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SEALING OF A LAYER OF A SOUND WITH EFFECT OF ELASTIC PROPERTIES AND INHOMOGENEOUS BORDER CONDITIONS

Abstract. In engineering practice, the problems of compacting an earth medium of finite thickness possessing an elastic property are of great interest. In this connection, the process of one-dimensional compaction of a three-phase base with a water repulsion at a depth and under the influence of an external load, depending on time and coordinate, is considered. In this case, boundary value problems for the partial differential equation are solved. Moreover, the boundary conditions are not homogeneous. Calculation formulas for calculation of pore pressure values, stresses in the skeleton of the ground and the value of precipitation for any time interval from the start of loading t have been determined. Calculations from these formulas show that the precipitation curves for inhomogeneous boundary conditions are, in their value, less than for homogeneous boundary conditions. In this case, the stresses in the skeleton of the ground for inhomogeneous boundary conditions are smaller in magnitude than for homogeneous boundary conditions and they tend to a certain asymptote

Keywords: Process, compaction, soil, parallelepiped, pressure, foundation, foundation, boundary conditions, continuity of functions, differential equations, hypergeometric equations.

All existing works, except for a few, in the field of consolidation of elastic and elastically creeping soils were solved for homogeneous boundary conditions, although they must be non-homogeneous. In this connection, it is time to solve multidimensional problems of consolidation of soils with heterogeneous boundary conditions and on the basis of the solutions obtained, to establish design formulas that make it possible to ensure the strength of any building or structure.

Prof. N.M. Gersevanov at one time pointed out that static calculations of the bases do not give a real picture of the work of the bases. In this connection, it is necessary to proceed to the study of processes in soils that depend on time. In this case, as noted above, in soils of structures very often there are soils impregnated with water, and the soil itself consists of groundwater filling the void between the soil particles and the soil skeleton, consisting of a set of solid particles and bound water. They can be used for compaction of soil massifs having large dimensions in comparison with the thickness of the layer. If the surface of such a soil is taken as a conditional horizon, then before the load is applied, the head H in the whole ground mass will be zero. At the first moment after application of the load, the head H in this section of the ground surface will increase by an amount equal to, then, depending on the time, it will fall. At all other points on the surface of the soil, the head remains zero. At the same time, at the points of the ground not on the surface, a scalar field of heads is created, i.e. at each point the head rises to the value H, which is a function of the point, and, in the future, also from the time t. If the field of heads is known, it is possible in the future to determine a number of quantities depending on the pressure H. And an instant increase in pressure produces an instantaneous pressure p [1-6].

Thus, the instantaneous application of a load on the surface of the soil is the cause of the appearance of a field of heads accompanying the movement of water from points with a high head to a point with hydrodynamic pressure. It can be calculated at any point in the ground mass, if the head function is known. And with a large load, the hydrodynamic pressure, trying to move the soil from under the bases, can destroy it [7-9].

Below we shall consider the compaction of homogeneous soils having elastic properties with inhomogeneous boundary conditions.

To the base of the building are layers of soil lying lower and toward the basement bottom, affecting the stability of the vertical movement of the building. Depending on the task being investigated, they can be represented as a ground layer of the final compaction power, a ground rectangle and a ground parallelepiped.

In engineering practice, the problems of compacting an earth medium of finite thickness possessing an elastic property are of great interest. In this connection, let us consider the process of compaction of a three-phase base with a water repellent at a depth that is under the influence of an external load, depending on time and coordinate [8,10-14].

The compaction of the soil is mainly determined by its compressibility. The compressibility of the substrate depends on both the type of soil and the nature of the load. Dynamic loads cause considerable compaction in sandy soils and a weak one in clayey soils. Long-acting loads on the contrary strongly compact clay soils and weakly-sandy.

The phenomenon of compressibility of soils is very important in the design of engineering facilities on a consolidated basis. At the same time, the deformation of the soil compression mainly occurs due to the approach of solid particles to each other and is estimated by a change in the porosity coefficient with a change in the compressive stresses in the soil skeleton σ. Determination of the relationship between the coefficient of porosity and compressive stresses in the skeleton of the soil is usually made by laboratory means in compression devices [9,12,15-16].

The equation of the compression curve of K. Terzagi [17] is represented by the logarithmic dependence

$$\varepsilon = -A\ln(\sigma + \sigma_0) + C, \tag{1}$$

where A, σ_0 and C are the parameters of this dependence, determined from the experiments.

In the case of a relatively small change in the stress σ, the curvilinear outlines of the compression curve (1) can be replaced by a straight line, i.e.

$$\varepsilon = -\alpha_0 \sigma + A, \tag{2}$$

where a_0 and A are the parameters of the linear dependence.

Moreover, a_0 is called the coefficient of instantaneous compaction, and the larger the value of a_0, the more compacted the soil, therefore this parameter is called the coefficient of compaction. However, the coefficient of compaction or compactability of the soil as the stresses increase gradually decreases. Moreover, the compaction coefficients for weakly compacted clays reach values a_0 = 0.10-0.01 MPa, and for compacted clays decrease to values a_0 = 0.05-0.01 MPa [18-21].

Next, we will use the equation of mechanics of three-phase compacted soils, obtained in [16], which will be written as follows:

$$\frac{\partial \varepsilon}{\partial t} + \beta'(1+\varepsilon_{cp})\frac{\partial p}{\partial t} = \frac{k(1+\varepsilon_{cp})}{\gamma_в}\frac{\partial^2 p}{\partial z^2}, \tag{3}$$

where β' – the coefficient of volume compression; *k* – the filtration coefficient; *εcp* - average porosity coefficient; γ - the volumetric weight of water; *p* - the pressure in the pore fluid.

Then the basic equation of compaction of the ground medium (3), which has the elastic property (2), has the form

$$\frac{\partial p}{\partial t} = C_V^{(1)} \cdot \frac{\partial^2 p}{\partial z^2} + A\frac{\partial q}{\partial t}, \tag{4}$$

where

$$C_V^{(1)} = \frac{1+\varepsilon_{cp}}{\gamma \cdot [a+\beta'(1+\varepsilon_{cp})]}; \quad A = \frac{a}{a+\beta'(1+\varepsilon_{cp})} \tag{5}$$

Equation (4) for (5) is solved under the following initial and boundary conditions:

$$p(z,\tau_1) = q(z,\tau_1), \quad z \in [0,h], \quad \tau_1 \in [\tau, T]; \tag{6}$$

$$\left.\begin{aligned} & p(0,t) = q(0,\tau_1)\text{æ(t)}, \quad \text{t} \in [\tau, \text{T}], \\ & \left.\frac{\partial \text{p}}{\partial \text{z}}\right|_{z=h} = 0, \quad 0 \le \text{æ(t)} < 1. \end{aligned}\right\} \tag{7}$$

Conditions (7) are not homogeneous. In order to bring them to homogeneous conditions, we make a change of the form

$$\overline{p}(z,t) = p(z,t) - q(\tau_1)\text{æ(t)} \tag{8}$$

from whence

$$p(z,t) = q(t)\cdot\text{æ(t)} + \overline{p}(z,t) \tag{9}$$

Bearing in mind (8) and (9), conditions (6) and (7) are reduced to the form

$$\overline{p}(z,\tau_1) = q(\tau_1)\cdot[1-\text{æ}(\tau_1)], \tag{10}$$

$$\left.\begin{aligned} & \overline{p}(0,t) = 0, \\ & \left.\frac{\partial \overline{\text{p}}}{\partial \text{z}}\right|_{z=h} = 0. \end{aligned}\right\} \tag{11}$$

In this case, the basic equation of compaction (4) is reduced to the form:

$$\frac{\partial \overline{p}}{\partial t} = C_V^{(1)} \cdot \frac{\partial^2 \overline{p}}{\partial z^2} + (A+1)\frac{\partial q}{\partial t}, \tag{12}$$

where the constants, $C_V^{(1)}$, (A+1)are found from (5).

Below we define the solution of equation (12) for the initial (10) and boundary (11) conditions. This solution satisfying the boundary conditions (11) can be represented in the form

$$\overline{p} = \sum_{i=0}^{\infty} T_i(t)\cdot\sin\frac{(2i+1)\pi}{2h}z, \tag{13}$$

where $T_i(t)$ – is an unknown function depending only on t.

Expression (13) satisfies the boundary conditions (11). Indeed, from (13) for z=0 we have, $\overline{p}(0,t) = 0$ and for z=*h*

$$\frac{\partial \overline{p}}{\partial z} = \sum_{i=0}^{\infty} \frac{(2i+1)\pi}{2h} T_i(t)\cdot\sin\frac{(2i+1)\pi}{2}z = 0. \tag{14}$$

Consequently, expression (13) is a solution of equation (12), which satisfies boundary conditions (11). Next, we find an unknown function. To this end, we substitute expression (13) in (10), then multiply both sides of the resulting equation by $\sin\frac{(2i+1)\pi}{2}z$, We integrate with respect to the independent variable z from 0 to h. In this case we relatively obtain an ordinary differential equation of the form

$$\frac{\partial T_i(t)}{\partial t} + C_V^{(1)}\lambda_i^2 T_i(t) = Q_i(t), \tag{15}$$

Where

$$Q_i(t) = \frac{4(A+1)}{(2i+1)\pi}\cdot\frac{\partial q}{\partial t}. \tag{16}$$

Equation (15) for (16) is an inhomogeneous differential equation, the particular solution of which can be represented in the form

$$T_{1i}(t) = \int_{\tau_1}^{t} Q_i(\tau)\cdot e^{-C_V^{(1)}\lambda_i^2(t-\tau)}d\tau. \tag{17}$$

The general solution of equation (15) with $Q_i(t)=0$

$$T_{0i}(t) = C\cdot e^{-C_V^{(1)}\lambda_i^2 t}$$

Then expression (13) is reduced to the following form

$$\overline{p} = \sum_{i=0}^{\infty}\left[C_i\cdot e^{-C_V^{(1)}\lambda_i^2 t} + \int_{\tau_1}^{t} Q_i(\tau)\cdot e^{-C_V^{(1)}\lambda_i^2(t-\tau)}d\tau\right]\cdot\sin\frac{(2i+1)\pi}{2h}z, \tag{18}$$

where C_i - an arbitrary constant, which is found from the initial condition (10). And with $t=\tau_1$

$$C_i = \frac{4}{(2i+1)\pi}\cdot e^{C_V^{(1)}\lambda_i^2\tau_1} q(\tau_1)[1-æ].$$

The solution (18) of equation (12) with (10) and (11) has the form

$$\overline{p} = \frac{4}{\pi}\sum_{i=0}^{\infty}\frac{1}{2i+1}\left[q(\tau_1)[1-æ]\cdot e^{C_V^{(1)}\lambda_i^2(t-\tau_1)} + (A+1)\int_{\tau_1}^{t}\frac{\partial q}{\partial\tau}\cdot e^{C_V^{(1)}\lambda_i^2(t-\tau)}d\tau\right]\sin\frac{(2i+1)\pi}{2h}z \tag{19}$$

Expressing (19) in (9), we obtain the calculated formula for the pressure in the pore liquid, when the boundary conditions of the compacted ground mass are non-uniform, i.e.

$$p(z,t) = q(t)æ(\mathrm{t}) + \frac{4}{\pi}\sum_{i=0}^{\infty}\frac{1}{2i+1}\left[q(\tau_1)[1-æ]\cdot e^{C_V^{(1)}\lambda_i^2(t-\tau_1)} + (A+1)\int_{\tau_1}^{t}\frac{\partial q}{\partial\tau}\cdot e^{C_V^{(1)}\lambda_i^2(t-\tau)}d\tau\right]\times$$

$$\times\sin\frac{(2i+1)\pi}{2h}z. \tag{20}$$

Then the stress in the skeleton of the soil will be calculated by the formula

$$\sigma(x,t) = q(t)[1-æ(\mathrm{t})] + \frac{4}{\pi}\sum_{i=0}^{\infty}\frac{1}{2i+1}\left[q(\tau_1)[1-æ]\cdot e^{C_V^{(1)}\lambda_i^2(t-\tau_1)} + (A+1)\int_{\tau_1}^{t}\frac{\partial q}{\partial\tau}\cdot e^{C_V^{(1)}\lambda_i^2(t-\tau)}d\tau\right]\times$$

$$\times\sin\frac{(2i+1)\pi}{2h}z \tag{21}$$

It should be noted that from (20), (21) we have that $p\to 0$, $\sigma\to q$ for $t\to\infty$.At the same time, for $t\to\tau_1$ we find that $p(z,\tau_1)\to q(\tau_1)$, $\sigma(z,\tau_1)\to 0$.

If we denote the ratio of the draft S_t of a compacted soil mass that has elastic properties for any time t to its total stabilized sediment S_∞, through

$$U=\frac{S_t}{S_\infty}, \tag{22}$$

Then

$$S_t=U\cdot S_\infty, \tag{23}$$

where the degree of consolidation U is from

$$U=1-\frac{8}{\pi^2}\sum_{i=0}^{\infty}\frac{1}{(2i+1)S_\infty}\left[q(\tau_1)[1-æ]\cdot e^{C_V^{(1)}\lambda_i^2(t-\tau_1)}+(A+1)\int_{\tau_1}^{t}\frac{\partial q}{\partial\tau}\cdot e^{C_V^{(1)}\lambda_i^2(t-\tau)}d\tau\right] \tag{24}$$

S_∞ has the meaning

$$S_\infty=\frac{aq(1-æ)h}{1+\varepsilon_{cp}}. \tag{25}$$

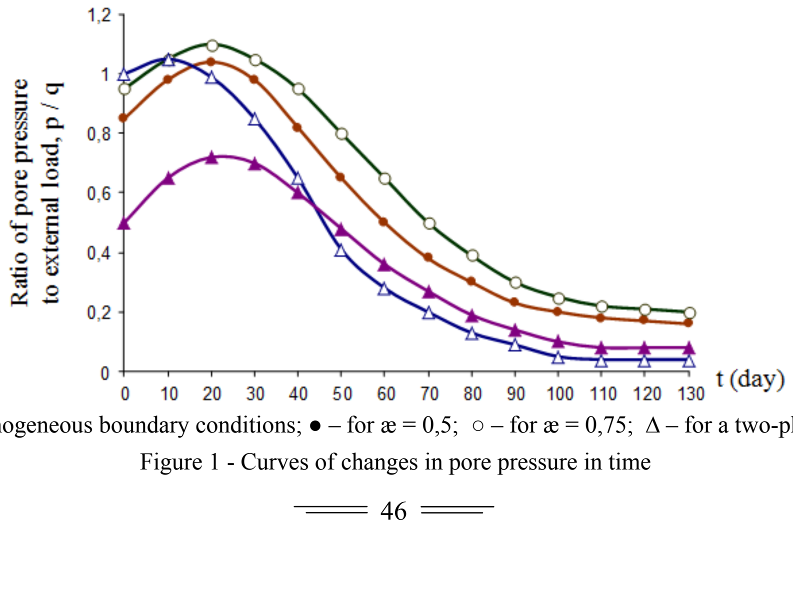
Using formulas (20) - (25) and values for e^{-x}, calculate the pore pressure $p(x,t)$, stresses in the skeleton of the ground $\sigma(x,t)$ and the value S_t of the precipitation for any time interval from the start of loading t, respectively.

Let it be required to determine the draft of a soil base with a size of *h*=5, if the medium to be condensed is characterized by the above parameters, and the external load varies according to law $q=2e^{-0{,}14t}$. In this case, the integral in (20) is equal to the following quantity;

$$\int_{\tau_1}^{t}\frac{\partial q}{\partial\tau}\cdot e^{C_V^{(1)}\lambda_i^2(t-\tau)}d\tau=\frac{2e^{C_V^{(1)}\lambda_i^2 t}}{C_V^{(1)}\lambda_i^2-0{,}14}\cdot\left\{e^{-[0{,}14-C_V^{(1)}\lambda_i^2]\cdot t}-e^{-[0{,}14-C_V^{(1)}\lambda_i^2]\cdot\tau_1}\right\}, \tag{26}$$

where k = $36\cdot 10^{-8}$ sm/sec; β′ = 0,35 MPa; ε_{cp} = 1,18; γ = 0,01 n/sm^3; a_0 = 0,022 MPa we have that $k(1+\varepsilon_{cp}) = 78{,}48\cdot 10^{-8}$ sm/sec; $\beta'(1+\varepsilon_{cp}) = 5{,}4936\cdot 10^{-7}$ MPa; $C_V^{(1)}$ = 0,0003923; A = 0,048.

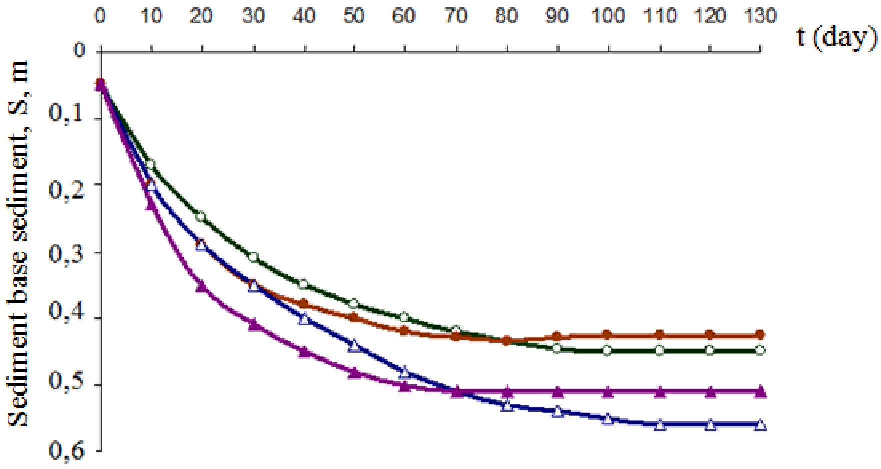
$$\lambda_i^2=\left[\frac{(2i+1)\pi}{2h}\right]^2;\quad æ=0{,}75;\quad æ=0{,}5;\quad æ=0{,}25;\quad æ=0.$$



▲ – for homogeneous boundary conditions; ● – for æ = 0,5; ○ – for æ = 0,75; Δ – for a two-phase medium;

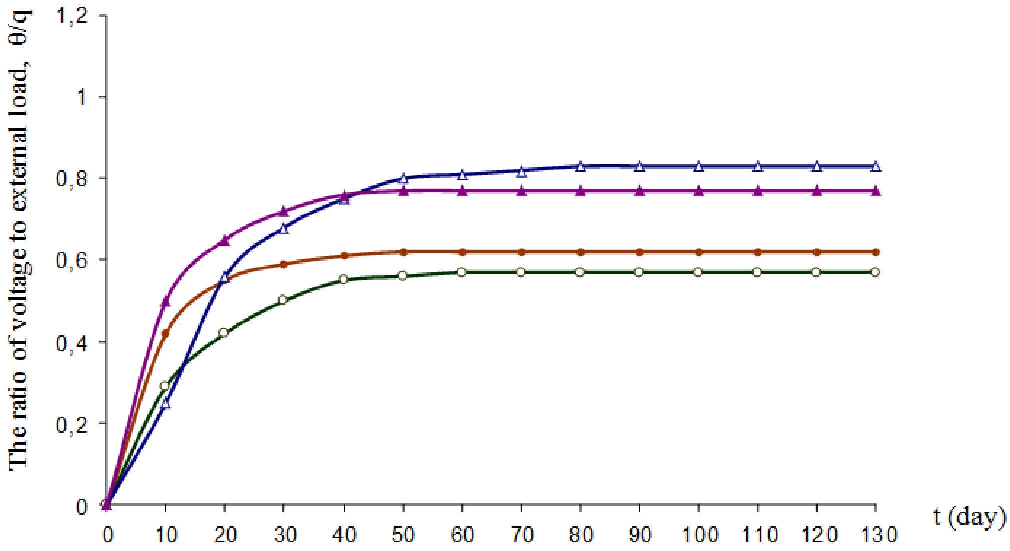
Figure 1 - Curves of changes in pore pressure in time

Using the relation (26) and the values $C_V^{(1)}, A, \lambda_i^2$, æ,of the quantities, æ, we obtain the numerical values of the pressure in the pore fluid, the stresses in the skeleton of the soil and the sediment of the compacted soil mass, the diagrams of which are given in Figures 1, 2, and 3.



▲ – for homogeneous boundary conditions;
● – for æ = 0,5; ○ – for æ = 0,75; Δ – for a two-phase medium;

Figure 2 - Curves of the changes in the sediment of the condensed massif in time



▲ – for homogeneous boundary conditions;
● – for æ = 0,5; ○ – for æ = 0,75; Δ – for a two-phase medium;

Figure 3 - Curves of stress changes in the skeleton of the soil

The precipitation curves for inhomogeneous boundary conditions show that for this case the vertical displacements of any points of the condensed array are smaller in value than in the case of homogeneous boundary conditions. In this case, the stresses in the skeleton of the ground for inhomogeneous boundary conditions are smaller in magnitude than for homogeneous boundary conditions and they tend to a certain asymptote.

It should be noted that recent work has been devoted to these questions [2-6,9].

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СЕРПІМДІЛІК ҚАСИЕТТЕРІ БАР ЖӘНЕ ШЕКТЕСТІК ЖАҒДАЙЛАРЫ ӘРТЕКТІ ТОПРАҚ ҚАБАТЫҢ ТЫҒЫЗДАУ

Аннотация. Серпімділік қасиеттері баржер қабатының соңғы қалыңдығын тығыздау есептері инженерлік тәжірибе саласында көп қызығушылық туғызады. Осыған орай, суға төзімді тереңдікте жәнеуақыт пен кооржинаттарға тәуелді сыртқы салмақ болған кезде үш фазалы негіздеменің біртекті тығыздалу үрдісі қарастырылды. Бұл жағдайда, дербес туындылар үшін шеттік есептер шығарылады. *t енгізілген кезден бастап кез-келген уақыт аралығы үшін* межелік қысым $p(x,t)$, топырақ қаңқасындағы (құрылымы ?) кернеу көрсеткіштері *о (x,t)* және шөгінді мәнін S_t есептеп шығару үшін есептеу формулалары анықталды. Осы формулалар негізінде табылған мәндер бойынша әртекті шектес жағдайлар үшін қисық шөгінділердің өзінтік көрсеткіштері біртекті шектес жағдайлар үшін көрсеткіштеріне қарағанда аз. Сонымен бірге, әртекті шектес жағдайлар үшін топырақ қаңқасындағы кернеу көрсеткіштері үлкендігі бойынша біртекті шектес жағдайла үшін кернеу көрсеткіштеріне қарағанда төмен және олар белігілі бір асимптотқа талпынады.

Түйін сөздер: үдеріс, топырақ, параллелепипед, қысым, негіздер, негіздеме, шектес жағдайлар, функциялардың үздіксіздігі, дифференциалды теңдеу, гипергеометриалы теңдеу

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УПЛОТНЕНИЕ СЛОЯ ГРУНТА, ОБЛАДАЮЩЕГО УПРУГИМ СВОЙСТВОМ И НЕОДНОРОДНОСТЬЮ ГРАНИЧНЫХ УСЛОВИЙ

Аннотация. В инженерной практике большой интерес представляют задачи уплотнения земляной среды конечной толщины, обладающей упругим свойством. В связи с этим рассмотрен процесс одномерного уплотнения трехфазного основания с водоупором на глубине и находящегося под действием внешней нагрузки, зависящей от времени и координаты. При этом решается краевые задачи для уравнении в частных производных. Причем граничные условия неоднородны.

Определены расчетные формулы для вычисления значений порового давления $p(x,t)$, напряжения в скелете грунта $\sigma(x,t)$ и значения осадки S_t для любого промежутка времени от начала загружения t.

Вычисления по этим формулам показывают, что кривые осадки для неоднородных граничных условий по своему значению меньше, чем для однородных граничных условий. При этом значения напряжений в скелете грунта для неоднородных граничных условий по величине меньше, чем для однородных граничных условий и они стремятся к определенной асимптоте

Ключевые слов: Процесс, уплотнения, грунт, параллелипед, давления, основания, фундамент, граничные условия, непрерывность функций, дифференциалные уравнения, гипергеометрические уравнения.

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