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Reinforcement of weak seasonally freezing soils at road consructing by double-cone hollow pile method

B S Yushkov[1], A A Degtyar[1], A B Shakhova[2]

[1]Department of Highways roads and bridges, Perm National Research Polytechnic University, 29, Komsomolsky Avenue, Perm 614990, Russia
[2]Department of Design, Graphics and Descriptive Geometry, Perm National Research Polytechnic University, 29, Komsomolsky Avenue, Perm 614990, Russia

E-mail: 1439sanek@mail.ru

Abstract. The article deals with the reinforcement method of weak seasonally freezing soils in subgrade base by using double-cone hollow piles, tridimensional geogrid, geotextile for roads in the northern regions of the Russian Federation. The method of calculating the road base in the form of pile strip foundation of double-cone hollow piles reinforced by tridimensional geogrid and geotextile on weak heaving soils taking into account traffic loads and weight of subgrade is considered. The developed method of calculation is based on the formation of soil compaction zones in the near-pile space as a result of pile driving into the ground, which leads to an increase in the structural strength of the weak soil, and also takes the arch effect that occurs in the soil between adjacent pile heads.

1. Introduction

Significant territories of the Russian Federation are characterized by a long winter period and a significant depth of soil freezing. In the northern regions of the country, soils are weak and loamy (water-saturated, clay, seasonally frozen). Presence of this kind of soils assumes their use as a base of automobile roads. High humidity of the soils makes them weak. Their use leads to the deformations which are unacceptable for roads such as shifts, sagging, soil creep. At negative temperatures frost heaving occurs as a result of the soil volume increase on freezing. The conditions for road construction on weak heaving soils are extremely unfavorable and require solving a number of problems to ensure strength, stability and durability of roads, without developing deformations.

When building roads in a ravine, in a forest zone with low evaporation capacity, in places of long standing surface water or difficult drainage, in areas with a high level of groundwater or in vadose zones, loamy soil at the road base will be in a water-logged state, which will lead to development of unacceptable structure deformations, and at negative temperatures – to heaving. Under such conditions the erection of a subgrade forces the builders to replace the entire depth of a weak ground base by sandy soil, which has a high filtration coefficient and a low capillary rise height [1]. This way of erecting a subgrade significantly increases the time and cost of building roads.

2. Innovations in road construction

To reduce the cost and time of the road construction on weak heaving soils, a new design solution was developed. Its feature is construction of a subgrade not replacing soft ground, but transferring the load from the subgrade weight and traffic to strong ground rocks through double-cone hollow piles (Figure 1) developed at the Department of Automobile Roads and Bridges in Perm National Research Polytechnic University (PNRPU) [2-7].

A double-cone hollow pile is a hollow structure having a taper toward the point and the head of the pile, made by centrifugation.

A distinctive feature of these piles is their resistance to frost, due to the unique shape, i.e. they do not change their design position, in contrast to prismatic piles, whose upthrust is 6-10 cm per year, which leads to impassability of the road. Therefore, prismatic piles are not used in the road construction on weak water-logged loamy seasonally freezing soils [8].

Design of the subgrade on the strip foundation of double-cone hollow piles is shown in Figure 2.

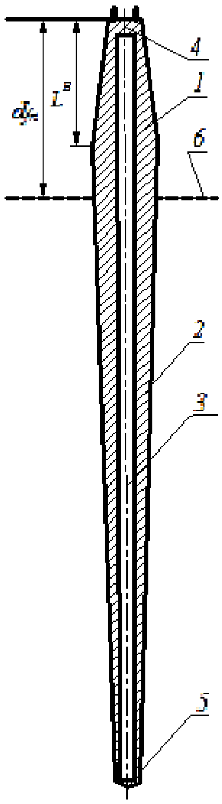

Figure 1. Pile structure:
1 - upper conical part;
2 - lower conical part;
3 - internal cavity;
4 - upper pile end;
5 - bottom pile end;
6 - boundary of the seasonally-freezing soil;
L^B - length of the upper part; d_{fn} - normative freezing depth.

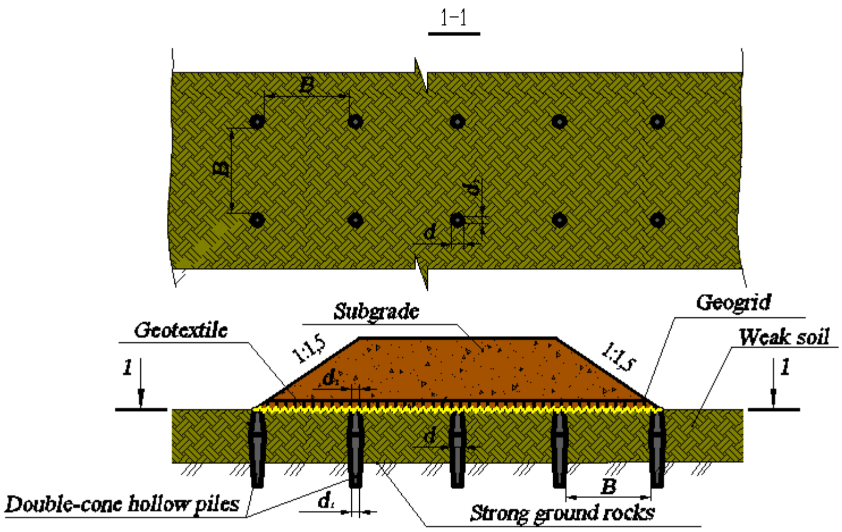

Figure 2. Subgrade on a strip foundation of double-cone hollow piles: d_1 *and* d are pile geometric characteristics, respectively diameters of the ends and the conjugation diameter of the planes of the upper and lower pile cones; B - distance between piles in clearance by conjugation diameters of planes of upper and lower pile cones d .

The basis for determining the distance between piles in a strip pile foundation is an analytical method for calculating the soil compaction zone around pile groups [9]. One of the main factors affecting change in the bearing capacity of pile foundations in time is formation of soil compaction zones caused by pile driving. The compaction zones of water-logged loamy soil depend on the natural

IOP Conf. Series: Materials Science and Engineering **451** (2018) 012044 doi:10.1088/1757-899X/451/1/012044

soil density, the method of pile driving, the number of piles in a group, the distance between piles, the cross-section of piles and the natural porosity coefficient.

3. Determine the compaction zone size of the pile foundation

To determine the compaction zone size of the pile foundation, we select a site in the form of a group of 9 piles. Each row of the group is a pile strip foundation, the piles in which are located at the same distance, both in the longitudinal and transverse direction. By virtue of symmetry, it is sufficient to consider ¼ of the distinguished pile foundation. As a result of pile driving in the horizontal layer Δh, displacement of the ground particles occurs. The soil excavated by driven piles compacts the adjacent soil, as a result of which the porosity coefficient in the space between piles decreases from ε to ε_{min}. Part of the ground is squeezed out of the pile foundation into a zone of width L. This zone consists of three sections - I, II, III (Figure 3).

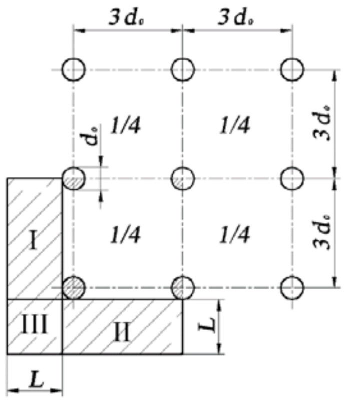

Figure 3. Calculation scheme for determining compaction zones: d_0 – pile diameter; $3d_0$ – distance between pile axes.

The rest of the soil is squeezed out into the compaction zone beyond the foundation.

The condition for the balance of particles displaced in the horizontal layer makes it possible to obtain an expression for the compaction zone width L in terms of ε and ε_{min}:

$$L = \frac{(\varepsilon - \varepsilon_{min})\left(1{,}5\sqrt{\dfrac{1+\varepsilon}{\varepsilon - \varepsilon_{min}}} - 3{,}5\right)}{(1+\varepsilon)(1+\varepsilon_{min})\mathcal{I}} d_0, \tag{1}$$

where $\mathcal{I}$ – a value characterizing the quadratic law of decrease of the soil porosity coefficient from ε to ε_{min}:

$$\mathcal{I} = \frac{1}{2\sqrt{1+\varepsilon}\sqrt{(\varepsilon - \varepsilon_{\min})}} ln \frac{\sqrt{(\varepsilon - \varepsilon_{\min})} + \sqrt{1+\varepsilon}}{\sqrt{1+\varepsilon} - \sqrt{(\varepsilon - \varepsilon_{\min})}} - \frac{1}{1+\varepsilon}, \tag{2}$$

Porosity in zone L decreases from ε at the boundary of the compacted zone to ε_{min} at the foundation boundary.

Having obtained the width of the compaction zone L, it is possible to calculate the distance between the piles in the pile strip foundation by the formula:

$$B = 2L, \tag{3}$$

where B – distance between piles in clearance.

Using the analytical method for calculating the soil compaction zone around the pile groups, we define the soil compaction zone around the pile foundation of 9 double-cone hollow piles with the following dimensions:

IOP Conf. Series: Materials Science and Engineering **451** (2018) 012044 doi:10.1088/1757-899X/451/1/012044

- diameter at the top and bottom ends $d_{01} = 30\,\text{cm}$;
- diameter along the conjugation line of the planes of the upper and lower cones $d_{02} = 50\,\text{cm}$;
- arbitrary total length, taken up equal to 3 m (upper cone part - 1 m, lower cone part - 2 m);
- distance between pile axes $-3d_0$,

(where d_0 – pile diameter $d_{01} = 30\,\text{cm}$ or $d_{02} = 50\,\text{cm}$).

At the same time, the porosity coefficient of a weak soil in the natural state is $\varepsilon = 0,80$, and the soil porosity coefficient at the maximum density obtained by pile driving $\varepsilon_{\min} = 0,65$.
According to formula (2), we find the value characterizing the quadratic law of decrease in the soil porosity coefficient from ε to ε_{min}:

$$\mathcal{I} = \frac{1}{2\sqrt{(1+\varepsilon)}\sqrt{(\varepsilon-\varepsilon_{\min)}}}\ln\frac{\sqrt{(\varepsilon-\varepsilon_{\min1)}}+\sqrt{(1+\varepsilon)}}{\sqrt{(1+\varepsilon)}-\sqrt{(\varepsilon-\varepsilon_{\min)}}}-\frac{1}{1+\varepsilon}=$$

$$=\frac{1}{2\sqrt{(1+0,8)}\sqrt{(0,8-0,65)}}\ln\frac{\sqrt{(0,8-0,65)}+\sqrt{(1+0,8)}}{\sqrt{(1+0,8)}-\sqrt{(0,8-0,65)}}-\frac{1}{1+0,8}=0,0152.$$

The width of the soil compaction zone in the near-pile space at $d_{01} = 30\,\text{cm}$ is:

$$L_1 = \frac{(\varepsilon-\varepsilon_{\min1})\left(1,5\sqrt{\dfrac{1+\varepsilon}{\varepsilon-\varepsilon_{\min}}}-3,5\right)}{(1+\varepsilon)(1+\varepsilon_{\min1})\mathcal{I}}d_{01} = \frac{(0,8-0,65)\left(1,5\sqrt{\dfrac{1+0,8}{0,8-0,65}}-3,5\right)}{(1+0,8)(1+0,65)0,0152}30 = 173\,cm.$$

The width of the soil compaction zone in the near-pile space at $d_{02} = 50\,\text{cm}$ is determined as follows:

$$L_2 = \frac{(\varepsilon-\varepsilon_{\min})\left(1,5\sqrt{\dfrac{1+\varepsilon}{\varepsilon-\varepsilon_{\min}}}-3,5\right)}{(1+\varepsilon)(1+\varepsilon_{\min})\mathcal{I}}d_{02} = \frac{(0,8-0,65)\left(1,5\sqrt{\dfrac{1+0,8}{0,8-0,65}}-3,5\right)}{(1+0,8)(1+0,65)0,0152}50 = 282\,cm.$$

Based on the results obtained after determining the values L_1 and L_2, we construct the diagram of the soil compacting zone width in the near-pile space as a result of pile driving (Figure 4).

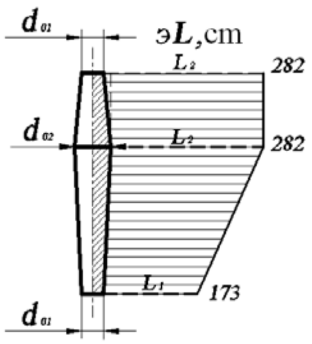

Figure 4. The diagram of the soil compaction zone width as a result of pile driving: d_{01} – diameter at the top and bottom end of the pile; d_{02} – diameter along the conjugation line of the planes of the upper and lower pile cones; L_1 – soil compaction zone width along the diameter of the pile bottom end d_{01}; L_2 – soil compaction zone width along the diameter d_{02} and on the ground surface.

The soil compaction zone width as a result of pile driving in the plane of the pile upper end (on the surface of the weak soil) will have the value $L_2 = 282\,\text{cm}$. This is due to the passage of a larger pile diameter $d_{02} = 50\,\text{cm}$ through the weak ground surface of at its driving.

With a decrease in the porosity coefficient of the soil from ε to $\varepsilon_{\min}$ in the near-pile space as a result of pile driving, the structural strength of the soil increases:

$$\text{from } P_{str1} \text{ to } P_{str2} \ (P_{str1} < P_{str2})$$

where $P_{str1} = 1,0 \ \ \text{kg/cm}^2$ – natural structural strength of soft soil at maximum porosity coefficient ε; P_{str2} – the greatest structural strength of soft soil at minimum porosity coefficient ε_{min} obtained as a result of pile driving in the near-pile space.

Structural strength of soil is inversely proportional to the change in its porosity coefficient [9]:

$$\frac{\varepsilon}{\varepsilon_{min}} = \frac{P_{str2}}{P_{str1}}, \tag{4}$$

$$P_{str2} = \frac{\varepsilon \cdot P_{str1}}{\varepsilon_{min}}.$$

When the soil porosity coefficient varies from $\varepsilon = 0,8$ to $\varepsilon_{\min} = 0,65$ the structural strength $P_{str1} = 1,0 \ \text{kg/cm}^2$ changes to P_{str2} according to formula (4):

$$P_{str2} = \frac{\varepsilon \cdot P_{str1}}{\varepsilon_{min}} = \frac{0,8 \cdot 1,0}{0,65} = 1,23 \text{kg/cm}^2.$$

After obtaining the width values of the compaction zones L_1 и L_2 determine the distance between piles in the pile strip foundation by formula (3):

$$B_1 = 2L_1 = 2 \cdot 173 = 346\,cm;$$

$$B_2 = 2L_2 = 2 \cdot 282 = 564\,cm.$$

From the two values obtained B_1 и B_2 we take $B_2 = 564\,\text{cm}$ as the distance between the piles. Within the space B_2 the soil has structural strength $P_{str2} = 1,23 \ \text{kg/cm}^2$ (figure 6).

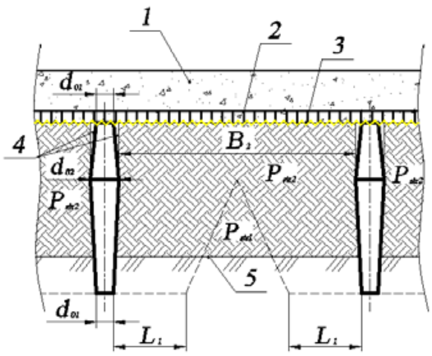

Figure 5. Vertical plan of the subgrade along the strip pile foundation axis: 1 – earth road bed; 2 - geocell material; 3 - geotextile material; 4- slit between the upper cone and natural soil, filled with sand or gravel; 5 - strong ground rocks, d_{01} - diameter at the top and bottom pile ends; d_{02} - diameter along the conjugation line of the planes of the upper and lower pile cones; L_1 - width of the soil compaction zone along the diameter of the lower end of the pile d_{01}; B_2 - distance between adjacent piles.

Installation of a strip pile foundation of double-cone hollow piles at the base of the subgrade improves the soft ground base bearing capacity by increasing its structural strength from $P_{str1} = 1,0\,\text{kg/cm}^2$ to $P_{str2} = 1,23\,\text{kg/cm}^2$ as a result of forming soil compaction zones in the near-pile space during pile driving. For the normal operation of roads without unacceptable deformations, it is necessary to fulfill the condition:

$$\delta_{zh} \le [\delta_{zh}] \tag{5}$$

$$\left[\delta_{zh}\right]=\frac{P_{str2}}{\left[n\right]}, \tag{6}$$

where $\delta_{zh}-$ the total stress from the traffic load and the subgrade weight, $\mathrm{kg/cm^2}$; $\left[\delta_{zh}\right]-$ the maximum allowable stress on the soft ground surface, $\mathrm{kg/cm^2}$, which is determined by formula (11):

$$\left[\delta_{zh}\right]=\frac{P_{str2}}{\left[n\right]}=\frac{1,23kg/cm^2}{1}=1,23kg/cm^2;$$

$\left[n\right]=1,0-$ safety factor; $P_{str2}-$ the greatest structural strength of soft ground in the near-pile space with a minimum porosity coefficient ε_{min}, obtained as a result of pile driving.

The total stress from the traffic load and the weight of the subgrade is determined by the formula:

$$\delta_{zh}=\delta_z+\delta_{zg}\ , \tag{7}$$

where δ_z- load stress, $\mathrm{kg/cm^2}$; $\delta_{zg}-$ stress from the ground's own weight, $\mathrm{kg/cm^2}$.

4. Determine the stresses

To determine the stresses arising on the soft ground surface from the traffic load, we use the formula of soil mechanics (Boussinesq formula [10]). This formula makes it possible to determine the stresses in the soil massif from the action of the vertical concentrated force (the stresses on the soft ground surface from the traffic load in the form of a concentrated force will be greater than from the uniformly distributed load over the area of the track of the wheel, i.e. we take the most unfavorable combination of loads) applied to the surface of a linearly deformed half-space according to the scheme given below (Figure 6):

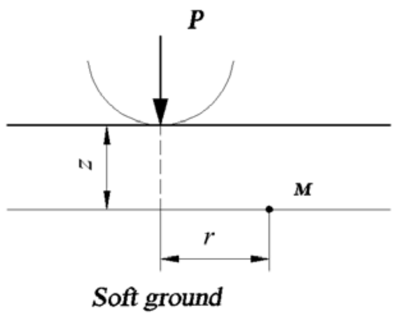

Figure 6. Scheme for calculating stresses on the soft ground surface from the traffic load: z – vertical distance to point M; r - horizontal distance to the point M; M - a point on the soft ground surface in which stress δ_z is determined.

$$\delta_z=\frac{K\cdot P}{z^2}\ , \tag{8}$$

where $K-$ coefficient depending on the ratio r/z; $K=f\left(r/z\right)$, from table 6.1. [10]; $P=10$ т $-$ axle load; $z=0,5\,м-$ vertical distance for points on the surface in which the stress δ_z is determined (at the subgrade height of 0.5 m).

The results of determining the stress value δ_z at various points on the soft ground surface are given in Table. 1.

From the obtained values δ_z (table 1) choose the maximum $\delta_z=1,9\,\mathrm{kg/cm^2}$, which directly corresponds to the place under the load application point (point №1) and is determined by formula (8):

$$\delta_z=\frac{K\cdot P}{z^2}=\frac{0,4775\cdot 10}{0,5^2}=19t/m^2=1,9kg/cm^2.$$

IOP Conf. Series: Materials Science and Engineering **451** (2018) 012044 doi:10.1088/1757-899X/451/1/012044

Table 1. Determination of stress values δ_z.

Point number	***r***, ***m***	***z***, m	r/z	***K***	$\boldsymbol{\delta_z}$, t/m^2
1	0	0,5	0	0,4775	19,1
2	0,5	0,5	1	0,0844	3,376
3	1,0	0,5	2	0,0085	0,34
4	1,5	0,5	3	0,0015	0,06
5	2,0	0,5	4	0,0004	0,016
6	2,5	0,5	5	0,0001	0,004

Note: $r-$ horizontal distance for the point at which stress is determined; $z-$ vertical distance for the point at which stress is determined; $K-$ coefficient dependent on the ratio r/z; $\delta_z -$ values of stresses arising on the soft ground surface.

The stress from the subgrade weight on the soft ground surface is determined by the formula:

$$\delta_{zg} = \gamma \cdot h \, , \tag{9}$$

where $\gamma = 1,3\,\mathrm{t/m^2} -$ average specific weight of the subgrade; $h = 0,5\,\mathrm{m} -$ height of the subgrade;

$$\delta_{zg} = \gamma \cdot h = 1,3 \cdot 0,5 = 0,65 t / m^2 = 0,065 \ \ kg / cm^2 .$$

Determine the value of the total stress from the traffic load and the subgrade weight, using formula (7):

$$\delta_{zh} = \delta_z + \delta_{zg} = 1,9 + 0,065 = 1,965 kg / cm^2 .$$

Check the strength condition by formula (5):

$$\delta_{zh} \le [\delta_{zh}],$$

$1,965 \dfrac{\mathrm{kg}}{\mathrm{cm^2}} \le 1,23\,\mathrm{kg/cm^2} -$ the condition is not met.

In order to increase the strength of the subgrade and avoid damage caused by excessive deformations or shifts in the strip pile foundation, it is necessary to use geocell material [12]. The geocell material (flexible grillage) is used as part of the pile foundation stabilization system - it allows to redistribute the load arising in the inter-pile space to the piles and maximize economic effect of pile driving in soft ground.

Through the use of flexible grillage, the following is ensured:

- Pile placement at a distance $B_2 = 564\,\mathrm{cm}$ and more from each other in the pile strip foundation and between adjacent strip pile foundations depending on the tensile strength of the geocell material;
- creating an obstacle to horizontal lateral movements of the filled soil;
- no need to install inclined piles on the subgrade base.

It can be guaranteed that the limiting state at which the amplification elements (geocell material) can be destroyed upon reaching the estimated lifetime will not occur if the following condition is met:

$$\frac{T_D}{f_n} \ge T_r \, , \tag{10}$$

where $T_{rp}-$ estimated tensile strength acting on the geocell material; T_D- tensile strength of the geocell material; $f_n=1{,}0-$ reliability coefficient account for the structure (road) class, taken from [11].

Due to significant differences in the pile deformation characteristics and surrounding soft ground, vertical stress distribution along and across the road base is uneven. In this regard, there may be an arch effect.

The arch effect that occurs in the soil between adjacent pile heads causes additional vertical stresses on the pile heads. The ratio of vertical stresses on the pile heads to the average vertical stresses at the base of the subgrade $\frac{P_c^{'}}{\sigma_c^{'}}$ can be estimated by Martson's formula [10] for designing underground water pipelines:

$$\frac{P_c^{'}}{\sigma_c^{'}}=\left[\frac{C_c d_{01}}{h}\right]^2 \tag{11}$$

$$P_c^{'}=\sigma_c^{'}\left[\frac{C_c d_{01}}{h}\right]^2$$

where $P_c^{'}-$ additional vertical stress on the pile head that occurs between adjacent heads as a result of the arch effect in the ground, $\mathrm{kg/cm^2}$; $\sigma_c^{'}=1{,}965\ \mathrm{kg/cm^2}-$ total stress from the subgrade weight and the moving transport load, taking into account the correction coefficients, which is determined by the formula $C_c=3{,}07-$ arch coefficient for piles.

Substitute specific values $\sigma_c^{'}$ и C_c into formula (11) and obtain additional vertical stresses on the pile heads, as a result of the arch effect occurring in the soil between the adjacent pile heads:

$$P_c^{'}=\sigma_c^{'}\left[\frac{C_c d_{01}}{h}\right]^2=1{,}965\left[\frac{3{,}07\cdot 30}{50}\right]^2=6{,}67\,kg/cm^2$$

Then, according to the formula given in [11], the distributed load is calculated, which is taken by the geocell material between the adjacent pile heads:

$$W_T=481{,}28\,kg/cm^2\ , \tag{12}$$

Knowing the value of the load that is taken by the geocell material between the adjacent pile heads W_T, it is possible to determine the total tensile force in the geotextile material by the formula:

$$T_{rp}=\frac{W_T}{2d_{01}}\sqrt{1+\frac{1}{6\epsilon}} \tag{13}$$

where $\epsilon=4\%-$ maximum possible ultimate stretch of geocell material determined by the formula:

$$\int=0{,}04\left(s-d_{01}\right)=0{,}04\left(614-30\right)=23{,}36\,cm; \tag{14}$$

$$T_{rp}=\frac{W_T}{2d_{01}}\sqrt{1+\frac{1}{6\int}}=\frac{481{,}28}{2\cdot 30}\sqrt{1+\frac{1}{6\cdot 23{,}36}}=8{,}05\,kg/cm^2\,.$$

After obtaining the value of the total tensile force in the geocell material T_{rp}, it is possible to determine the maximum tensile force per unit of the geocell material T_r by the formula:

$$T_r=T_{rp}\cdot\delta=8{,}05\cdot 0{,}12=0{,}97\ \ kg/cm\,, \tag{15}$$

IOP Conf. Series: Materials Science and Engineering **451** (2018) 012044 doi:10.1088/1757-899X/451/1/012044

where $\delta = 0{,}12\,cm -$ wall thickness of the geogrid; taken from [12].

Verify the condition of geocell material tensile strength according to formula (10):

$$\frac{T_D}{f_n} \geq T_r \ ,$$

where $T_D = 5\,\text{kg / cm} -$ tensile strength of geocell material (spatial geogrid of "ST" brand with cell size of 20x20 cm and height of 15 cm); taken from [12].

$$\frac{T_D}{f_n} \geq T_r, \ \ when\ f_n = 1 \rightarrow 5\text{kg / cm} \geq 0{,}97\,\text{kg / cm} - \text{the condition is met.}$$

Efficiency of the construction significantly increases if the geocell material is laid on the geotextile, allowing to prevent passage of aggregate particles through geocell material into soft ground.

5. Conclusions

As a result of experimental and theoretical studies, the author's certificate for the utility model of the Russian Federation No. 156221 [13] was obtained. Application of soil strengthening technology using double-cone hollow piles, geocell and geotextile materials with properties to increase the bearing capacity of soft, heaving, seasonally freezing soils is of particular interest. This is due to the novelty approach to solving this engineering problem in the construction of roads under difficult climatic and engineering-geological conditions. The subgrade construction on a strip pile foundation of double-cone hollow piles reinforced with geocell and geotextile materials can be considered as one of the alternatives for subgrade construction with replacing soft ground.

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