

Application of thawed and peat soils for construction of road-bed for automobile roads

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Abstract: The presence in the permafrost zones of loose rocks unsuitable for the erection of embankment set a number of problems for science before converting them into a group of suitable rocks. The practice of transport construction in the specific conditions of Siberia for the last two decades has accumulated experience and expanded the list of local materials used in the erection of embankments, namely frozen clay and peat soils. The strength, reliability and durability of such soil massifs depend on their water-heat regime and stress-strain state. In the developed physics-mechanical model, the principal moment that distinguishes it from the known ones is a sufficiently detailed account of the mutual influence of the temperature field, the distribution of moisture, ice content, as well as stresses and deformations. Theoretical premises were tested in experimental construction.

1. Introduction

A vast region in Russia is occupied by permafrost soils (over 11 million km², which is almost 63% of the territory) with unique climatic and ground-geological conditions which are unique in complexity and variety and create significant difficulties in the design and construction of both roads and railways. The complexity of conditions is determined by the almost fully absence of stone materials and the widespread distribution of silty, fine dispersed and wetland clay and peat soils. The road construction requires large material costs, from 3 to 5 times higher than in the middle part of Russia. The current normative documents (Industry Building Code 84-89 "Research, design and construction of highways in permafrost regions" and the Interstate Standard GOST 33149-2014 "Roads of common use for motorways, the rules for designing highways in difficult conditions") are in most cases not ideal and impeccable, especially since the first ones were created 30-40 years ago, while the latter do not go beyond the framework of SNiP 2.05.02-85, developed more than 50 years ago and updated in the Design and Construction Specifications 34.13330.2012. [1]. The reasons for the roads failure in these conditions are not so much the impact of vehicles, but the natural factors effect on the road. These problems are the most complex and have not been yet adequately studied. Over the past 50 years, the practice of road construction in Siberia has shown that when roadbed construction an expansion of the use of local materials such as wetlands of frozen and thawed clay and peat soils should be considered as the most effective ways to reduce costs and increase in the pace of roads construction. An acute problem is the forecasting of the formation of the embankment and the adoption of measures to ensure the stability and durability of the roadbed on the basis of constructive-technological and organizational solutions.



2. Research

The operating procedures have the following drawbacks: one-layer structures are considered, one-dimensional problems are solved, only final precipitations are considered, not taking into account that they go in time. As a result, not enough reliable calculations are obtained.

3. Results and discussion

The aim of this work is the development of a complex model for forecasting the water-heat regime and the stress-strain state of the wetland thawed and frozen clay and peat soils of the disturbed structure in the embankment body during its formation at the stage of construction and operation, taking into account the model universality and the analysis simplicity of individual determinants phenomena.

Physical principles of modeling are: the law of conservation of energy (heat balance) in the form of an equation of thermal conductivity, taking into account the heat of phase transitions, the equation of the moisture balance in different phases, determining the saturation in different parts of the investigated region, and also the equations determining the stresses and deformation.

For an arbitrary small but finite volume V , the soil components averaged over the volume are: the ground densities $\rho_1 = \frac{m_1}{V}$, water $\rho_2 = \frac{m_2}{V}$, ice $\rho_3 = \frac{m_3}{V}$ and gas $\rho_4 = \frac{m_4}{V}$ are found from the integral law of conservation of mass [2]:

$$\int_V \frac{dp_i}{dt} dv = - \int_S p_i v_i^n ds + \int_V \sum_j J_{ji} dv \quad (1)$$

where: $J_{ji} = -J_{ij}$ have the dimension $\text{kg/m}^3 \cdot \text{s}$ and characterize the intensity of the phase transition of the mass from j to the i^{th} component per unit volume of the medium per unit time, that is J_{32} characterizes the intensity of ice melting, J_{23} - hardening of water; J_{42} - condensation of steam; J_{24} - evaporation of

water; $p_i v_i^n$ are the projections onto the external normal to the surface S of the phase fluxes, that is $p_i v_i^n = (J_i)^n$; $p_i = \frac{m_i}{V}$; .

The change in the averaged mass density of the soil skeleton $p < l$, water p_2 and ice p_3 can occur due to water migration, phase transitions and changes in soil porosity.

Velocities \bar{v} and flows \bar{J}_i of matter in the solid phase: for the soil skeleton $\bar{J}_1 = p_1 \bar{v}_1$ and ice $\bar{J}_3 = p_3 \bar{v}_3$ are exclusively related to the deformation motion and are determined by the equations of the stress-strain state of the soil [3,4]:

$$\bar{v}_1 = \bar{v}_3 = \bar{v}_{\text{def}}(x, y, z, \sigma_i, \tau_{ij}, t) \quad (2)$$

where: $\sigma_i (i = x, y, z)$, $\tau_{ij} (ij = x, y, z)$ are the normal and tangential stresses in the soil, determined by external loads on the roadbed, the pressure of the overlying layers and the soil properties.

The flow of liquid moisture in a colloidal capillary-porous body in the general case is due to diffusion, convective and effusion flows [3] and is determined by Onsager's linear relations:

$$\bar{J}_2 = p_2 \bar{v}_2 = -\lambda_{2m} \nabla \theta - \lambda_{2m}^T \nabla T - k_2 \nabla P \quad (3)$$

where: $\nabla \theta = \frac{1}{c_m} (\nabla w)_T$ – gradient of moisture transfer;

$c_m = \left(\frac{dw}{d\theta} \right)_T$ – isothermal specific moisture capacity;

$\lambda_{2m} = a_{2m} p_1 C_m$ - coefficient of moisture conductivity;

$\lambda_{2m}^T = \frac{\lambda_m}{C_m} \delta$ - coefficient of thermal conductivity

For each reference volume V at each time t , the humidity and ice content are calculated from the mass conservation equations (1):

$$W = \frac{m_2}{m_1} = \frac{p_2}{p_1} \quad \text{and ice content} \quad L = \frac{m_3}{m_1} = \frac{p_3}{p_1}, \quad (4)$$

The model of the heat and moisture regime of the soil in the integral form is written using (4) as:

$$\int_v \frac{dp_1}{dt} dV = - \int_s p_1 (\bar{v}_1)^n ds \quad (5)$$

$$\int_v \frac{dp_2}{dt} dV = - \int_s [\nabla \lambda_{2m} (\nabla \theta)^n + \lambda_{2m}^T (\nabla T)^n + k_2 (\nabla P)^n] dS + \int_v J_{32} dV + \int_v J_{42} dV \quad (6)$$

$$\int_v \frac{dp_3}{dt} dV = - \int_s J p_3 (\bar{v}_3)^n ds - \int_v J_{32} dV \quad (7)$$

$$\int_v \frac{dp_4}{dt} dV = - \int_s [\lambda_{4m} (\nabla \theta)^n + \lambda_{4m}^T (\nabla T)^n + k_4 (\nabla P)^n] dS - \int_v J_{42} dV \quad (8)$$

The heat balance equation can be represented as:

$$\int_v \frac{d(pH')}{dt} dV = - \int_s \lambda (\nabla T)^n dS, \quad (9)$$

where $H = \int_0^T C_{ef} dT$ is the equivalent enthalpy of the soil mixture, determined taking into account the presence of skeleton soil material, ice, unfrozen water and steam-air mixture.

The initial data for the analysis of the heat and humidity and stress-strain state of the soil are given at the initial time $t = 0$ in the form: $T = T_0(x, y, z, o)$; $p_i = p_i(x, y, z, o)$; where $v_j = 1, 2, 3, 4$ (9) in the whole considered calculation area. The boundary conditions for heat and moisture transfer are given on the outer surface by [4] and at a distance to the depth from the surface. Calculation of the humidity temperature in non-stationary conditions on the external surface is determined by:

$$\lambda_{3F} \frac{p_F}{b_F} \left(\frac{dW}{dn} \right)_F + \lambda_{3F} b_F \left(\frac{dT}{dn} \right)_n + \lambda_3 \frac{p_F}{b_F} (W_F - W_B) = 0 \quad (10)$$

Equations (5-9) are a mathematical integral form of recording the basic laws of frozen ground physics - the equations of conservation of mass and energy. The equations of conservation of mass and energy in differential form are written according to [3]:

$$p_0 \frac{du_i}{dt} = -d_i V \left[\bar{j}_{i \text{ dif}} + p_i k \frac{k_i}{\eta_i} (\nabla P - p \nabla h) \right] + I_i \quad (11)$$

The water pressure in the soil can be determined from the equation of soil consolidation in the form of V.A. Florin for the control volume V :

$$\int_v \frac{\partial H}{\partial t} dV = \int_v \left(\nabla \frac{1}{\omega} \frac{\partial H^*}{\partial t} + \frac{1}{2\gamma\omega} \frac{\partial \sigma_z^*}{\partial t} \right) dV + \int_s \frac{1+\theta}{2\gamma\omega} \left(\frac{d\theta}{d\theta} \right) k \nabla H^n dS \quad (12)$$

$$\omega = 1 - \frac{\beta(1+e)}{2 \frac{(de)}{d\theta}}; H^* u \sigma_z^*$$

where H^* is the water pressure and the sum of the stresses in the soil skeleton at the end of consolidation, i.e. in a stabilized state.

The quantities H^* and θ^* do not change in the future and the equation of consolidation according to V.A. Florin [4] has the form:

$$\frac{dH}{dt} = C_v \nabla^2 H \quad (13)$$

where: $C_v = (1+e)(1+\xi) R/2\gamma\omega a$ - coefficient of consolidation.

Under the influence of stresses σ_s the volume v , which coincides with the control volume at the time instant t , decreases. In this case, the soil skeleton v_s is assumed to be constant. The pore volume

$v_n = v - v_s$ decreases according to the law of damped hereditary creep on ΔV_{def} [3]:

$$\Delta V_{def} = \varepsilon_{v_{def}} V = \frac{e(t_1) - e(t_2)}{1 + e(t_1)} V, \quad (14)$$

The total change in volume v during the time $\Delta t = t_2 - t_1$ due to the action of the stresses σ_Σ and the phase transition is determined by the following formula:

$$\Delta V_p = \Delta V_{def} + \Delta V_{phase \ tr.} \quad (15)$$

The coefficient of volume strain or $\varepsilon_v = \frac{\Delta V}{V}$ is equal to the sum of the relative longitudinal strains [2, 3]:

$$\varepsilon_v = \varepsilon_z + \varepsilon_x + \varepsilon_y = (1 + \mu_y) \varepsilon_z, \quad (16)$$

where: $\varepsilon_v = \mu_y \varepsilon_z$; $\mu_H = a \left(\frac{y}{b'/2} \right)^n$ when $Z > Z_0$ is the coefficient taking into account the possibility of transverse soil deformations; $\varepsilon_x = 0$ under the assumption that there is no deformation in the X direction along the road axis.

Equations (16) and (15) yield expressions for the relative strains $\varepsilon_x, \varepsilon_z, \varepsilon_y$ of the linear strains S_z and S_y in the directions Z and Y and the deformation rates v_{1z} and v_{1y} :

$$\varepsilon_z = \frac{\varepsilon_v}{1 + \mu_y} = \frac{\Delta V_{def} + \Delta V_{phase \ tr.}}{V(1 + \mu_y)}; \quad \varepsilon_y = \mu_y \varepsilon_z; \quad (17)$$

$$S_z(t, z, y) = \int_0^z \varepsilon_z(t, z, y) dz; \quad S_y = \int_0^y \varepsilon_y(t, z, y) dy; \quad (18)$$

or with the condition of constancy of ground movements in the plane of the surface of the embankment, it can be approximately taken:

$$V_{1z} = \varepsilon_z \frac{\Delta z}{\Delta t}; V_{1y} = \varepsilon_y \frac{\Delta y}{\Delta t};$$

$$\bar{v}_1 = \bar{v}_1 \left(\varepsilon_z \frac{\Delta z}{\Delta t}, \varepsilon_y \frac{\Delta y}{\Delta t}; \right). \quad (19)$$

To study the processes of stabilizing the embankments of clay soils erected in winter during 2009-2013, the Department of Highways and Aerodromes of Tyumen State Architectural and Construction University conducted field research on cluster sites and highways of the OOO TNK-Uvat field network. The research task consisted of:

- 1 – study of the dynamics of settlements of soil embankments erected of clay soil at a negative temperature (in winter);
- 2 – study of the dynamics of the temperature mode and physics-mechanical characteristics of the soils during thawing (in summer).

Samples were collected at the study sites to determine the physics-mechanical characteristics of the soil. To control the settlement of the embankments, metal marks were set at the stage of the embankment erection (Figure 1).



Figure 1. Stamps setting.

To determine the vertical and horizontal distribution of temperature fields, the temperature sensors DS18S20, located at the base of the roadbed along the axis, the break points and on the surface of the deposited layers, were used. The deformations of the embankments of loam (Figure 2) with moisture content of 10%, 19% and 27% with freezing-thawing are significantly different when compared with a sandy embankment. When frozen, its deformations occur, which magnitude is more from 2.5 to 3 times. In the initial stage of thawing, there is a sharp swelling and an increase in deformations of up to 20%, followed by attenuation. Compared to sand, a residual deformation equal to half of the total one

is observed. The curve K_y is inversely proportional to the strain curve. The initial value of K_y is 0.97 then it drops to 0.925 and stabilizes. At the stage of swelling, a decrease in K_y continues, after which it assumes a constant value, but less than tolerated 0.92. Thus, after one cycle of freezing-thawing of cohesive soil, the embankment is decompressed to values less than those permitted by regulatory documents.

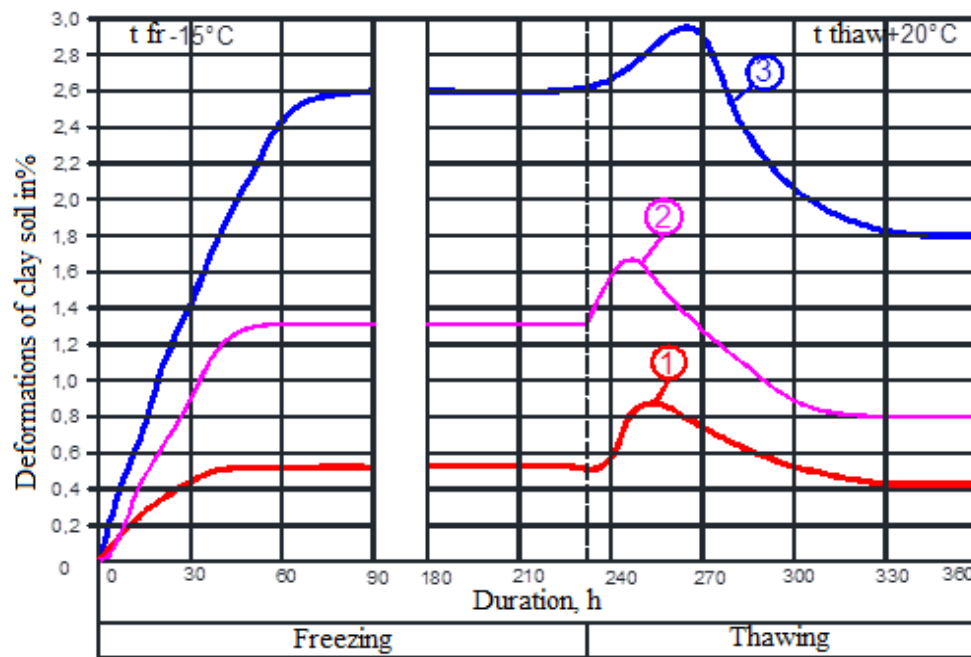


Figure 2. Deformations of embankment of loam during freezing and thawing
1- humidity -10%; 2 - humidity of 19%; 3 - humidity 27%.

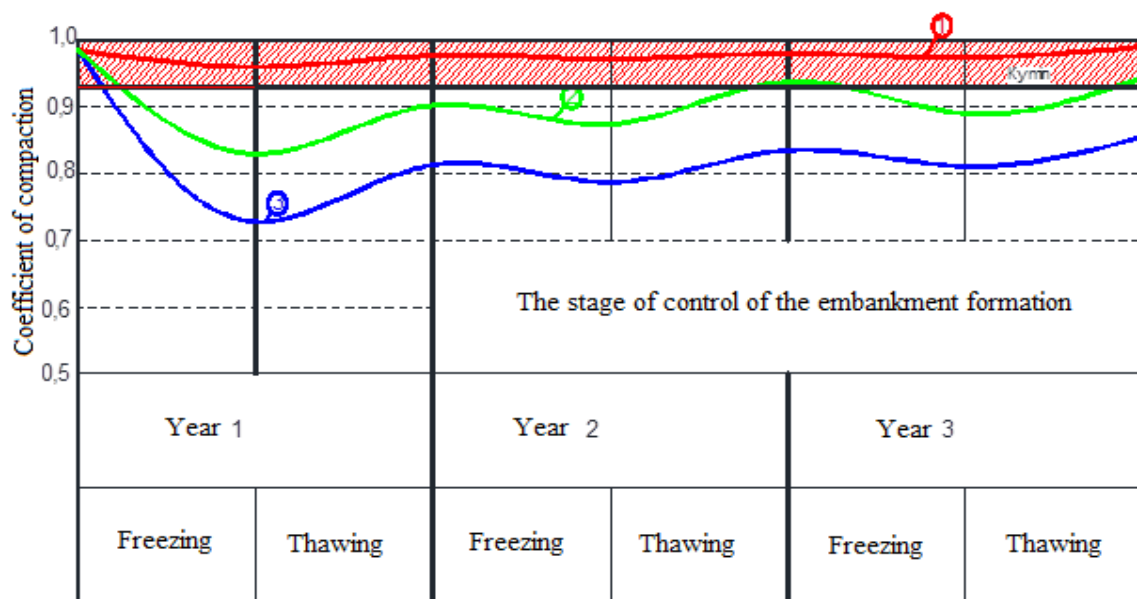


Figure 3. Dynamics of soil compaction factor for the embankment
1 - fine sand $W=7\%$, 2 - loam $W=19\%$; 3 - loam $W=27\%$.

Figure 3 shows the dynamics of the coefficient of compaction of various soils. Analysis shows that sandy soils embankments are formed after the first freeze-thaw cycle. Fluctuations in density, and at the same time K_y from 1 to 0.88 for loam with a moisture content of 19%, occur within 2 years. In the first freeze-thaw cycle, decomposition of the soil to $K_y=0.8$ occurs. For loam with a moisture content of 27% the coefficient of compaction is reduced to 0.72.

The formation of a soil embankment with a humidity of 19% occurs during 2 freeze-thaw cycles, and loam with a moisture content of 27% for more than 3 years.

4. Conclusion

The main factors influencing the formation of the roadbed from "substandard" soils were determined and substantiated; an algorithm and computer program were developed; experimental field and laboratory studies were conducted; the monitoring of the built-up road sections in territories occupied by permafrost soils was carried out.

The developed physical and mathematical model allows calculating the variation in time and space of the main parameters of frozen and dehydrated soils: humidity, $w(x, y, z, t)$; the ice content, $L(x, y, z, t)$; temperature, $T(x, y, z, t)$, normal and tangential stresses in the soil $\sigma_i(x, y, z, t)$, $\tau_{ij}(x, y, z, t)$, and also deformations $\epsilon_i(x, y, z, t)$. A quasi-three-dimensional approach to the mathematical modeling of the roadbed has allowed us to obtain computational equations with two independent variables y and z , and a time t , which allows us to analyze the development of processes in time. The conducted field research and monitoring showed good convergence with theoretical assumptions.

References

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