Ground Freezing Applied to Mining and Construction

Application de la congélation artificielle des sols dans l'industrie minière et la construction

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Summary

The authors describe the successfull application, in construction work in the U.S.S.R., of artificial freezing of the ground under complex conditions of the sinking of deep escalator shafts for underground railways ; in the protection of foundations from powerful streams of groundwater, and in mining. The results of investigations of the artificial freezing process are given, together with formulae for determining the freezing time and time of closing of ice-soil cylinders, and methods of engineering calculation for strength and creep of temporary ice-soil barriers are substantiated.

Sommaire

Ce rapport décrit une application réussie de la congélation artificielle des sols dans les Travaux Publics en U.R.S.S., dans des conditions compliquées, devant les profondes entrées du métropolitain, et la protection des fouilles contre les fortes venues d'eaux de sous-sol et dans l'industrie minière. Il comporte les résultats des recherches portant sur le mode de congélation, et donne des formules qui définissent le temps de congélation et de fermeture des enceintes de sol congelé ainsi que des méthodes de calcul de la résistance et du fluage des parois provisoires de sol congelé.

1. Field of application of artificial freezing *

Artificial freezing of ground is being ever more widly applied in the U.S.S.R. in excavation (without lining) of deep open building excavations, for example, in the construction of underground vestibules and escalator shafts for underground railways, for protection of the foundations of hydraulic structures from the influx of groundwater, and in sinking deep shafts in unstable soils [1.2.].

The following examples are from building experience in the U.S.S.R. and will serve to illustrate the process.

(a) During the construction, in 1949-50 in Moscow, of a high frame building (138·5 metres in height), excavation for an underground railway station (24 m deep, 50 m long, and 22 m wide) was simultaneously proceeding next to the building in soft ground up to 22 m deep, with an underlying stratum of limestone, and without the use of lining inside the excavation, which was protected only by an artificially made ice-soil wall (Fig. 1) carried out under the supervision of engineer Ya. A. Dorman.

The foundation of the central part of the high building was made of 26·1 × 44·1 m (in plan) channel-type reinforced concrete (Fig. 2).

A preliminary calculation of the strength of the ice-soil wall, with due allowance for relaxation of stress, which was carried out by the authors, made it possible to dispose of all support inside the excavation, thus making it possible to mechanize excavation work and considerably reduce building time.

(b) Using artificial freezing for the building of a percolation-proof cofferdam (1 200 metres long and 19-21 metres deep) in alluvial and tertiary sands up to 13·1 m in depth,

Fig. 1 General view of ice wall protecting deep escalator shaft of underground railway.

Aspect général du mur de sol congelé qui entoure la fouille profonde de l'entrée du métropolitain.

* Sections 1 and 3 were written by Prof. N. Tsytovich; Section 2, by Prof. K. Khakimov.

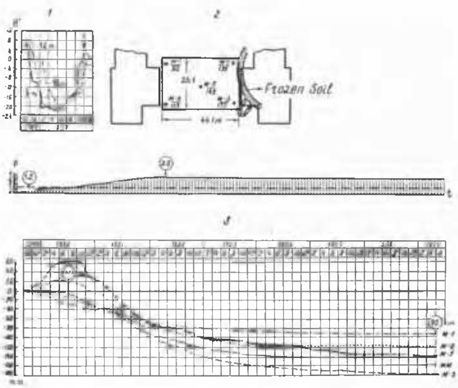

Fig. 2 Results of observations of uprise and subsidence of the foundation of a high building in the case of a deep open excavation protected by a wall of frozen soil : 1. soil temperature graph at various depths ; 2. foundation scheme (the protective wall made of artificially frozen soil is cross-hatched) ; 3. result of observations of uprises and subsidence of the central foundation of a high building.

Résultats des observations sur le soulèvement et le tassement de la fondation d'un grand immeuble lors de l'exécution d'une fouille ouverte profonde, protégée par un mur de sol congelé : 1. graphique de la température du sol à diverses profondeurs ; 2. schéma des fondations (la paroi protectrice de sol congelé artificiellement est indiquée par des hachures) ; 3. résultats d'observations de soulèvements et de tassements des semelles de fondation d'un grand immeuble.

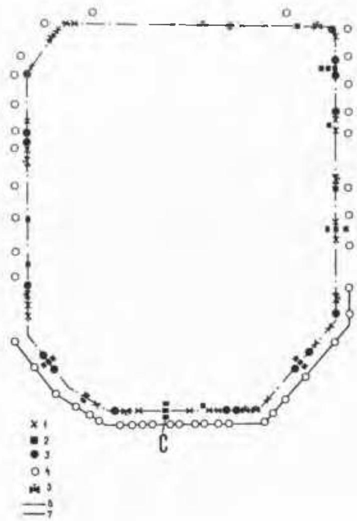

Fig. 3 The distribution of boreholes for an ice wall of an open excavation : 1. water gauge ; 2. temperature holes ; 3. freezing holes ; 4. piezometers and well points ; 5. Ludlot gates ; 6. axis of ice wall ; 7. line of water-lowering well-points.

Distribution des sondages ayant servi à congeler la paroi de la fouille ouverte : 1. compteur à eau ; 2. sondages thermométriques ; 3. sondages de congélation ; 4. Piezomètres et wellpoints ; 5. jauges de Ludlow ; 6. axe du rideau de sol congelé ; 7. ligne de wellpoints pour l'abaissement du niveau d'eau.

with 18 to 20 metres of underlying (in alternating strata) marl, clay, and compact clay, one of the authors [4] succeeded in effecting a reliable protection from groundwater of an excavation for a hydro electric power station. Additional groundwater lowering was necessary, and the wall was intact for about two years (Fig. 3).

(c) The artificial freezing of deep soft ground is almost the only rational method applicable to sinking deep shafts under complex geological conditions. The theoretical calculation of the quantity of frozen soil is extremely important in practice (as the load-carrying structure) as well as maximum stress and the limit of permissible plastic deformation.

2. The theory of artificial freezing and its applications

A rigorous solution of the problem of soil freezing is exceedingly involved, since so many factors must be taken into account ; crystallization of free water when the ground is cooled to freezing point, further partial freezing of bound water as the temperature is taken below zero, and changes in the humidity and thermo-physical properties of the ground in the greezing process are some of them.

The author derived a formula for determining the freezing time of homogeneous soil of a given radius (ξ) on the basis of the following assumptions [4] :

(a) undetermined motion of heat is regarded as a succession of stationary states ;

(b) the radius of efflux of heat (R) and the freezing radius (ξ) are directly related linearly, that is, we have $R = a \cdot \xi$, where a is a non dimensional constant of proportionality ;

(c) the average values of freezing and humidity of the ground are regarded as constants.

The resulting equation is of the form :

$$\tau = \frac{1}{2\lambda_1 \theta}\left[(1-n)W\gamma i_0 \sigma + c_1\gamma_1\theta_s + c_2\gamma_2\theta_s\left(\frac{a^2-1}{2 ln a} - 1\right)\right] \cdot \left(\xi^2 ln \frac{\xi}{r_0} - \frac{\xi^2 - r_0^2}{2}\right) + \frac{c_1\gamma_1}{2\lambda_1} \cdot \frac{\xi^2 - r_0^2}{2} \quad \ldots. \quad (1)$$

where τ : is the freezing time in hours, λ_1 : is the coefficient of thermal conductivity of the frozen ground in kg cal. per metre per hour per degree Centigrade, θ : is the absolute value of negative temperature of the soil (in °C) adjoining the surface of the freezing pipe (according to observational results carried out by the author, the value of θ may be taken at 2° above the temperature of the coolant), n : is the porosity, w : is the humidity, γ : is the specific weight of the ground in kg per cub. m, i_0 : is the intensity of freezing, σ : is the heat of crystallization of water in kg cal. per kg, c_1 and c_2 are the specific heats of frozen and thawed ground, respectively, in kg per cub. m, γ_1 and γ_2 are the weights of frozen and thawed ground, respectively, in kg per cub. m, θ_s : is the natural temperature of the ground in °C, and r_0 : is the radius of the pipe.

According to the findings of special experiments and of field observations carried out by the author, $a = 4{\cdot}5$ for the case of plane thermal flow, and $a = 3$* for the freezing period up to complection of the ice cylinders.

* See Reference [3].

An analysis of formula (1) shows that, other conditions being equal, the time required for the formation of an ice-soil cylinder of radius ξ is inversely proportional to the absolute value of the negative temperature of the coolant, whereas freezing is directly proportional to the square of the freezing radius.

Comparisons of formula (1) with the solutions of the same problems on a hydro integrator showed good coincidence, while comparisons with the results of field observations yielded discrepancies not exceeding 15 per cent (4).

The criterion for retaining the natural texture of the sand during freezing is established by the following formula :

$$K_0 \geqslant \frac{n\alpha}{i\left(2A\ln\frac{\xi}{r_0} + B\right)\xi} \qquad (2)$$

where α : is the coefficient of expansion of the water during its transformation into ice ($\alpha = 0{\cdot}091$), K_0 : is the coefficient of percolation (m/hr), and i : is the hydraulic gradient.

$$A = \frac{1}{2\lambda_1\theta}\left[(1-n)W\gamma\cdot 0{\cdot}91\sigma + c_1\gamma_1\theta_s\left(\frac{a^2-1}{2\ln a} - 1\right)\right];$$

$$B = \frac{c_1\gamma_1}{2\lambda_1}$$

The remaining designations are as described earlier.

Condition (2) is applicable to a layer of sand (undergoing freezing) on the surface or with outcrops to the ground surface.

Numerical calculations show that when a layer of sand undergoing freezing lies between two layers of water-repellent material, condition (2) may be fulfilled with outcropping of the sand layer on the ground surface even at a distance of several kilometres from the freezing section, depending on the coefficient of percolation and the depth of occurrence of the sand layer.

In the field of soil freezing for construction purposes, a highly complex problem has arisen, which is to overcome the effects of *percolation flow* on the freezing of sands. Experience has shown that in certain cases, when this rate of flow is high, it is difficult to seal the cylinders of frozen soil.

In the process of freezing highly filtering soils, a state of thermal equilibrium may be created, so that the freezing radius no longer increases and the frozen cylinders of ice do not close.

In forming the above-mentioned ice-soil wall (1 200 metres in length), the design called for the following : distance between freezing pipes 1·4 m, coolant temperature — 16° C. The project did not take into account the effect of percolation flow. The principal percolation flow passed through the freezing section B-C-D-E-G (Fig. 3) and proceeded from the direction of the bank into the contour undergoing freezing. The average rate of flow before freezing began was 2·5 m/24 hr, while in the gravelly layer at the foot of the alluvial sands the rate was higher.

The data characterizing the state of thermal equilibrium are given in Fig. 4. In this state, the freezing pipes absorbed only as much heat as was left by the passing stream, and there was no increase in the freezing radius.

As may be seen from Fig. 4, in claycy soils (depth : 11-19 metres) in which there was no percolation flow, the temperature after one month of freezing reached freezing point. In sands with percolation flow, the temperature stayed at + 5° C for a long time.

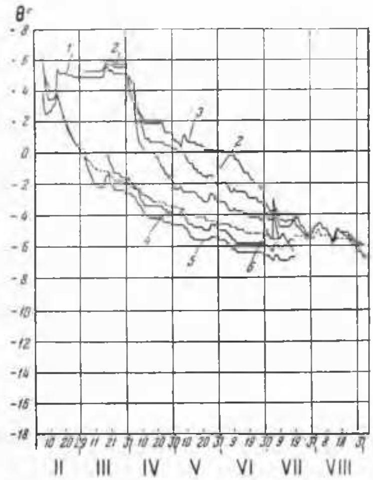

Fig. 4 Variation of temperature in thermal well "C" at various depths : 1. 3·72 m ; 2. 5·83 m ; 3. 8·33 m ; 4. 11·54 m ; 5. 12·64 m ; 6. 18·94 m.

Variation de température dans le sondage « C » à diverses profondeurs : 1· 3·73 m ; 2. 5·83 m ; 3. 8·33 m ; 4. 11·54 m ; 5. 12·64 m ; 6. 18·94 m.

This state of thermal equilibrium is described by the equation :

$$\frac{2\pi\lambda_1\theta 24}{c_w n\gamma_w l\varkappa\ln\left[(1-l_0)\frac{l}{2r_0}\right]} = \Delta t_\Phi v_{np} = \text{const} \qquad \ldots\ldots (3)$$

where C_w : is the specific heat of water in kg cal. per kg per Deg. C, γ_w : is the specific weight of the water in kg per cub.m, l is the distance between the freezing pipes in metres, Δt_Φ : is the temperature difference of flow before the ice wall and after it (in practice, in considered case $\Delta t\varphi = 2\text{-}3°$ C), v_{np}: is the rate of flow before commencement of freezing in m/24 hr, and $\varkappa$: is the coefficient of consumption.

$$\varkappa = l_0 - (1-l_0)^2\left[\sum_{k=-m}^{k=+m}\frac{1}{2k-(1-l_0)} - \sum_{k=-m}^{k=+m}\frac{1}{2k-1}\right] \qquad (4)$$

k assumes any value from $-m$ to $+m$, where $2m+1$ is the number of greezing pipes, and l_0 is the relative distance between the outer surfaces of the ice-soil cylinders, equal to $l_0 = \frac{l-2\xi}{l}$.

In this case, the condition for closure of the ice-soil cylinders is expressed by the flolowing formula :

$$l\ln\left[(1-l_0)\frac{l}{2r_0}\right] \leqslant \frac{2\pi\lambda_1\theta\times 24}{n\gamma_w[c_w(\Delta t_\varphi)v_{np}\chi + \pi(1-l_0)(\Delta\xi)\sigma]} \qquad (5)$$

In Eq. (5), $\Delta\xi$: is the increment in freezing radius in 24 hours under the condition of no flow.

Investigations have shown that thermal equalibrium sets in at $l_0 \approx 0{\cdot}4$. In this connection, it appears possible to determine Δt_0 from equation (3).

In the temperature of the coolant, the initial rate of percolation flow, and the other quantities are given, it is possible to satisfy condition (5) by selecting the appropriate distance between pipes (l).

Practically, the effect of percolation flow was overcome by carrying out the following measures :

(a) diminishing the distance between the freezing pipes;

(b) reducing the temperature of the coolant (to — 25° C), and

(c) reducing the rate of percolation flow by ground-water lowering through the use of well points placed in front of the wall.

These measures produced positive results, and excavation was successfully carried out behind the protection of artificially frozen soil.

3. The mechanical properties of frozen soils and calculation of ice-soil barriers on the basis of limiting states

As has already been established (1, 2, 5 and 6) a study of the mechanical properties of frozen soils involves distinction being made between instantaneous (maximum) strength, reduction in strength with time, and minimum strength. The reduction in strength of frozen soils with time is due to relaxation of resistance and differs for different types of soil. Thus, in the case of rupture this reduction will be slight, falling to 1/10-1/15 of the instantaneous resistance, while for shear it is somewhat compensated for by the action of the normal compacting pressure, since the resistance to friction increases with increasing pressure. The magnitude of the forces of cohesion of frozen soils (the principal component of their resistance to shear) diminishes in time by different values, depending on the composition of frozen soils and the extent to which they are frozen, as well as the value of negative temperature; the reduction attains from 1/3 to 1/8 of the value of instantaneous cohesion.

Thus, the principal strength characteristic of frozen soil is not their instantaenous resistance, but the resistance corresponding to a given time of action of the load.

Appropriate studies (2, 5, 6) show that in strength calculations of frozen soils the theory of tangential stresses (More) is sufficiently applicable. According to this theory, the limiting resistance to shear is a function of normal stress and depends on the temperature ($-\theta°$) and time of action of the load (t), that is, $\tau_{\Phi,t} = f(\sigma_{\Phi,t})$. In practical calculations, the envelope of the circles of limiting stress (for $\sigma_1 \leqslant \frac{\sigma_d}{2}$, where σ_d is the resistance to uni-axial compression) may be taken to be rectilinear (Fig. 5*a*). Experimental determinations, carried out by the authors, of *prolonged* resistance of frozen ground show that the angle of inclination of the envelope of circles of *prolonged* resistances may, in practical calculations, be considered as independent of the negative temperature and equal to the angle of internal friction of unfrozen soil, whereas prolonged cohesion will, largely depend upon the value of negative temperature. This latter fact enables the author to propose that for frozen soils the following very simple practical construction of the envelope of circles of limiting stresses may be used : by applying a spherical die suggested by the author [2], a determination is made of prolonged cohesion of frozen ground $c_{\sim} = 0{\cdot}18\ P/\pi D h_{\sim}$, where P : is the load on the spherical die, D : is the diameter of the die, and $h_{\sim}$:is the total subsidence of the die, while the angle of inclination of the diagram of prolonged resistance to shear is equated to the angle of internal friction of unfrozen ground.

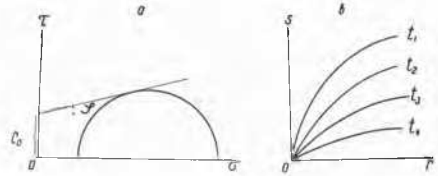

Fig. 5 Limiting circle of stresses and curves $S = f(\Gamma)$ for frozen soils. (a) envelope of circles of limiting stresses; (b) the intensity of shear (S) as a function of the intensity of rates of deformation to shear (Γ).

Cercle de Mohr et courbes $S = f(\Gamma)$ pour sols congelés. (a) droite enveloppe des cercles de rupture; (b) relations entre le déplacement (S) et la vitesse de déformations Γ.

In calculations of ice-soil barriers on the basis of the permissible deformation due to creep, the most acceptable is the theory of strength of Mizes-Botkin [7], based on the dependence of intensity of shear stress S on the intensity of deformations of shear (Γ). In the case of non linear-viscous flow, this relationship may be taken to be of the power type (see, for instance, 8), which (with respect to frozen soils) does not contradict experimental findings [6], though it does not reflect them completely, because for frozen soils the resistance to tension is considerably less than the resistance to compression. Thus, we have $S = k \cdot \Gamma^m$, where k, and m are parameters of the creep curves. Investigations show that generally the quantity k is equal to several tens of kg per sq. cm, and $m \leqslant 1$.

The $S = f(\Gamma)$ curves, may be constructed from the results of tests of samples of frozen soils for triaxial and uniaxial compression, with the time factor entering parametrically (Fig. 5*c*).

To illustrate the application of these principles in practice, it may be noted that the temporary ice-wall lining of a deep excavation for the escalator shafts of the Moscow underground already been describes (see Fig. 1) was calculated by the author on the basis of prolonged resistances of frozen soils; pressure from unfrozen soil was computed on the basis of conventional formulae of the plane problem of the classical theory of earth pressure. It was found possible to take a prolonged design resistance to compression, equal to 14 kg per sq. cm, on frozen, saturated silty sand at an average temperature of the ice-wall of — 10° C. The successful excavation of a foundation 24 m deep under the sole protection solely of a wall of artificially frozen ground corroborated the accuracy of these calculations.

On the basis of appropriate tests of soil samples, a prognosis was made, for the above-described ice-wall, of the heave of soils during freezing and of compacting settlement of foundations. Subsequent detailed observations of the rises and subsidences of the foundations of a high building under construction (the results of which are given in Fig. 2 [9] have shown that the heave was nearly equal to the value predicted. Thus, the predicted value was 60 mm, and the measured value 62·6 mm. The design bend at a pressure on the ground of 3·6 kg per sq. cm was equal to 5·6 mm, while the measured value was 13 mm; the designed slope of the foundation slab was 0·001, while the measured value was 0·0017. The reason for an increase in sag and inclination was the additional subsidence of the soil after artificial freezing and subsequent thawing.

In lining deep shafts that are sunk in running ground, an essential problem is the determination of the dimensions of ice-soil cylinders on the basis of the limiting-permissible magnitude of deformation of creep of the walls of the cylinder, which arises during the time from the beginning of sinking until permanent lining is installed.

In solving this problem, use may be made of the equation $S = k \cdot \Gamma^m$ for a sufficiently long cylinder of frozen soil.

In the case of an axial-symmetrical distribution of stress, the intensity of stress and the intensity of deformations from shear are expressed by the following equations.

$$S = \frac{1}{\sqrt{6}} \sqrt{(\sigma_r - \sigma_\Phi)^2 + (\sigma_\Phi - \sigma_z)^2 + (\sigma_r - \sigma_z)^2 + 6\tau_{rz}^2}$$

$$\Gamma = \sqrt{\frac{2}{3}} \sqrt{(\varepsilon_r - \varepsilon_\Phi)^2 + (\varepsilon_\Phi - \varepsilon_z)^2 + (\varepsilon_r - \varepsilon_z)^2 + \frac{3}{2}\gamma_{rz}^2}$$

where $\sigma_r\ \sigma_\Phi\ \sigma_z$ are the normal stresses, and $\varepsilon_r\ \varepsilon_\Phi\ \varepsilon_z$ and γ_{rz} are the corresponding longitudinal and angular relative deformations.

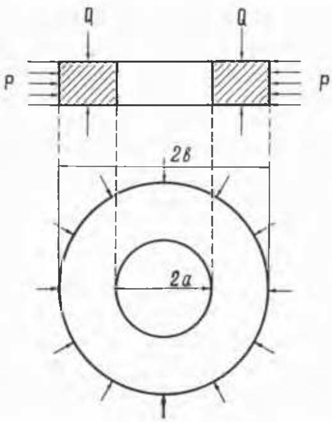

Fig. 6 Action of loads on annulus made of artificially frozen soils in the lining of shafts (schematic).

Schéma de l'action des forces agissant sur un anneau de sol congelé entourant un puits de mine.

If we consider an element of an ice cylinder as an annulus under the action of uniform lateral pressure (Fig. 6), using for the components of deformations the expressions given by Genki :

$$\varepsilon_r = \frac{\varkappa}{2G}(\sigma_r - \sigma); \qquad \varepsilon_\Phi = \frac{\varkappa}{2G}(\sigma_\Phi - \sigma);$$

$$\varepsilon_z = \frac{\varkappa}{2G}(\sigma_z - \sigma); \qquad \gamma_{rz} = \frac{\varkappa}{G}\tau_{rz}$$

and take into consideration that $\frac{\varkappa}{2G} = \frac{\Gamma}{2}$; $\varepsilon_r = \frac{du}{dr}$; $\varepsilon_\Phi = \frac{u}{r}$ nad for the conditions of plane deformation $\varepsilon_z = 0$, then, solving the well-known differential equation of equilibrium $\frac{d\sigma_r}{dr} + \frac{\sigma_r - \sigma_\Phi}{r} = 0$, with account taken of the boundary conditions of the posed problem in the general case of a parametric specification of the creep curves (Fig. 5 *c*), the Laboratory of Frozen-Soil Mechanics, U.S.S.R. Academy of Sciences, has obtained an expression for the limiting load (p) on an annulus of the ice cylinder in closed form [10] :

$$p = \frac{k}{m}\left(\frac{2u_a}{a}\right)^m \left[1 - \left(\frac{a}{b}\right)^{2m}\right] \tag{7}$$

where u_a is the quantity (specified according to the conditions of work) of deformations of creep of the ice cylinder.

In the case of a linearly deforming material ($m = 1$), expression (7) passes into the Lame formula, and in the case of $m = 0$, it passes into the well-known formula for pressures on an annulus of ideally plastic masses :

$$p = 2k \ln \frac{b}{a}$$

It is interesting to note that concrete calculations on the basis of the latter formula show that it may be used, without considerable errors for relatively small external pressures of the order of 20 to 25 kg per sq. cm; at high pressures the wall thickness of ice-soil cylinders becomes prohibitive.

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