Magazine of Civil Engineering

ISSN 2712-8172

journal homepage: http://engstroy.spbstu.ru/

Research article

UDC 624.1

DOI: 10.34910/MCE.125.2

Consolidation of water-saturated viscoelastic subgrade

***T.V. Maltseva*[1] ✉, *A.V. Nabokov*[1], *N. Vatin*[2]**

[1] *Industrial University of Tyumen, Tyumen, Russian Federation*
[2] *Peter the Great St. Petersburg Polytechnic University, St. Petersburg, Russian Federation*

✉ *maltsevatv@tyuiu.ru*

Keywords: weak soils, water-saturated foundation, stresses and deformations, soil viscoelasticity, consolidation

Abstract. The stress-strain state of the foundations of buildings and structures made of weak viscoelastic soils is considered. The mechanical characteristics of a viscoelastic water-saturated soil base were determined experimentally. A macro sample of soil in a pipe about 1 m high had a water lock on top to create excess pore pressure in the sample. Excessive pore pressure simulated the depth of the sample from the surface. From the experiment, the universal parameter of the kinematic model was determined, and the foundation was calculated. Theoretical data obtained within the framework of a kinematic model considering the viscoelastic properties of the soil are compared with the known Flamant solution and experimental data for a stabilized state of the soil. The deviation of vertical displacements from experimental data is no more than 4 % (one-dimensional case). The deviation of the theoretical solution of the flat Flamant-type problem (considering residual pore pressures) from the known solution of the Flamant problem is 16 %. The proposed calculation method makes it possible to predict the deformation of foundations made of water-saturated viscoelastic soils more accurately than the solution for elastic and elastoplastic soils without the influence of pore pressure. The technique is novel because it allows one to simultaneously consider the soil's residual pore pressures and the soil's viscoelasticity.

Funding: This research was supported by a grant from the Russian Science Foundation No. 23-29-00118, https://rscf.ru/project/23-29-00118/

Citation: Maltseva, T.V., Nabokov, A.V., Vatin, N. Consolidation of water-saturated viscoelastic subgrade. Magazine of Civil Engineering. 2024. 17(1). Article no. 12502. DOI: 10.34910/MCE.125.2

1. Introduction

To improve methods for calculating soil foundations under construction projects, it is necessary to experimentally and theoretically study the process of consolidating foundations from water-saturated soils. Many factors influence the consolidation process: the presence of pore water in the soil and the soil skeleton's creep [1] since the soil is a viscoelastic material. For weak water-saturated soils (water saturation coefficient of more than 80 %), the work [2] proposed nonlinear patterns (stress-strain relationship) for the deformability of an elastoplastic soil model. In soil mechanics, time-dependent soil reactions have long been considered the result of the compaction process of elastic and single-phase soil [3]. Later, models of two-phase soils (soil skeleton and pore fluid) appeared. It was assumed that soil compaction would cease when the pore fluid pressure had completely dissipated. However, this assumption contradicts the results of laboratory [4] and field [5] experiments. It is proposed in [6] to consider soft soils (for example, clay and loams) as a viscoelastic or viscoplastic material with rheological properties without considering the influence of pore fluid.

The manifestation of viscoelasticity properties in calculation models is considered through physical laws. The mechanical characteristics of the material included in the physical laws are determined from the experiment. Experimenting is another urgent task in studying the consolidation process [8].

In [10], the time-dependent behavior of soil is associated with primary consolidation (filtration consolidation) and with the viscoelastic properties of soils. On the one hand, the primary compaction of soils, caused by the dissipation of pore pressure, affects the stresses and deformations of bases and foundations. On the other hand, the soil is a viscoelastic medium; A. Mishra [11] used a three-parameter viscoelastic soil model. The viscoelasticity problem was converted into an elasticity problem using the Laplace transform. However, the soil is considered a solid body with viscoelastic properties, and the influence of residual pore pressure is not considered.

To predict changes in the stress-strain state of soil over time, the work [12] developed an elasto-visco-plastic soil model that considers the change in soil shape simultaneously with consolidation (without considering filtration consolidation). This model includes a linear relationship between the rate of development of shear strains and the effective stresses. The soil is single-phase, and the mechanics of a deformable solid body are applied to its modeling [13, 14].

Article [15] examines the influence of groundwater levels on the mechanical properties of soil. A two-layer soil model is proposed, which includes an unsaturated layer of water-saturated soil and a layer of non-water-saturated soil. The viscoelastic properties of the soil were not considered.

The creep of a weak two-phase soil considering excess pore water pressure was studied in [16] using the undrained triaxial unloading creep tests during the unloading of the sample. It was shown that soft soil's unloading creep behaviors are related to deviatoric stress and time. If the deviatoric stress is lower than the yield stress, then the deviatoric stress-strain curve exhibits linear viscoelastic properties. If the deviatoric stress is higher than the yield stress, then it exhibits strong nonlinear viscoplastic properties.

Previous studies have generally reduced soil consolidation to either compaction of saturated soil or compaction of unsaturated viscoelastic soil. The proposed soil models are not applicable for the analysis of real soil settlement since experiments show the presence of residual pore pressure in the soil after the completion of the filtration consolidation process. The soil is both water-saturated and viscoelastic. This article is devoted to partially eliminating these gaps. That is, it is proposed to model the process of soil consolidation considering water saturation and viscoelasticity. For this purpose, a kinematic soil model was chosen [7], into which physical laws for the soil skeleton and pore fluid were additionally introduced in the form of integral Boltzmann relations. Using the methods of operational calculus, the integral Boltzmann relations in the images coincided, up to notation, with the physical laws of the theory of elasticity. The viscoelastic problem solution is reduced to solving the problem in two stages. In the first stage, the elastic sample was calculated without considering time. In the second stage, a solution to the viscoelastic problem considering time was obtained. The authors have not found any other publications with this idea.

The kinematic model quantitatively describes the contribution of residual pore pressures to the stressed and deformed state of the two-phase foundation after the end of the filtration consolidation process. The soil is considered water-saturated. The water saturation is confirmed by numerous field experiments lasting up to 5 years, according to which the residual pore pressure at a depth of three to four meters can be up to 50 % of the total [5].

The kinematic model is based on two hypotheses [7]:

1. The difference in pore pressure does not cause the speed of water movement but causes small relative movements of particles of the soil skeleton and pore water.
2. The connection between solid and liquid soil particles is represented through relative movements (volume changes in the three-dimensional case).

It is necessary to conduct tests with water-saturated soil to consider the viscoelastic properties of the soil in the kinematic model (determining the model parameters). Creep tests mainly include macroscopic and microscopic aspects. Many researchers have proposed experiments at the macro level in an odometer [17] and at the micro level using a microscope to study the microstructure of the soil [18] or combine both aspects at once [8]. Testing samples of standard sizes (from 4 to 8 cm) cannot characterize the soil as two-phase. Firstly, all these experiments were carried out with samples of low height, up to 10 cm, and did not capture the influence of pore water in the soil sample. Secondly, the purpose of these experiments was to exclude pore water from consideration; as described above, there were no mathematical models that considered the influence of residual pore pressure after the end of the filtration consolidation process. There was no need to conduct experiments with a soil sample that would remain saturated at the end of the filtration consolidation process. When using a kinematic model, the available experimental data is insufficient, and a new experiment with a large-sized soil sample is proposed to determine the model parameters.

The purpose of the study was to study the process of consolidation of water-saturated soil foundations, considering the viscoelastic properties of the soil. To achieve the goal, experimental and analytical-numerical solution methods were used.

The object of study is a foundation made of weak soils such as loam and clay. The subject of the study is the prediction of the stress-strain state of the foundation. In this article, to determine the parameters of the kinematic model, it is enough to experiment with a large sample (1 m or more in height) because the model is based on the physical equations of the state of a continuous medium. The novelty of the experiment is described below in the Methods section.

2. Methods

The article uses two research methods: experimental and theoretical.

2.1 Experimental studies

A laboratory experiment was carried out to determine the mechanical characteristics (parameters of the kinematic model). The description of the experimental stand and experimental conditions is given in [19]. Fig. 1 shows the setup in which the experiment was carried out.

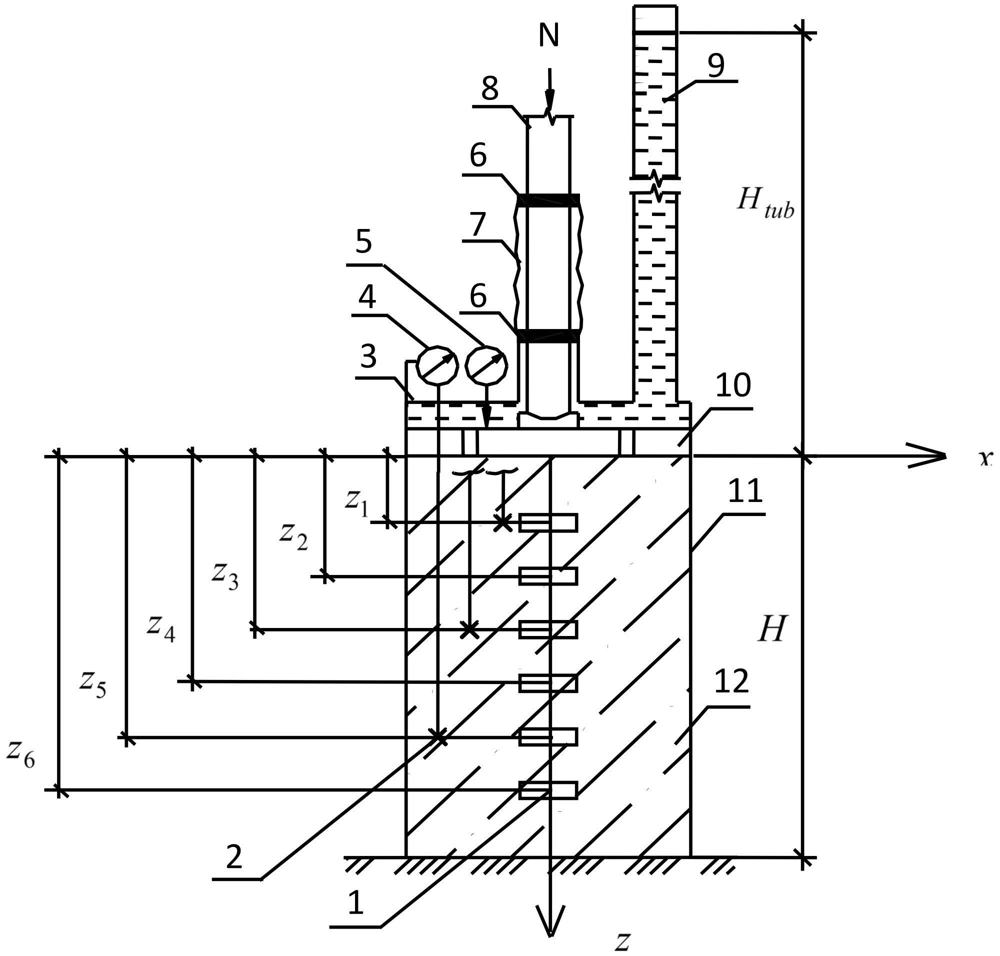

Figure 1 Test setup: 1 – sensor for determining total and pore pressure (mass dose); 2 – sensor for determining the movement of soil skeleton particles (brand); 3 – plug; 4 – deflection meter; 5 – dial indicator; 6 – clamp; 7 – rubber cuff; 8 – loading rod; 9 – water column tube; 10 – perforated stamp; 11 – pipe section; 12 – soil sample.

A two-phase soil sample had a diameter of d = 0.311 m and a height H = 0.8 m. The water column had a height of H_{tub} = 2.2 m above the sample. The sample was placed in a section of pipe with watertight walls and bottom. A polyethylene film was glued to the inner wall of the pipe using lithol, then a second layer of film was glued using technical petroleum jelly to eliminate friction of the sample against the pipe walls. The friction of one layer of film against another is so small that the tangential stresses that arise between the layers of the film can be neglected.

The water column simulated the distance from the daytime surface of the soil base to the soil sample, located at a depth of more than one meter. A working pressure of σ_0 = 0.1 MPa was applied to the perforated stamp with holes. Inside the sample, spaced apart in height, three membrane-type pressure sensors (BEC-A, manufactured by KYOWA, Japan) are installed to determine the total and pore pressure and sensors to determine the movements of soil skeleton particles (ICH-50, LLC NPP "Chelyabinsk Instrumental Plant", Chelyabinsk, Russia). The marks are installed at the same level as the messages. Fig. 1 shows the arrangement of messages and brands.

Dial indicators with a division value of 0.01 mm were installed on the stand to determine the movement of the stamp and stamps. The stamp was made of steel with dimensions: a height of 0.01 m and a diameter of 0.310 m. The stamp has 335 holes with a diameter of 0.004 m. Two layers of filter mesh and two layers of filter paper were laid at the stamp's base to prevent the stamp's holes

The experiment lasted 95 days. The maximum value of pore pressure, measured by the mass dose at the base of the sample, was 55 % of the applied pressure. Fig. 2. shows the experimental pore pressure curve in the horizontal section z = 0.69 m.

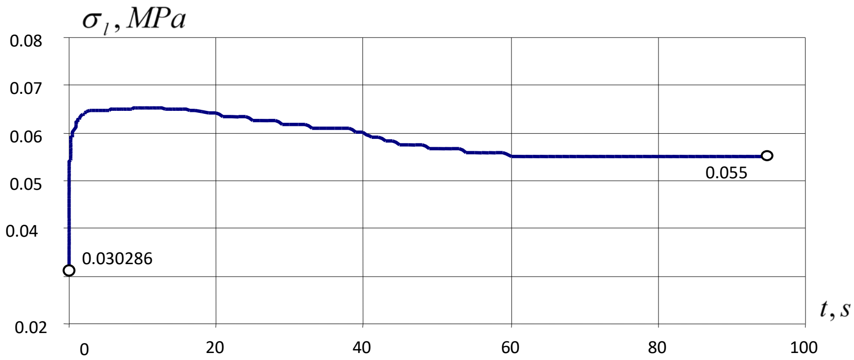

Figure 2. Pore pressure.

The novelty of the experiment was as follows: 1) the experiment was carried out with a large-sized sample; 2) water was additionally used over the sample, forming a water lock; 3) the waterlock increased the influence of pore water on the soil skeleton compared to the test without the waterlock.

2.2 Theoretical studies

Classical (viscoelastic) and generalized (structural viscoplastic) rheological models of soil are known. In [20], a generalized Voigt model was considered to simulate the viscoelastic properties of clay. The fractional calculus theory was first introduced in [21] with the dashpot element in a Kelvin-Voigt-type body to describe the compaction of viscoelastic saturated soils. In this theory, the soil's time-dependent stress-strain relationship is interconnected through the damage factor to a fractional degree. The model of viscoelastic plastic creep with a fractional derivative for frozen viscoelastic soils is considered in [22].

In contrast, in this article, the differential dependences of strains on stresses obtained in various viscoelastic models were not used. Still, the integral form of the dependence of strains on stresses was considered. The integral form is because the universal parameter of the kinematic model is determined from the experiment, so the structure of the material (soil) model is not interesting. The physical law of deformation from stress is specified using Boltzmann integrals based on experimentally found influence functions (creep). The experimental data were processed within the framework of the model of the linear hereditary theory of viscoelasticity proposed by A.A. Ilyushin [23]. The solution to the viscoelastic problem was obtained in two stages. In the first stage, the elastic sample was calculated without considering time. The resulting solution based on the Volterra principle (redesignation system) is written in Laplace-Carson images. In the second stage, for a fixed point in space, an approximate transition from the known image to the original was made using the method of broken lines by L.E. Maltsev [24]. The original is also represented by a special broken line (spline) recorded in the images. Several numerical values of the image parameter are assigned, that is, a set of points p_i, $i=1,...,n$, and the condition for the coincidence of the image of the broken line with the known image of the solution to the problem on the set of points is written. As a result, a system of linear algebraic equations is obtained, the order of which coincides with the number of links of the broken line.

Methodology of Ilyushin A.A. (splitting the problem's solution into two stages) for a Boussinesq-type problem about loading a viscoelastic water-saturated soil foundation with standard loads was applied earlier in [25, 26]. This article uses a two-stage technique to obtain a solution to the Flamant-type problem of loading a viscoelastic water-saturated soil foundation with a strip load.

3. Results and Discussion

3.1. Obtaining mechanical characteristics based on experimental data

The experimental graph $\sigma_l(t)$ (Fig. 2) was presented using polyline links and the Heaviside function:

$$\sigma_1(t)=\sigma(0)\cdot\left[1-\sum_{i=0}^{6}(b_i-b_{i+1})\cdot(t-t_i)\cdot h(t-t_i)\right],\ \ t=\frac{\bar{t}}{1s}. \tag{1}$$

The parameters of the function were as follows: $t_o=b_o=b_7=0,\ \ \sigma(0)=0.030286\ \text{MPa};$ $t_1=0.45;$ $t_2=3.95;$ $t_3=11.45;$ $t_4=38;$ $t_5=60;$ $t_6=95;$ $b_1=2.14973;$ $b_2=0.04857;$ $b_3=0.00279;$ $b_4=-0.00553;$ $b_5=-0.00889;$ $b_6=0.$

Here $\sigma(0)$ is the voltage value at the initial moment of dimensionless time.

Using the retardation theorem, the Laplace-Carson image of the stress function with known parameters σ_o, $b_1,...,b_5$ is determined:

$$\left[\sigma_1(\mathrm{t})\right]^*=\sigma_l^*(p)=\sigma(0)\cdot\left[1+\sum_{i=1}^{6}b_i\cdot\frac{1}{p}\cdot\left(e^{-p\cdot t_{i-1}}-e^{-p\cdot t_i}\right)\right]. \tag{2}$$

The formula for the kinematic model parameter in images (the square means that the parameter always has a positive value) was [19]:

$$\left[a^2(t)\right]^*=\left[a^2\right]^*(p)=-\frac{1}{z}\cdot\ln\left[1-\frac{\left[\sigma_l^{\exp}(t)\right]^*}{\sigma_o}\right], \tag{3}$$

where σ_o is the load on the stamp, $z=0.69$ m is the depth of the mass dose.

The broken line method [24] was used to find the original from a known image. The function $a^2(t)$ was represented by a polyline with five links:

$$\bar{a}^2(t)=a(0)\cdot\left[1-\sum_{i=0}^{5}(d_i-d_{i+1})\cdot(t-t_i)\cdot h(t-t_i)\right]. \tag{4}$$

The image of the polyline, according to Laplace-Carson, is as follows:

$$\left[\bar{a}^2(\mathrm{t})\right]^*=a(0)\cdot\left[1+\sum_{i=1}^{5}d_i\cdot\frac{1}{p}\cdot\left(e^{-p\cdot t_{i-1}}-e^{-p\cdot t_i}\right)\right]. \tag{5}$$

Here $a(0)$ and d_i are the required parameters, determined from the condition of coincidence of functions in the images $\left[\overline{a}^{-2}\right]^*(p_j)=\left[a^2\right]^*(p_j)$. The points of coincidence of functions were assigned according to the formulas [7] as:

$$p_j = \frac{\ln T_j - \ln T_{j-1}}{T_j - T_{j-1}}, \quad j=1,...5, \quad T_j = 0.1T_{j+1}.$$

The obtained system of linear algebraic equations for points p_j, $j=1,...5$ of the fifth order was:

$$\begin{cases} \sum\limits_{i=1}^{5} d_i\left(e^{-p_1 T_{i-1}} - e^{-p_1 T_i}\right) = \left[\dfrac{\left[a^2\right]^*(p_1)}{a(0)} - 1\right]\cdot p_1 \\ \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots \\ \sum\limits_{i=1}^{5} d_i\left(e^{-p_{n-1} T_{i-1}} - e^{-p_{n-1} T_i}\right) = \left[\dfrac{\left[a^2\right]^*(p_{n-1})}{a(0)} - 1\right]\cdot p_{n-1} \\ \sum\limits_{i=1}^{5} d_i\left(T_i - T_{i-1}\right) = \dfrac{\left[a^2\right]^*(p=0)}{a(0)} - 1 \end{cases}. \tag{6}$$

The last equation of the system of equations (6) was written for the point $p_5 = 0$:

$$\sum_{i=1}^{5} d_i\left(T_i - T_{i-1}\right) = \frac{\left[a^2\right]^*(p=0)}{a(0)} - 1.$$

The parameter $a(0) = 0.522854$ for the point $p_0 = \infty$ was determined from the last equation of the system of equations (6).

The coefficient matrix of the system of equations (6) is weakly defined. The accuracy of solving the system of equations depended on the choice of coincidence points (collocations) p_j. Next, the collocation points were refined from the solution of the transcendental equation [25]:

$$m\left(e^{-p_j t_{i-1}} - e^{-p_j t_i}\right) = \left(e^{-p_j t_i} - e^{-p_j t_{i+1}}\right), \quad m = 0.8.$$

The supradiagonal elements of the matrix are 0.8 of the diagonal elements. This method led to improved conditionality of the matrix. The method was justified in [25].

As a result of solving the system of linear algebraic equations (6), the spline (broken line) parameters of the mechanical characteristics of water-saturated viscoelastic soil $a^2(t)$ were obtained: $T_o = 0$; $T_1 = 0.1$; $T_2 = 1$; $T_3 = 5$; $T_4 = 25$; $T_5 = 60$; $p_o = \infty$; $p_1 = 1.618$; $p_2 = 0.2275$; $p_3 = 0.03876$; $p_4 = 0.01713$; $p_5 = 0$; $d_1 = 2.8277$; $d_2 = 0.14067$; $d_3 = 0.00074$; $d_4 = -0.02296$; $d_5 = -0.0034$.

The graph of function $a^2(t)$ is shown in Fig. 3.

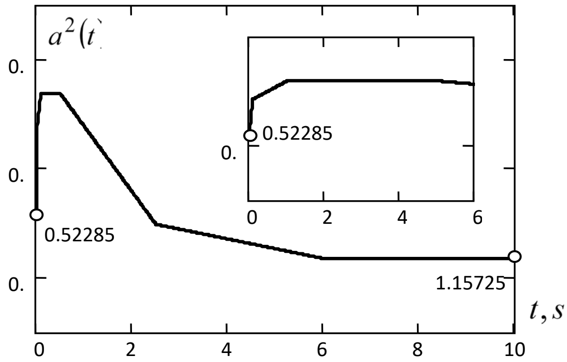

Figure 3. Model parameter $a^2(t)$.

3.2. Solution of the Flamant-type problem

A Flamant-type problem was solved within the framework of a kinematic model to describe the consolidation process of water-saturated viscoelastic soil under load.

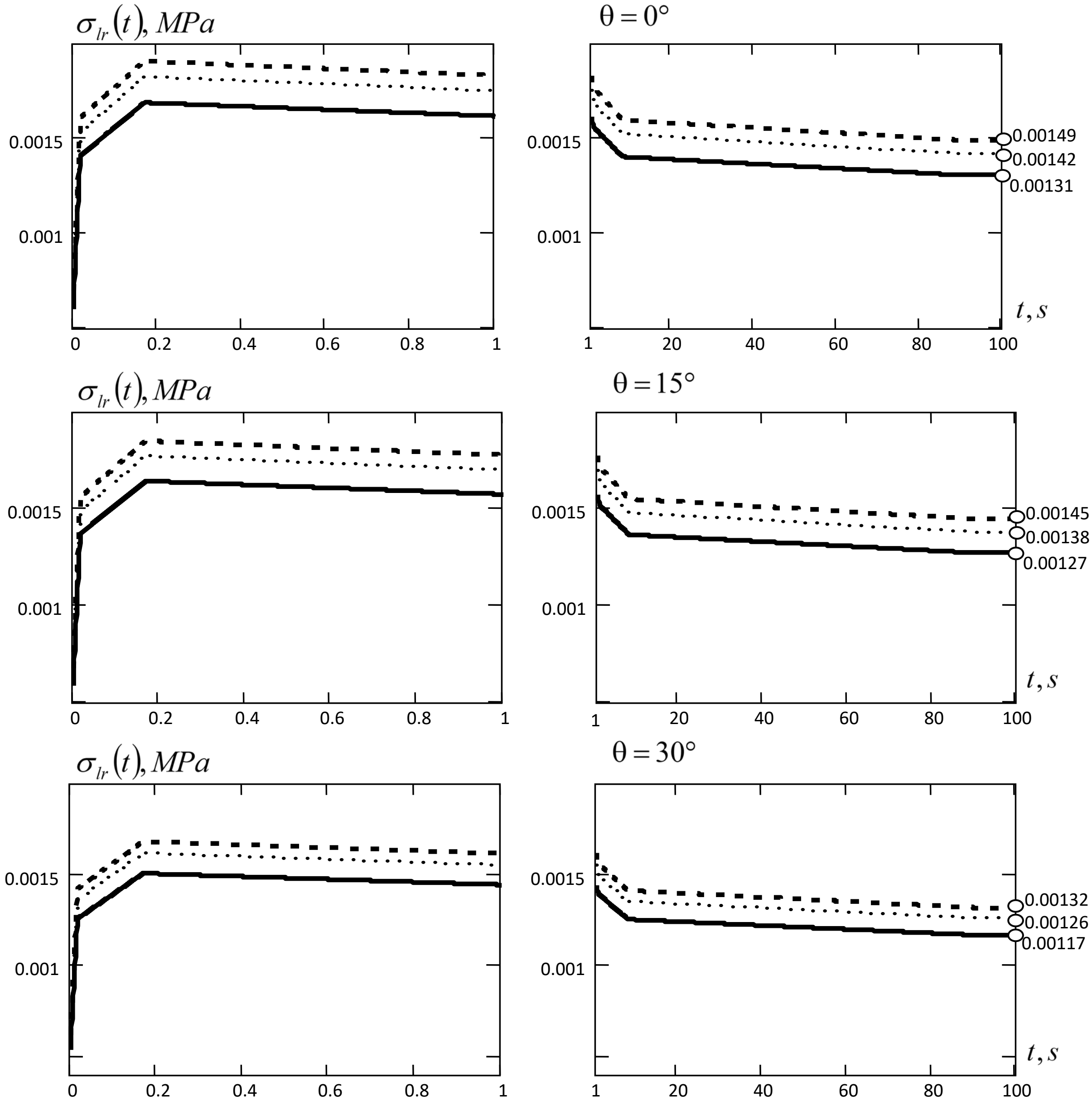

Figure 4. Pore pressure in different sections: *Θ = 0: (—) –* $r_1 = 0.5$ m, *(· · ·) –* $r_2 = 0.75$ m, *(- - -) –* $r_3 = 1$ m*; Θ =* 15^0*: (—) –* $r_1 = 0.52$ m, *(· · ·) –* $r_2 = 0.76$ m, *(- - -) –* $r_3 = 1.03$ m*; Θ =* 30^0*: (—) –* $r_1 = 0.58$ m, *(· · ·) –* $r_2 = 0.87$ m, *(- - -) –* $r_3 = 1.16$ m.

The resulting universal parameter $a^2(t)$ of the kinematic model was used to calculate stresses and displacements in the problem of the action of a strip load on a two-phase viscoelastic half-space based on the analytical solution from [7]. The solution in Laplace-Carson images to the Flamant-type problem for a two-phase elastic half-space in polar coordinates is presented as:

$$\left[\sigma_{lr}(t)\right]^* = \frac{2F\cdot\cos\theta}{\pi}\cdot\lambda\cdot\left[a^2(t)\right]^*\cdot e^{-\lambda\cdot\left[a^2(t)\right]^*\cdot r}\cdot\int\limits_{\rho}^{r}\frac{e^{\lambda\cdot\left[a^2(t)\right]^*\cdot r}}{r}dr, \tag{7}$$

where the strip load F, radius $\rho = 0.07$ m, and scale factor $\lambda = \left(H_{sam}/H_{tr}\right)$.

For a fixed point in space $\left(\theta_i, r_i\right)$, using the broken line method, we obtained the original pore pressure function $\sigma_{lr}(t)$ (Fig. 4), for which the initial time of nonmonotonic change was identified. The contribution of pore pressure increases with distance from the surface. Soil movements were determined using the polyline method.

The experiment conducted in a flume (Fig. 5, dimensions in mm) with the same soil as in a pipe (Fig. 1) was considered to compare the theoretical values obtained by solving a flat Flamant-type problem and the experimental values. The pore pressure curves and the function of the time of movement of the soil mass were obtained as a result of the experiment. Fig. 5 shows the sensor placement diagram. The values for a fixed time of the pore pressure function (stabilized state t = 95 days) are given in Table 1, and the displacement functions of soil skeleton particles are given in Table 2.

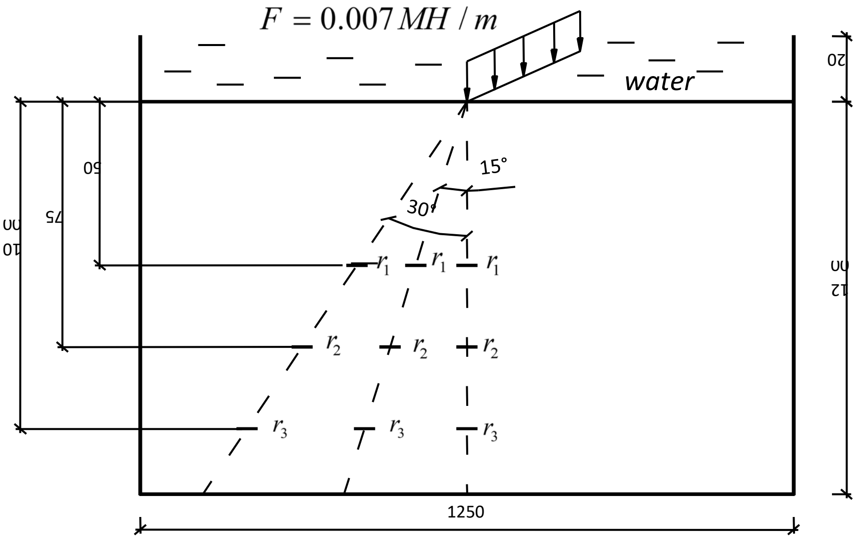

Figure 5. Tray with water-saturated clay and location of mass doses.

Table 1. Experimental and theoretical values of displacements.

θ, deg	r, m	$\sigma_{lr}^{exp}(t=95days), MPa$	$\sigma_{lr}^{th}(t=95days), MPa$	$\delta, \%$	σ, MPa, Flamant solution
0	0.5	0.0011	0.00131	16.0	0.008913
	0.75	0.0013	0.00142	8.5	0.00594
	1.0	0.0014	0.00149	6.0	0.004456
15	0.518	0.001	0.00127	21.3	0.00831
	0.776	0.00105	0.00138	20.3	0.005547
	1.035	0.0012	0.00145	17.2	0.00415
30	0.577	0.0009	0.00117	23.1	0.006686
	0.776	0.001	0.00126	20.6	0.004973
	1.155	0.0011	0.00132	16.7	0.003341

Table 2. Experimental and theoretical values of displacements for Θ = 0°.

r,m	$w_s(r),m$ experiment	$w_s(r),m$ Flamant solution	$\delta,\%$	$w_s(r),m$, theory	$\delta,\%$
0.25	0.00079	0.00086	8.2	0.00081	3.7
0.50	0.00048	0.00052	7.8	0.00048	2.1
0.75	0.00027	0.00031	16.1	0.00027	3.7
1.0	0.00011	0.00017	35.2	0.00014	21.4

Tables 1 and 2 contain theoretical calculation data based on formula (7) for a fixed time t = 95 days. A comparison of experimental and theoretical values of vertical displacements showed good convergence except for the edge point r = 1 m. Here, the discrepancy was 21.4 %.

4. Conclusion

Based on an experiment with a large water-saturated sample using the proposed method, a mechanical characteristic of the soil (model parameter) was obtained. The viscoelastic mechanical characteristic was used to calculate the stress-strain state of the soil foundation. As a result of the calculation of a viscoelastic water-saturated foundation loaded with an external load, broken time functions of pore pressure and vertical displacements of the soil skeleton were obtained using the approximate method. A comparison was made with the experiment and the known Flamant solution, because of which it was established that:

1. The pore pressure changes nonmonotonically under constant external load. These nonmonotonic changes are a feature of two-phase soil. Pore pressure increased with increasing depth to 50 % of the total stress due to the unloading effect of pore water.
2. The maximum discrepancy between the movements of particles of the soil skeleton and the experimental data occurred at a depth of 1 m. It amounted to 35 % according to the well-known Flamant solution, and 21 % according to the solution proposed in the article. Still, this discrepancy is not indicative since the effect of a solid bottom of the tray is manifested. The actual discrepancies between theoretical and experimental values are within 1–4 %. The tests in the flume showed that pore water was actively involved in perceiving external load, starting from a depth of one meter from the surface. The total stresses in the pore fluid and soil skeleton coincided with the stresses obtained by Flamant. The highest calculated pore pressure was 39.5 % of the total at the point $(\theta = 30°, r_3 = 1.155m)$.

Studying the influence of the viscoelastic properties of the skeleton of water-saturated soil will significantly expand the field of geotechnical design using more accurate and efficient methods for calculating soil foundations.

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Information about authors:

Tatyana Maltseva, *Doctor of Physics and Mathematics*
ORCID: https://orcid.org/0000-0002-0274-0673
E-mail: maltsevatv@tyuiu.ru

Alexander Nabokov, *PhD in Technical Sciences*
E-mail: nabokovav@tyuiu.ru

Nikolai Vatin, *Doctor of Technical Sciences*
ORCID: https://orcid.org/0000-0002-1196-8004
E-mail: vatin@mail.ru

Received 09.10.2023. Approved after reviewing 05.12.2023. Accepted 05.12.2023.