wave interactions of offshore structures with shelf soil it is necessary to study vibrations of complex dynamical systems such as: vibrations source; properties of shelf soils and construction of pile foundations by which vibrations are transmitted to soil foundations. It is

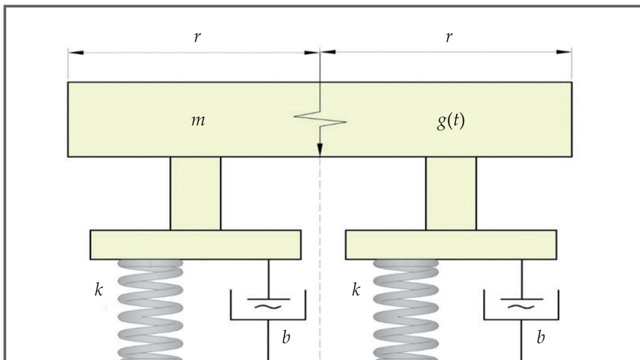

Fig. 1. Design schemes of offshore structures with combined rheological models on pile foundations

necessary to consider vibrations from strong wave impact to pile foundations, that do not cause damages and plastic deformations of the offshore structural element and vibrations of such structures including constructions of pile foundations occur in elastic stage of operation.

In the cases when vibrations from the force of wave impact in offshore structures and also pile foundations cause long-term undamped settlements, the level of foundations vibration and soil of the base may be estimated satisfactorily proceeding from the linear statement of the settlement of the pile foundation for a cycle of vibrations. Wave impacts acting on pile foundations interact with surface constructions of offshore structures and shelf soil bases. This time bored piles of large section with «hard core» play a role of connecting chain between shelf soils and upper construction of offshore structures forming a design model of half-space.

To estimate the shelf soil state from transmission of wave impact force through the pile foundations we use combined rheological model of shelf soil suggested by us in , when connecting the Kelvin and Maxwell bodies that allow to describe appearance and propagation of longitudinal, transversal and surface waves in shelf soil as systems for elastic half-space.

Thus, when studying interaction of offshore structure with soil base by bored piles of large section with «hard core» in harmonic model of vibrations from the force of impact wave is changed by rheological combined models suggested by us and that consist of elastic spring and Newton's damper (viscous damper) with characteristics dependent on the vibrations frequency. We accept the design scheme of offshore structure as was shown in figure 1, that offshore structures transmit harmonic vibrations to shelf soil base by pile foundations.

As we see from figure 1 one can neglect deformations of offshore structures to bored piles of large section with «hand core» compared with its displacement. We can express the displacement $W(t)$ by the following formula:

$$\left.\begin{aligned} m\frac{d^2W}{dt^2} &= q(t)-r(t) \\ W(t) &= \int_0^t r(t-t_1)W_{os}(t_1)dt_1 \end{aligned}\right\} \tag{1}$$

where $r(t)$ is the force of reaction between the structure and soil; $W_{os}(t_1)$ are initial displacements at time t_1. The system's vibrations satisfy the harmonic conditions:

$$q(t)=Qe^{i\omega t} \tag{2}$$

Then displacement in time t will take the form:

$$W(t)=Qf_0(\omega)e^{i\omega t}=Q[f_1(\omega)+if_2(\omega)]e^{i\omega t} \tag{3}$$

where ω is angular frequency of forced vibrations; i is imaginary unit. The function

$$f_0(\omega)=f_1(\omega)+if_2(\omega) \tag{4}$$

is a transfer function through the piles to shelf soils, $f_1(\omega)$ and $f_2(\omega)$ are its real and imaginary parts, respectively. On the other hand, the function $f_0(\omega)$ will be of the form:

$$f_0(\omega)=\int_{-\infty}^{\infty}W_0(t_1)e^{-i\omega t}dt_1 \tag{5}$$

$$f_1(\omega)=\int_0^{\infty}W_0(t_1)\cos\omega t_1 dt_1 \tag{6}$$

$$f_2(\omega)=\int_0^{\infty}W_0(t_1)\sin\omega t_1 dt_1 \tag{7}$$

$$W_0(t)=\frac{1}{2\pi}\int_{-\infty}^{\infty}f_0(\omega)e^{i\omega t}d\omega \tag{8}$$

here $W_0(t)=\frac{2}{\pi}\int_0^{\infty}f_1(\omega)\cos\omega t d\omega$; $W_0(t)=\frac{2}{\pi}\int_0^{\infty}f_2(\omega)\sin\omega t d\omega$.

The large section bored piles are accepted as a right circular stamp approximately equal to the mean integral displacement of the half-space surface within the circle loaded by the load from the own weight distributed in coordinate by the law of statical contact stresses and changing in time τ, from the force of sea wave and determined by a formula. Vertical displacements of a pile from the own weight of the structure and wave force will be:

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$$W_0(\tau)=-\frac{S}{\pi r^2\rho_0 c_1}\cdot\frac{1}{q}W_{00};\quad W_{00}=W_{01}+W_{02}+W_{03} \tag{9}$$

The components of Eq. (9) have the form:

$$\begin{aligned} W_{01} &= \begin{Bmatrix} D_0R(\theta,\tau),\ 0<\tau<2\theta \\ 0,\ 2\theta<\tau \end{Bmatrix} \\ W_{02} &= \begin{Bmatrix} \frac{1}{\pi}\int_{\frac{\tau}{2}}^{q}D_1(v)R(v,\tau)dv,\ 0<\tau<2q \\ 0,\ 2q<\tau \end{Bmatrix} \\ W_{03} &= \begin{Bmatrix} \frac{1}{\pi}\int_{\frac{\tau}{2}}^{1}D_2(v)R(v,\tau)dv,\ 2q<\tau<2; \\ 0,\ 2q<\tau\ ; \end{Bmatrix} \end{aligned} \tag{10}$$

In formulas Eq. (9) and Eq. (10) the following notations were introduced: r is the radius of large section bored piles (or radius of the stamp; ρ_0 is density of reinforced concrete of the pile as of a half-space; t is time, $c_2=\sqrt{G/\rho_0}$ is velocity of elastic longitudinal waves; $c_1=\sqrt{\lambda+2G/\rho_0}$ is velocity of elastic transversal waves in half-space; $\tau=c_2t/r$ is time interval; G is shear modulus of the pile; λ and G are Lame constants; $q=\frac{c_2}{c_1}=\sqrt{\frac{1-2\mu_0}{2(1-\mu_0)}}$; μ is Poisson ratio of soil; S is the value of pulse; $R(v,\tau)$ is reactive press are of soil under the pile toe (i.e. reactive pressure of soil under the lower end of the pile) may be determined by the following formula:

Table 1

The values of the coefficients n_1 and n_2 depending on μ_0

μ_0	0.00	0.05	0.1	0.15	0.2	0.25	0.3	0.35	0.4
n_1	1.215	1.207	1.200	1.195	1.193	1.197	1.211	1.243	1.317
n_2	1.094	1.113	1.132	1.149	1.163	1.173	1.177	1.169	1.141

$$R(v,\tau)=\int_{-\infty}^{\infty} r(t)e^{-i\omega t}d\omega \tag{11}$$

The parameters in Eq. (10) will be determined by the following expressions:

$$\left.\begin{aligned}
&D_0=\beta_R\left\{4\theta\left[2\left(1-2\theta^2\right)+2\alpha_R\beta_R+\frac{\theta^2\alpha_R}{\beta_R}+\frac{\theta^2\beta_R}{\alpha_R}\right]\right\}^{-1};\\
&\text{where}\quad \alpha_R=\sqrt{\theta^2-1};\quad \beta_R=\sqrt{\theta^2-q^2};\\
&D_1(v)=\beta_1\left[\left(1-2v^2\right)^2+4v^2\alpha_1\beta_1\right]^{-1};\\
&\alpha_1=\sqrt{1-v^2},\quad \beta_1=\sqrt{q^2-v^2};\\
&D_2(v)=4v^2\beta_2^2\alpha_1\left[-16\left(1-q^2\right)v^4+8\left(3-2q^2\right)v^4-8v^2+1\right]^{-1};\\
&\alpha_2=\sqrt{v^2-1};\quad \beta_2=\sqrt{v^2-q^2}
\end{aligned}\right\} \tag{12}$$

The function $W_0(\tau)$ will be of the form:

$W_0(\tau=0)=\dfrac{S}{\pi r^2\rho_0 c_1}$; $W_0(\tau=0)$ for $\tau=0$ and for $\tau>2\theta$.

Under the values of Eq. (12) the function $W_0(\tau)$ can be approximated as follows:

$$W_0(\tau)=-\frac{S}{\pi r^2\rho_0 c_2}\left(q\pi e^{-\frac{2\tau}{q}}+B\tau e^{-\beta\tau}\right) \tag{13}$$

The function Eq. (13) is a displacement of the upper part of offshore structures from the pile foundation under the action of instant pulse on this upper part of the structure. Hence it follows that when calculating structures for vertical vibrations from the pulse of wave interaction of pile foundations with shelf soil one can change a half-space simulating a soil by the combined rheological models with the indicated mechanical system with parameters of rheological model (Hook's spring and Newton damper b), where:

$$b=\pi r^2\rho_0 c_1;\quad \kappa=2\pi r\rho_0 c_1=Gr\frac{2\pi}{q^2} \tag{14}$$

The upper part of the system (table 1), i.e. the part over the pile foundations is accepted as a weightless plate with own weight m with acting pulse loads $q(t)$ interconnecting with soil foundations with combined rheological models having a spring of rigidity k and damper b models an effect created by transversal waves. The parameters b and k depend on the radius of bored piles of large section as of a stamp, the density of the material of half-space and velocity of transversal waves.

Pulse loads and own weight of the upper part create longitudinal vibrations to pile foundations and depend on the Poisson ratio μ_0 and motion of the stamp (pile foundation).

Pile foundations transmit acting loads to shelf soil bases and additionally acting wave transversal impacts and surface Rayleigh waves crease from the acceleration the motion of the upper part of the structure and this finds its reflection in the appearance of the mass m. For $\mu_0\to 0.5$ that corresponds to $c_1\to\infty$, the coefficients (parameters) $\kappa\to\infty$, $b\to\infty$ and the system approach to the system with one degree of independence. For the range of the values of the Poisson ratio μ_0 in the intervals $0\le\mu_0\le 0.4$ the function Eq. (9) may be approximated by the following formula:

$$W_0(\tau)=-\frac{S}{\pi r^2\rho_0 c_1}\cdot n_1 e^{-n_2\tau} \tag{15}$$

The values of n_1 and n_2 depend on the Poisson ratio of soil

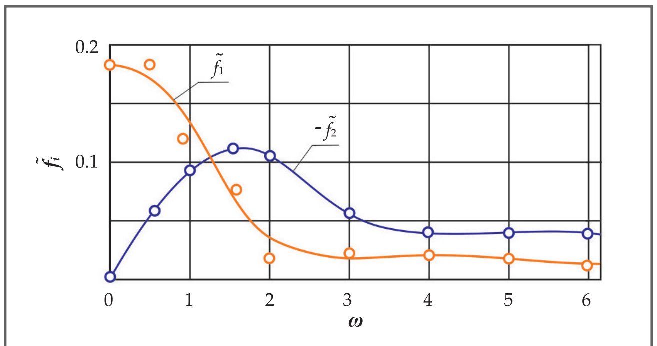

Fig. 2. Change of the function $\tilde{f}_1$ and $-\tilde{f}_2$ from the angular frequenc ω_0

under the pile toe (table 1) as in half-space, may be changed by parallel way connected rheological models suggested by us (combined) spring and damper whose parameters have the following values:

$$\kappa=\frac{4Gr}{1-\mu_0};\quad b=\pi r^2\rho_0 c_1 n_1^{-1}=\frac{4r^2\rho_0 c_2}{n_2\left(1-\mu_0\right)} \tag{16}$$

The indicated mechanical model Eq. (15) with the parameters of rheological models given in Eq. (16) was shown in figure 2.

To represent Eq. (15), in the graph given in figure 2 it is necessary to calculate the following transfer functions:

$$\tilde{f}_1=\left[1+\omega_0^2\frac{4}{n_2^2\left(1-\mu_0\right)}\right]^{-1};\quad -\tilde{f}_2=\frac{\omega_2}{n_2}\tilde{f}_1 \tag{17}$$

If we consider the totality of pulse loads with loads from the own weight of offshore structures and wave process forces as actions creating harmonic regime of vibrations of pile foundation lying in a half-space of soil base from shelf soils satisfying the combined rheological models suggested by us, that resist to parameters of the spring with rigidity $\kappa(\omega)$ and a damper with damping coefficients $b(\omega)$, connected in a parallel way shown in formula Eq. (16), perceive the indicated actions and ensure normal exploitation of structures.

Assume that a load from the offshore structure and vibrations of wave impact to shelf soil base consisting of medium sand, sand loam, fine sand, is transmitted through bored piles of large section (of diameter D=1200 mm), with «hard core» (the section of the core is 30×30 sm) and bears against more stable solid clays. Since bored piles run three layers and penetrate to the solid soil (solid clay) more than 2 meters, then the pile may be accepted as «built-in» rod in half-space. The state of foundations from 3 comparatively weak soils is accepted according to gauge rheological models from the Hooke spring and Newton damper, while solid clays as rigid connections of the pile (fig. 2) that have interrelations with offshore structures.

Harmonic vibrations $q(t)=Qe^{i\omega t}$ that cause displacements $w(t)$ happen from the totality of acting loads at time t from the own weight of the structure $q(t)$ and wave impact in shelf soil.

If we accept some parameters of the shelf soil and pile, more exactly: [the Poisson ratio for the shelf soil μ_0=0.25 (from table 1), the coefficients n_1=1.2; n_1=1.18 the density (specific weight) of the pile from the reinforced concrete ρ_0=25 kN/m³ time t=1hour, radius of the bored pile r=0.6m velocity of elastic longitudinal waves $c_1=\sqrt{\lambda+2G/\rho_0}=$

=1000 m/s, velocity of elastic transversal waves $c_2=\sqrt{G/\rho_0}=425$ m/s, for the frequency of vibrations ω=35Hz where G and λ are Lame constants (G is pile's shear modulus, i.e. $G=E/2(1+\mu)$, $E=3.25\cdot10^4$ MPa is modulus of elasticity of the pile made of the concrete B30 and $\mu=0.3$ is the Poisson ratio of the pile: ratio of the velocities of waves $q=c_2/c_1=425/1000=0.425$ time interval $\tau=c_2\ t/r=1.97$ hour), we can determine the vertical displacement of the pile in shelf soils.

We can accept the value of the velocities of elastic waves from table 2 for different vibration frequencies:

Table 2
Dependence of the velocity on angular frequency

Vibration frequencies ω, Hz	35	40	45	50	55	60
c_1, m/s	1000	1143	1286	1429	1571	1714
c_2, m/s	425	486	546	607	668	729

For the value of the angular frequency of the pile $\omega_0=1.0$, for which the transfer functions of load from offshore structures through bored piles have the greatest values on shelf soils (see fig. 2), we accept $\tilde{f}_1=0.12;\ \tilde{f}_2=0.09$. If we accept the value of the pulse $S=1.0$, the sum of the components of displacements $W_{00}=W_{01}+W_{02}+W_{03}=3$, then vertical displacements of the pile from the own weight of structures and impact force of wave in time $\tau=1.97$ hour, by formula Eq.(9) will be:

$$W_0\left(\tau=1.97hour\right)=-\frac{S}{\pi r^2\rho_0 c_1}\cdot\frac{1}{q}W_{00}=-\frac{1}{3.14\cdot0.6^2\cdot25\cdot1000}\cdot$$
$$\cdot\frac{1}{0.425}\cdot3=-0.00025\,m=-0.25\,mm$$

As is seen, vertical displacements of the pile $W_0(\tau=1.97\,\text{hour.})=-0.25$ mm are negligible. So, wave interaction of offshore structures with shelf base through large section bored piles with a «hard core» have sufficient stability.

Consider the bearing capacity of piles used in the construction of marine island foundations. The load-carrying ability of the pile consists of N_{fric} (friction along lateral surfaces) and N_k (pressures on the shoe sole and end of the pile)

$$N=N_{fr}+N_k \tag{18}$$

The load-carrying ability of the pile along the lateral surface N_{fr} in practice of building of offshore island foundations is determined by the formula:

$$N_{fr}=\frac{\gamma_p U tg\varphi l^2}{2} \tag{19}$$

Here γ_p is volumetric weight of cement or cement-sandy solution; U, l is perimeter and length of a precast pile; φ is internal friction angle of soil.

The performed experimental works on determination of load-carrying ability of precast piles in ordinary conditions showed consistency of formula Eq. (19) and at present it is used when designing offshore island foundations. Formula Eq. (19) is used for a great length piles. This is justified by the fact that the load-bearing ability of the end of the pile is negligible in comparison with lateral surface. However, the results of experiments carried out in earth conditions can not give complete answers for precast piles used in offshore conditions.

For these piles when defining load-carrying ability, in addition to loading of soil load, the loading from the mass of water should also be taken into account.

In this case, influence of load-carrying ability of the end, even for not long piles may be essential. In view of the fact that the end is a circular section in a plan, the end of a pile may be considered as a rigid stamp with a circular contour. The solution of the problem of ultimate resistance of bases under circular rigid stamp is given by [51]. He has obtained a formula for mean intensity of pressure under the bottom of rigid circular stamp in the form:

$$P_{me}=(A_\gamma\gamma a+B_q q)(1+sin\varphi) \tag{20}$$

where a is a radius of a circular stamp; γ is volumetric weight of soil; q is intensity of uniformly distributed load; A_γ, B_q are the coefficients connected with internal friction angle of soil.

For the case considered by us, in formula Eq. (20), the first expression in the first bracket need not to be taken into account Justification for it is that usually in building offshore island foundations the precast piles of length more than 10m and of diameter no more than 0.5 mare used.

So, for example, for a soil with 30° internal friction angle, the pressure intensity under the bottom of pile of length 10 m and of diameter 0.5 m when taking the first term in the bracket into amount, equals 3030 kN/m^2. As is seen the difference is less than 5%.

Based on what has been said, the value of the load-bearing ability of the pile end is obtained in the form:

$$N_k^{sv}=qF_{sv}B_q q(1+\sin\varphi) \tag{21}$$

$$B_q=\frac{e^{\pi tg\varphi}}{3(1-\sin\varphi)}\left(1+M^{\omega}e^{\frac{\pi}{2}tg\varphi}\right) \tag{22}$$

here $M=\dfrac{1+\sqrt{2}e^{\left(\frac{\pi}{4}-\frac{\varphi}{2}\right)tg\frac{\varphi}{2}}/\sin\left(\frac{\pi}{4}-\frac{\varphi}{2}\right)}{1+e^{\left(\frac{\pi}{2}-\frac{\varphi}{2}\right)tg\frac{\varphi}{2}}/\cos\frac{\varphi}{2}};\ \ \omega=2tg\varphi tg\left(\frac{\pi}{4}-\frac{\varphi}{2}\right)$

As in earlier used constructions of offshore structures the pad was connected with the pile and takes on itself a part of the load, it is necessary to define its load-bearing ability. The pad may be considered as a quadratic rigid stamp with a circular hole.

The solution of such a problem does not exist in references. The exact solution of the problem on calculation of ultimate resistance of soil under pressure, transmitted along the area of the square with a circular hole cannot be obtained by means of the method worked out for an axisymmetric problem. However, for the case under consideration it is possible to state soma arguments on approximate estimation of the size of ultimate pressure.

Observations for the process of formation of condensated core of the soil under rigid stamps, with concentrated load, show that under gradual pressure increase there happens redistribution of pressure along the bottom of the stamp.

In spite of the fact that pressure distribution along the bottom of the quadratic stamp differs from pressure distribution for a circular stamp, the stress state at the moment of limit equilibrium is rather close to each other.

Based on what has been said, we accept the considered construction of the pad as an annular section stamp. Consequently, the problem is reduced to definition of ultimate value of pressure symmetric with respect to axis and transmitted to the area of annular ring.

Distribution of ultimate pressure of annular section circular stamp was given by [51] in connection with caisson and movable wells. The character of pressure distribution for caisson and movable wells will be identical also for the construction of the pad with a difference that bulging out or shear of soil may be directed not only into the well as was observed for movable wells, but outwards. This is explained by the presence of sufficient load outside the well, for a caisson and for a pad, the existence of a pile.

Change of intensity of ultimate pressure in thickness of the ring will have the same form for a pad, but this time the greatest value of pressure will be on the contours of circular hole of the pad.

For a such a statement of the problem, the calculation formulas for determining intensity of ultimate pressure obtained by [51] preserve their form. The intensity of ultimate pressure for the points a and b has the form:

$$\begin{gathered} p_a = F_q q; \;\; p_b = D_\gamma \gamma_b + E_q q; \\ F_q = e^{\pi tg\varphi} tg^2\left(\frac{\pi}{4}+\frac{\varphi}{2}\right) \end{gathered} \tag{23}$$

$$\begin{gathered} D_\gamma = \frac{n+1}{\lambda-1}\left[\left[1-M_1^{\lambda-1}+(\lambda-1)M_1^{\lambda}M_2 e^{\pi tg\varphi+\frac{J\lambda}{2\cos\varphi}}\right] tg\left(\frac{\pi}{4}+\frac{\varphi}{2}\right)\right]; \\ E_q = M_1^{\lambda} e^{\pi tg\varphi+\frac{J\lambda}{2\cos\varphi}} tg^2\left(\frac{\pi}{4}+\frac{\varphi}{2}\right); \;\; \lambda = 2tg\varphi tg\left(\frac{\pi}{4}+\frac{\varphi}{2}\right); \\ J = \int_{\frac{\pi}{4}-\frac{\varphi}{2}}^{-\left(\frac{\pi}{4}+\frac{\varphi}{2}\right)} \frac{\sin(\delta+\varphi)-\cos\delta}{\frac{n}{N_1}e^{-\delta tg\varphi}-\sin(\delta+\varphi)}d\delta; \;\; N_1 = \frac{e^{\left(\frac{\pi}{4}+\frac{\varphi}{2}\right)tg\varphi}}{2\cos\left(\frac{\pi}{4}+\frac{\varphi}{2}\right)}; \\ M_1 = \frac{n+1}{n+\frac{1}{2}}; \; M_2 = \frac{1}{n+1}\left[n-\frac{1}{2}e^{\frac{\pi}{2}tg\varphi}tg\left(\frac{\pi}{4}+\frac{\varphi}{2}\right)\right]; \\ n = e^{\frac{\pi}{2}tg\varphi}tg\left(\frac{\pi}{4}+\frac{\varphi}{2}\right) \end{gathered} \tag{24}$$

where b is the length of the edge of the pile to the edge of the pad; q is intensity of uniformly distributed load.

The integral I may be calculated by one of the approximate inetgration methods.

Neglecting influence of angels of pads based on formula Eq. 23 we get load bearing ability of the pad's bottom in the form:

$$N_k^b = \pi b\left[D - \frac{(p_b+2p_a)b}{3(p_b+p_a)}\right]\frac{p_b+p_a}{2} \tag{25}$$

where b is distance from the edge of the pad to the pile's contour; D is the size of sides in the plan of square pad.

In formulas Eq. (21) and Eq. (23) the value q is a load distributed uniformly outside of the piles contour and pad. The greater the size of this load, the greater is the load-bearing ability of the piles construction. The load for a pile in ordinary conditions in civil engineering is a load lying above and surface of a pile, for offshore field and similar constructions, soil and water.

Consequently, the load for the considered constructions may be a soil of thickness l lying above the end surface and water of depth H. Hence from formulas Eq. (22) and Eq. (23) with regard to what has been said we write

$$N_k^{sv} = F_{sv}B_q(\gamma l + \gamma_b H)(1+\sin\varphi) \tag{26}$$

$$p_a = \gamma_q H F_q; \;\; p_b = D_\gamma \gamma b + E_q \gamma_b H \tag{27}$$

There H is the sea depth; l is the thickness of the soil layer calculated from the surface of the pile's end or the length of the pile in soil; γ, γ_b is volumetric weight of soil and water, respectively.

Substituting in formula Eq. (18) the values for N_{fric} and N_k found by formulas Eq. (19), Eq. (24) and (Eq. 26) we obtain load-bearing ability of the considered construction of the pile in the form:

$$\begin{gathered} N = \frac{\gamma_p U tg\varphi l^2}{2} + F_{sv}B_q(\gamma l+\gamma_b H)(1+\sin\varphi) + \\ +\pi b\left[D - \frac{b(p_b+2p_a)}{3(p_b+p_a)}\right]\frac{p_b+p_a}{2} \end{gathered} \tag{28}$$

So estimate the efficiency of the increase of load-bearing ability of the pile used in offshore island structures, we give such an example.

The leg of the foundation for the sea depth 10 m is subjected to calculated vertical load 2000 kN. The leg of the block (pile) rests against the soil with the internal friction angle φ=30° and volume weight γ=19 kN/m^3, volume weight of water γ=10 kN/m^3, volume weight of cement solution γ=18 kN/m^3.

In soil l=10 m the pile has diameter d=0.325 m, perimeter U=1.02 m, the square of cross section F=0.083 m^2, the size of the sides of the pad of square B=1.5 m, the distance from the edge of the pad to the pile's contour b=1/2(D-d)=0.587 m.

The value of coefficients B_q, F_q, D_γ, E_q we determine by formulas Eq. (22) and Eq. (24): B_q=20.2; F_q=18.3; D_γ=42.8; E_q=35.4.

We calculate the ultimate pressure intensity by formula Eq. (27):

$$p_a=\gamma_q H F_q=10\cdot 10\cdot 18.3=1830 \text{ kN/m}$$
$$p_b=D_\gamma\gamma_b+E_q\gamma_b H=42.8\cdot 19\cdot 0.587+35.4\cdot 10\cdot 10=4020 \text{ kN/m}$$

The determine the load-bearing ability of the considered construction by formula Eq. (28):

$$\begin{gathered} N = \frac{18\cdot 1.02\cdot 0.576}{2}\cdot 10^2 + 0.083\cdot 20.2(19\cdot 10+10\cdot 10+3.14\cdot \\ \cdot 0.587\left[1.5-\frac{4020+2\cdot 1830}{4020+1830}\times\frac{0.587}{3}\right]\frac{4020+1830}{2} = \\ = 530+730+6700 = 7960 \text{ kN} \end{gathered}$$

As is seen from the example, the critical load mainly increases at the expense of the account of the work of the pad in the pile construction. When determining load bearing ability of the pad during mounting of the block, the first two terms are not taken into account in formula Eq. (28). Formula Eq. (18) may be used in the case when the washing-outs under the pad by the ground flows will be excluded.

Taking into account that the long term practice established the fact that of washing of the pad and absence of mechanical relations of a pad with a pile in offshore foundations constructions produced now, the load-bearing ability of the pile should be realized by the following formula not taking into account the work of the pad:

$$N = \frac{\gamma_p U tg\varphi l^2}{2} + F_{sv} B_q \left(\gamma l + \gamma_b H\right)(1 + \sin\varphi) \quad (29)$$

In practice of buildings of offshore island structures the calculation of piles is performed only for friction neglecting load-bearing ability of the pile's end. As was shown in the example, for a pile of length 10 m the load-bearing ability in friction (N=530 kN) in 1.37 times less than of the end of the pile $N_k^{sv} = 730\,\text{kN}$. This difference will increase with increasing the sea depth. So, for example, for the same length of a pile, for sea depth 20 m, the load bearing ability of the piles end will have the quantity $N_k^{sv} = 980\,\text{kN}$.

Conclusions

Wave impacts acting in pile foundations interact with surface constructions of offshore structures and shelf soil bases. This time we can use a combined rheological model of shelf soil with harmonic vibrations on rod systems. We can use bases of shelf soils foundations as a model of a half-space connected with combined rheological models consisting of Hook's spring and Newton damper in a parallel way. The main causes of these shortcomings are analyzed, the corresponding scientific and methodological foundations of new methods for calculating the bearing capacity of pile foundations are developed.

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