Permafrost in the U.S.S.R. as Foundations for Structures

Étude sur le pergélisol en tant que fondations des édifices en U.R.S.S.

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SUMMARY

Permafrost, which is widespread in the U.S.S.R., occupying nearly half of the surface area of the country and ranging up to 1.5 km in depth, is classified among the soils with an unstable structure. U.S.S.R. scientists were the first to investigate permafrost not only geologically but also as a base for structures, and to work out, together with engineers, methods for erecting stable structures on it. These methods were based on a detailed study of natural factors and the use of scientific achievements in thermophysics and the mechanics of frozen soils, branches of science which are being successfully developed in the U.S.S.R.

Experimental and observational research (under natural conditions) has paved the way for the elaboration of rigorous methods for solving problems of foundation, and a number of solutions have already been obtained and successfully applied in practice. These solutions use both the method in which the frozen condition of the base soils is retained and that in which the frozen base soils are thawed. In the first case full consideration is given to the rheologic properties of frozen soils, in the second to the primary and secondary consolidation of thawing bases and their non-uniformity in depth.

In the case of shallow-seated rock, and/or where theoretical forecasts indicate that the thawing depth is moderate, use is made of the method of pre-construction thawing and soil strengthening as well as the method of erecting deep footings.

The next task is a further improvement in methods of designing bases and foundations, and complete mechanization of foundation work.

SOMMAIRE

L'auteur classe le pergélisol, qui est très répandu en U.R.S.S., (occupant environ la moitié de la surface du pays sur une épaisseur pouvant atteindre 1,5 km) parmi les sols à structure instable. Les hommes de science de l'U.R.S.S. ont été les premiers à étudier le pergélisol non seulement au point de vue géologique, mais aussi en tant que base sur laquelle reposent les édifices, et à mettre au point, de concert avec les ingénieurs, des méthodes de construction d'édifices stables sur le pergélisol. Cette élaboration des méthodes s'est basée sur les résultats de l'étude des facteurs de l'environnement et sur l'emploi des connaissances acquises en thermophysique et en mécanique des sols gelés, qui sont des domaines scientifiques explorés avec succès par les chercheurs en U.R.S.S.

L'expérimentation et les observations entreprises dans les conditions naturelles ont frayé le chemin à l'élaboration de méthodes rigoureuses de résolution des problèmes concernant les fondations, et un certain nombre de solutions sont déjà apparues et sont mises en pratique avec succès. La construction est étudiée soit de façon à préserver l'état de gel du sol servant de base, soit en prévoyant l'utilisation des méthodes de dégel préalable des mêmes sols. Dans le premier cas, l'accent est porté sur les propriétés rhéologiques des sols gelés, et dans le second, sur la consolidation primaire et secondaire de sols dégelés et sur leur manque d'uniformité selon la profondeur.

Dans le cas où une assise rocheuse se trouve à faible profondeur, et aussi quand l'épaisseur de sol gelé est faible, on utilise des méthodes de dégel et de consolidation du sol avant la construction, ainsi que des méthodes d'installation d'empattement sur pieux profonds.

Les prochaines tâches à entreprendre touchent l'amélioration des méthodes de conception des fondements et des embases et la complète industrialisation des travaux de fondations.

FROZEN AND PERMANENTLY FROZEN SOILS are widespread in the U.S.S.R., as well as in Canada and the U.S.A. and in parts of Norway, Denmark, and some other mountainous countries. In the U.S.S.R., where permanently frozen soils (permafrost) occupy about 49 per cent of the entire area of the country, they have a thickness varying from a few metres to 1.5 km (in the Viliuy River Valley) and cover the northern regions of the European part of the U.S.S.R. and West Siberia, nearly the whole of East Siberia and the Trans-Baikal region, and most of the Far East. Fig. 1 is a map of occurrence of permafrost in the U.S.S.R., line 1 on the map corresponding to the southern boundary of permafrost, line 2 to the boundary of one- or two-year-old frozen soils (*pereletki*), line 3 to the minimum soil temperatures at the zero amplitude depth, and line 4 to the maximum thickness of permafrost in metres.

Frozen and permanently frozen soils have a peculiar cryogenic texture formed as a result of the drop of the temperature of the water in these soils to below the freezing point, when part of the pore water is transformed into ice with consequent cementing (cohesion) of the mineral particles with ice. Special investigations show that in this process a certain amount of unfrozen water invariably remains in any frozen soil (Tsytovich, 1945). The factors which influence the formation of cryogenic texture are the composition and natural compaction of the soils, the

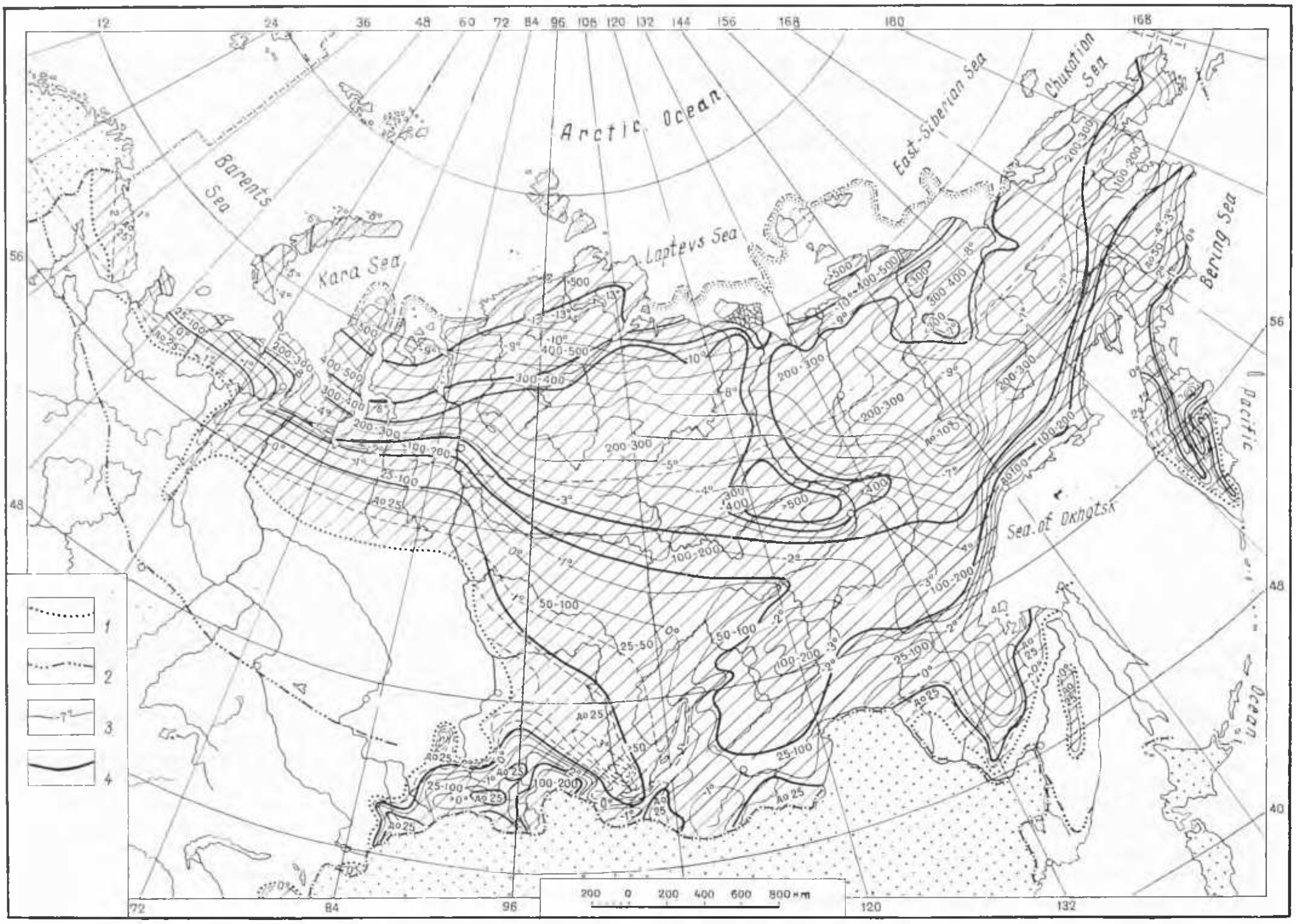

FIG. 1. Map of occurrence of permafrost in the U.S.S.R.

direction and rate of frost penetration, the migration of water (discussed below), and so forth.

Here it need only be pointed out that the cryogenic formation of the frozen soil texture explains the looseness (due to the expansion of water upon freezing, and excessive, extra-porous ice formation), the supersaturation of individual horizons with ice, and the temperature instability of frozen soils. Minute changes in the temperature of frozen soils, even in the sub-zero range (Centigrade) substantially affect the properties of frozen soils; a temperature rise above zero (to the point of thawing of void ice) brings about a drastic, avalanche-like breakdown of the texture leading to the subsidence of thawing soils.

Lack of consideration for the properties of frozen soils, for their sensitivity to temperature rise and subsidence upon thawing, results in impossible deformations of structures erected on permafrost and creates considerable difficulties in the development of permafrost regions which are often very rich in minerals and other natural resources. The need to solve these problems has necessitated much research into the natural and historical conditions of the formation of the properties of frozen soils, as well as into the changes which occur as effects of external factors (action of structures, for example).

Permafrost was first reported by the Russian armed forces in the seventeenth century. In the middle of the nineteenth century (more precisely from 1844 to 1846) the Russian academician, A. F. Middendorf, measured the temperature of permafrost in Yakutsk to a depth of 116 m in a dry Shergin well, and first described permafrost rocks. The construction of the Trans-Siberian trunk line greatly fostered the investigation of permafrost. Planned investigation of permanently frozen soils began in the late 1920's in connection with the development of mineral resources in Siberia and the Far East. Here mention should be made of the fundamental work by Dr. I. M. Soumgin (1927) and the comprehensive work by Tsytovich and Soumgin (1937). Later, research on permafrost was carried out in a number of scientific organizations and co-ordinated by the V. A. Obruchevs Permafrost Institute. On the basis of investigations into geophysics, thermal physics, and the mechanics of frozen soils (Soumgin, *et al.*, 1940; *Geocryology*, 1959), the groundwork of general and engineering geocryology was laid between the 1930's and the 1950's.

Based on the research carried out, we know that the properties of permafrost are substantially affected both by the natural-historical conditions of their formation and the conditions of present-day existence. Frozen masses of soil resulting from heat exchange between the lithosphere and the atmosphere, depending on the geophysical and geographical environment, as well as on palaeographic and neotectonic conditions, may vary widely in respect of their age, thickness, temperature, and ice saturation. Thus, for instance, the measurement by Moscow University of the

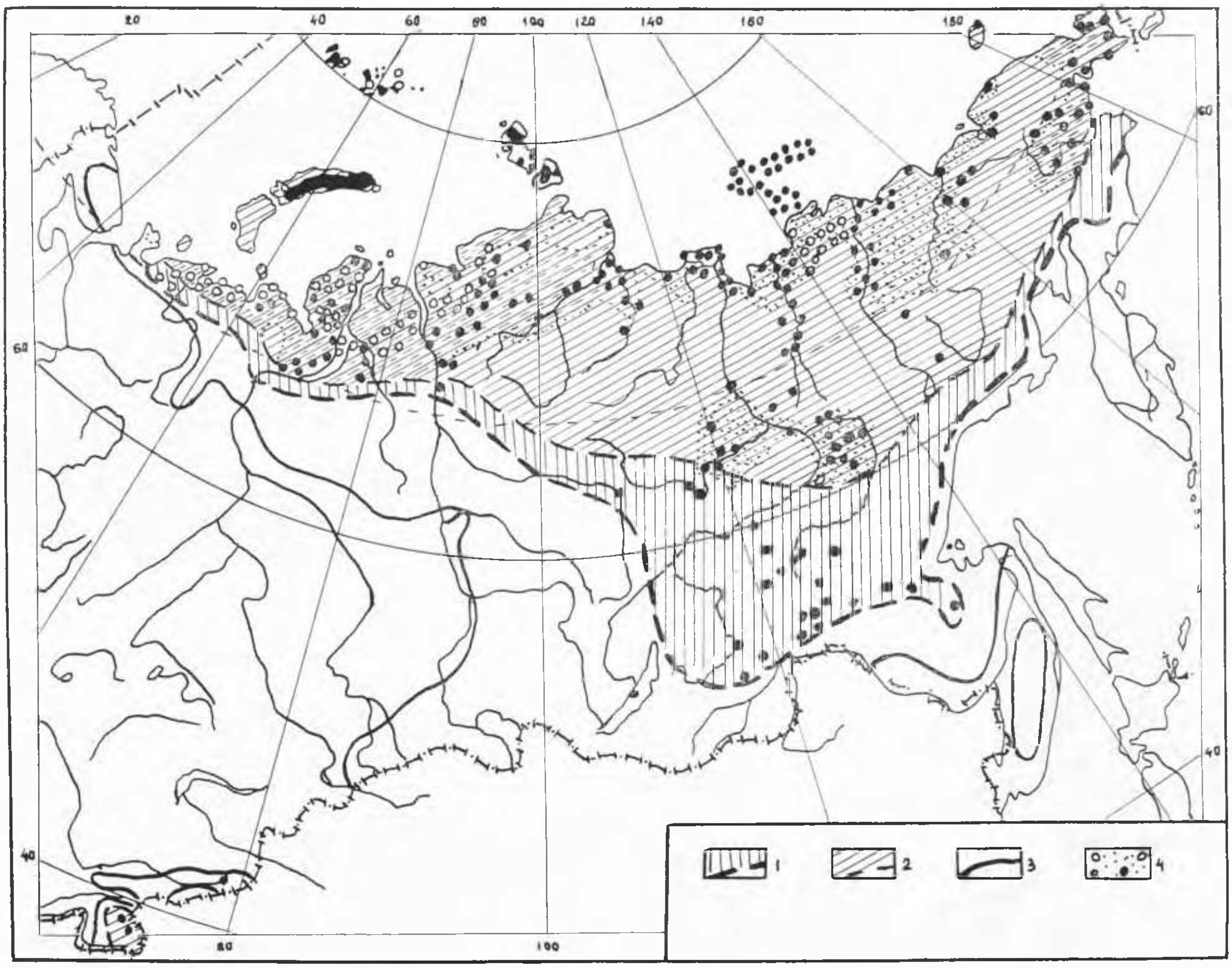

FIG. 2. Map of occurrence of recurrent vein ice.

absolute age of the frozen masses of West Siberia indicated their age to be 280 thousand years (*Conference on Geocryology*, 1963), whereas in other areas it is 10 to 12 thousand years, and under suitable conditions frozen masses may form even now. The formation of permafrost (its temperature, ice saturation, and so forth) is affected both by long-term changes in the heat exchange between the atmosphere and the lithosphere (of the order of several tens of thousands of years or even several hundred thousand years) and by short-term changes (within five or six years according to investigations conducted in Bolshezemelskaya Tundra) as described by Shvetsov (*Conference on Geocryology*, 1963). This explains the instability of the properties of frozen grounds (Tsytovich, 1963). These changes are such that in many cases they cannot be neglected even when evaluating frozen soils as structure bases.

When new permafrost areas are developed, the most important factor is the presence in the permafrost of fossil ice, its thickness, mode of occurrence, and other features.

According to Shumsky's classification (*Conference on Geocryology*, 1963), ice in frozen soils forms in three principal ways: (*a*) when moist soils freeze through (constitution ice); (*b*) when voids are filled with ice (recurrent vein ice), and (*c*) when snow and ice are buried (buried ice). The investigations of the last 50 years (Shumsky, 1955; Popov, 1953; Dostovalov, 1960) have shown that the vast majority of underground ice is recurrent vein ice formed during the recurrent winter cracking of the upper layers of the soil. In some regions of the far northern U.S.S.R. this type of ice sometimes constitutes 50 per cent of the volume of the entire 20-m top layer of the soil and considerably affects the local relief and the properties of the soil (*Conference on Geocryology*, 1963). Fig. 2 is a map of the occurrence of recurrent vein ice in the U.S.S.R. (after Shumsky and Vtyurin (*Geocryology*, 1959)). The vertical shadowing (1) shows the area of fossil recurrent vein ice; inclined shadowing (2) the area of contemporary and fossil recurrent vein ice; the solid line (3) the permafrost boundary; separate dots (4) are sites of direct detection of this type of ice.

Recurrent vein ice is widespread in the most severe regions of the permafrost area and is the cause of thermokarst which is quite common in the Far North (Kachurin, 1961; Tomirdiaro, 1965). As a rule, thermokarst originates at polygonal intersections of vein ice devoid of moss cover. Plunge-basin thermokarst lakes are therefore often characterized by a piecewise-rectangular shape due to the polygonal pattern of the vein ice (Kachurin, 1961). Fig. 3 represents a plan of thermokarst lakes in the Anadyr Tundra drawn from Professor Kachurin's data. The configuration of the thermokarst lakes conclusively proves their origin as the result of the thawing of recurrent vein polygonal ice.

Engineering investigations carried out at the North-East Integrated Research Institute of the Academy of Sciences (Tomirdiaro, 1965) indicate that thermokarst lakes originating on recurrent vein ice rapidly extend in plan and, under conditions of plain ground shift, remain shallow, since ice veins quickly disappear with depth. The constant washout of weltering masses of soil and the thawing of vein ice result in a rapid retreat of the higher shore, while the forming trail creates a new subsiding mass which, after flood decline, is again broken up into polygons by the recurrent vein ice and

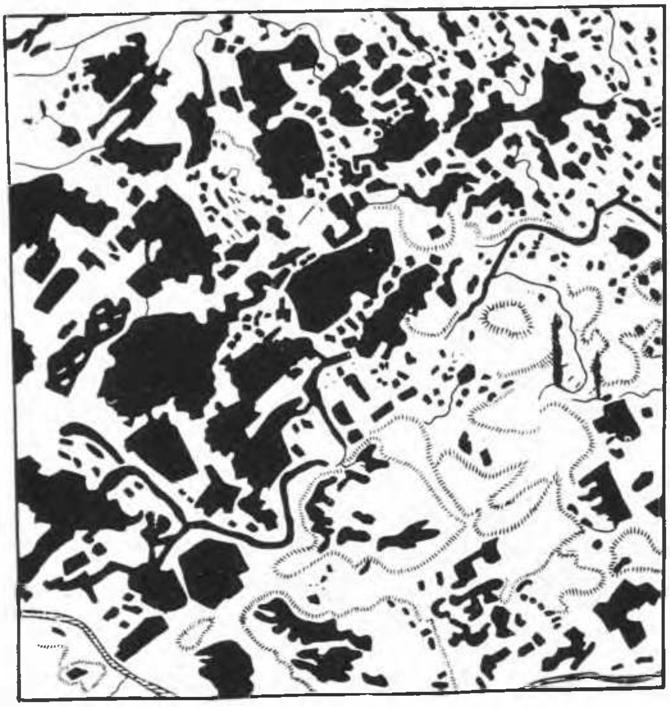

FIG. 3. Outlines of thermokarst lakes in Anadyr Tundra.

the process of thermokarst formation recurs. Fig. 4, borrowed from the paper by Tomirdiaro (1965), is a photograph of peat outcrops in the trail of a lake shifting over the tundra. Tomirdiaro has shown that the necessary condition for the formation of thermokarst in any severe Arctic environment is the simultaneous occurrence of the following two requirements: (1) the ground ice should be exposed to the sun, and (2) the thawing water should remain in the depression in the ice.

The results of borings in the beds of thermokarst lakes indicate considerable self-compaction and consolidation of previously ice-saturated soils. In tundra areas massive cryogenic reprocessing of the soil occurs (Tomirdiaro, 1965) as lakes shift over the tundra, the origin of which is associated with the thawing out of recurrent vein ice. The surface of plains in a permafrost area is being continuously reprocessed, and in many cases the area changes beyond recognition within a few years, and dispersed soils undergo drastic changes. Thus, the formation of frozen and permanently frozen soils is affected not only by long-term and short-term changes in the external thermal conditions, but also by thermokarst phenomena, by the direction and rate of frost penetration and the accompanying frost heave, and by the formation of ice interlayers and lenses, ice bodies, etc.

Of the other types of underground ice, the important ones are: (1) ice cement which forms a massive structure of

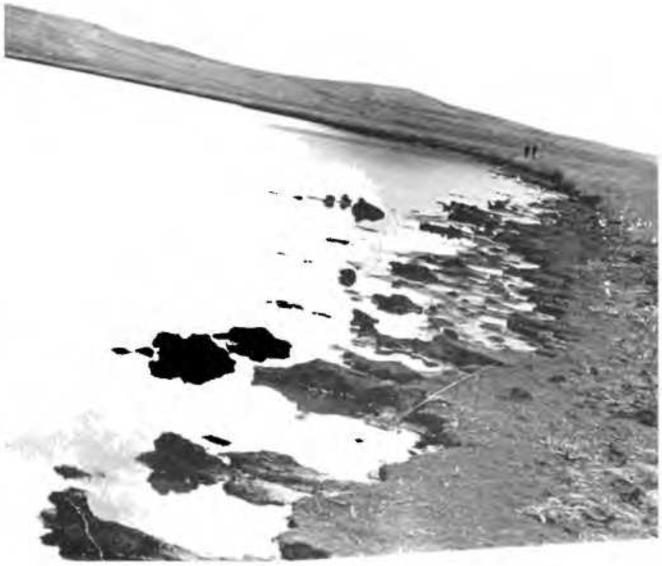

FIG. 4. Soil outcrops in trail of shifting tundra lake.

frozen soils and originates as a result of rapid freezing of soils; and (2) interlayer ice (segregational) which originates under conditions of relatively slow soil freezing with water redistribution in the soil and especially with inflow of water from outside and forms the layered and reticulate texture of frozen soils. The degree of cementing of the mineral particles by ice, and the properties of the ice cement, are the principal factors determining the strength of frozen soils of massive texture, whereas the forms and thickness of the interlayers of segregational ice determine both the general deformability of frozen soils and their anisotropy.

Turning now to an over-all evaluation of permafrost as a foundation for structures, one may, in very general terms, suggest the following classification of these soils, taking into account their diversified composition and state and the associated different behaviour of soils as structure bases.

1. *Material composition*
   (*a*) colloidal-clayey
   (*b*) gravelly-sandy
   (*c*) peaty-marshy (organic and organo-mineral masses)
2. *Temperature*
   (*a*) high (from 0 C to −1 C for clayey and peaty soils and from 0 C to −0.3 C for sandy soils)
   (*b*) low (below −1 C, −2 C)
3. *Ice content*
   (*a*) high (with a considerable amount of excessive, super void ice; flowing under thawing conditions)
   (*b*) low (containing exclusively ice cement or a negligible amount of excessive ice, plastic or semi-hard under thawing conditions)

Colloidal-clayey frozen soils contain a considerable amount of unfrozen water which is prevented from freezing by the adsorption forces of mineral soil particles (high-temperature—up to 60 or more per cent of the total amount of water), and are characterized by high compressibility and creep in a frozen state.

Gravelly-sandy soils and frozen sand contain small amounts of unfrozen water, and this, coupled with a rigid skeleton, explains their negligible compressibility and high carrying capacity in a frozen state. They always possess a loose structure, however, and when thawing, although far from converting into flowing masses, cause settlements which exceed many times the settlements in this type of soil which has not been subjected to freezing.

Peaty-marshy soils possess the properties of colloidal soils and have a strongly deformable (soft) skeleton which rapidly decomposes and mineralizes when exposed to air.

The subdivision of permafrost into high-temperature and low-temperature soils is necessary because construction on high-temperature soils is much more complicated than on low-temperature ones, since it is usually difficult to maintain the former in a frozen (and, consequently, strong) state owing to the high content of non-frozen water.*

Of great importance is the subdivision of these permanently frozen soils into high-ice and low-ice soils. As a rule, high-ice soils in a frozen state undergo considerable deformation under load; upon thawing they behave as subsiding soils, and when supersaturated they turn into liquefied masses. It has been established (Tsytovich, *et al.*, 1965) that the consolidation of thawing high-ice soils obeys the filtration theory of consolidation, but has its own peculiarities. For instance, during the whole period of thawing in the case of a unidimensional problem pore water pressure remains constant. The properties of low-ice soils differ widely from those of high-ice soils. These soils in a frozen state have considerable continuous strength, which exceeds several times that of non-frozen soils of the same composition. Upon thawing their deformation is determined mainly by the skeleton creep which is well described by the theory of non-linear hereditary creep of solid masses.

We turn now to consider the mechanical processes in freezing, frozen, and thawing soils, which arise from external factors such as temperature and pressure. Investigation of these processes made it possible in the U.S.S.R. to work out measures for control of associated unwanted phenomena and to lay the foundation for developing rigorous methods of solving problems of the theory of structure bases. It should be noted that in studies of the mechanical processes in frozen soils it appeared expedient to regard frozen soils not as solid homogeneous masses, but as complex dispersed multicomponent systems (solid mineral particles, plastic ice inclusions, different kinds of water as regards the degree of binding, gases) possessing their own specific features, undergoing different deformations under the action of external forces, and exerting a mutual influence upon each other.

The principal process in freezing soils under natural conditions (at non–super-low temperatures) is the redistribution of water in them and water migration with subsequent formation of ice at the freezing front. As far back as 1931 Jung's experiments showed that only at very low temperatures (of the order of −70 C and lower) did no water redistribution or formation of layers of ice occur (Jung, 1931). On the contrary, water fixation was observed. However, though experiments by Bouycous (1917) and Beskow (1930), the author (Tsytovich and Soumgin, 1937), Goldstein (1940), and many others have long since established the fact of water migration to the freezing front, no acceptable theory of quantitative determination of the migration flow has yet been developed owing to the extreme complexity of the migration process.

In recent years, at the Institute of Bases (Moscow) laboratory of Thermal Physics headed by B. V. Porkhayev, G. M. Feldman, in studying the process of water migration in dispersed soils, made an attempt to use the general equation of water transfer in a capillary porous medium, which was developed by A. V. Lykov (1954).

Since water migration in soils occurs as a result of the action of adsorption forces of the mineral skeleton of the soil (Bouycous, 1917; Beskow, 1930; Tsytovich, 1941) and since, according to the principle of equilibrium state of water in frozen soils formulated by the author (Tsytovich, 1945), only a definite amount of water, corresponding to a given temperature and pressure, turns into ice upon freezing, a water potential gradient originates at the freezing boundary, and this gradient is the driving force of migration. Lykov's differential equation of water transfer in a capillary porous medium takes the following form:

$$(1 - \zeta)\ (\partial w/\partial T) = \text{grad}\ [K\ (\text{grad}\ w) + K\delta\ (\text{grad}\ \Theta)], \quad (1)$$

where ζ = phase transition coefficient, T = time (hr), w = moisture content, K = coefficient of potential conductivity (sq.m./hr), δ = thermogradient coefficient (kg/m deg.), Θ = temperature.

Detailed investigations of water migration in freezing soils carried out at the Thermal Physics Laboratory of the Research Institute of Bases (Feldman, 1963; Porkhayev, 1964) have shown that both the component of the migration flow due to the temperature gradient and the changes in the water content above the plasticity limit of soils can be neglected in practice.

In addition, Lykov's equation was solved by Feldman (Porkhayev, 1964) with the aid of Lukianov's hydrointegrator (*Conference on Geocryology*, 1963), and this made it possible to determine numerically (within the limits encompassing the feasible cases of changes in the mass exchange characteristics and rate of freezing) the approximate value of the migration water flow (i_0) in the absence of groundwater and the average value of the migration flow in the zone of influence of groundwater (i_w).

If the migration flow is known, the amount of heave (increase in the height of the soil layer upon freezing) will be determined by a simple expression, because this value is proportional to the magnitude of the flow (i_0) and the migration time (T). We are given:

$$h_{\text{heav}} = 1.09\ i_0\ T \quad (2)$$

for the case of an open system, and

$$h_{\text{heav}} = 1.09\ i_0\ T_{\text{act}} \quad (3)$$

for the case of a closed system, where T_{act} = active heaving time equal to h_{act}/V, h_{act} is the thickness of the layer actively heaving (which, according to Prof. Dalmatov (1957), is equal to ⅔ of the depth of maximum freezing), calculated according to Feldman as a function of depth (h_{fr}) and rate (v) of freezing and the magnitude of the migration flow (i_0) (Porkhayev, 1964). The expression for the value h_{heav} in the zone of influence of groundwater has also been obtained with the aid of the hydrointegrator, but has a more complicated form:

$$h_{\text{heav}} = 1.09\ (1/V)\,[i_0(h_{\text{fr}} - Z_w) + i_w Z_w] \quad (4)$$

where Z_w is the difference between the height of the zone of influence of groundwater and the distance between the freezing line and the groundwater table. The values of migration flows (i_0 and i_w) have complicated expressions with numerical coefficients obtained with the aid of the hydrointegrator (Porkhayev, 1964).

*As described by Nersesova and Tsytovich in the paper, "Unfrozen Water in Frozen Soils" (*Conference on Geocryology*, 1963).

Equations 1–4, as well as the expressions for i_0 and i_w, are shown in Fig. 5.

$$h_w = 1{,}09\,\frac{1}{V}\left[i_0 (H - Z_w) + i_w Z_w\right]$$

by water level action

$$i_w = f(A, H_{cz}, Z_w, W_p, V);$$

$$A = (W_0 - W_\zeta)(0{,}34 + 670 \cdot K)\,\gamma_d;$$

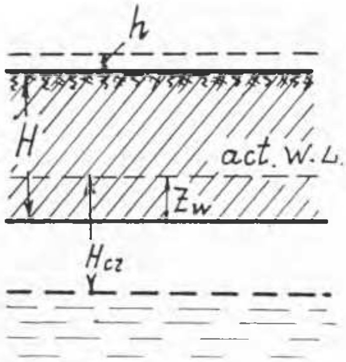

FIG. 5. Formulae for computing water migration flow during freezing of soils.

It is worth noting that by using Lykov's theory of water transfer in porous bodies it is possible at present, by means of computers, to predict the maximum frost heave of soils with sufficient practical accuracy. A comparison of the calculated values of heaving with the results of measurements under different natural conditions (on seven field sites) shows a deviation of about 14 per cent from the measured values.

Field experiments conducted in the U.S.S.R., Alaska and Canada also indicate that the amount of frost heave decreases upon application of an external load to the soil surface. This, we believe, can also be taken into account by means of the theory of thermomoisture conductivity, if we recall that, all other things being equal, the migration flow is proportional to the difference between the initial water content of the soil and the water content corresponding to the content of non-frozen water at the boundary of the freezing zone, and that the content of non-frozen water, according to the principle of the equilibrium state of water in frozen soils, increases with external load (Tsytovich, 1945).

In our opinion, predicted values of the frost heave of soils can be used to advantage in working out engineering heave prevention measures for road bases and pavements.

It should be noted that water migration in freezing dispersed soils (particularly clayey and peaty types) is the principal factor forming their texture, which in turn substantially determines the strength properties of frozen soils. In the case of one-sided freezing, a layered texture of frozen soils is formed, with ice interlayers whose magnitude depends both on the intensity of water migration to the freezing front and on the rate of freezing; if the freezing is not one-sided and non-uniform, a reticulate texture of frozen soils is formed. In the course of migration mineral particles of the soil are also displaced.

Mechanical processes in frozen soils arise under the action of both external load and internal forces. The internal forces are the temperature effect and the effect of heaving forces originating in an increase in the volume of freezing water drawn (in the course of migration) to the freezing front. If the upper layers of the soil are non-uniform in composition and water content, temperature effects cause cracks therein which lead to the formation of recurrent vein, wedge-like ice. Heaving forces cause non-uniform heaving of the soil (heaving bulges or "pingos," ice bodies, etc.), and, with the participation of the forces causing the freezing together of soils with the foundation material, the heaving of the latter (in the absence of heave-prevention measures).

The external forces are the total and local loads. A total load results in the compaction of frozen soils, and experiments show that only vertical settlements take place in this case. A local load (from structure foundations) of a certain value will also cause only compaction settlements (even in permafrost, this being demonstrated by direct observations), but if the load is increased, then, at its limit value, the stability of the base will be impaired and impermissible shifts and bulges will occur. It is the engineer's task to determine the limit (and from it the safe) load at structure bases; methods for this have been successfully worked out in the U.S.S.R. (Tsytovich and Soumgin, 1937; Tsytovich, 1941; Vyalov, 1959 and 1963; *Technical Specifications*, 1960).

Relevant investigations (Tsytovich and Soumgin, 1937; Vyalov 1959 and 1963) have shown that the most important factors determining the mechanical properties of frozen soils are the *rheological processes* arising in them which govern, for instance, the influence of the rate of growth of the load and the time of its action upon the resistance of frozen soils to external forces. Under a short-duration load the resistance of frozen soils is great and, as under long-duration loads, depends on the composition, ice content (content of non-frozen water, especially in the phase of intensive transitions), and value of subzero temperature (Fig. 6b). Under long-duration action of the load a relaxation (weakening) of the stresses takes place, and the soil resistance falls off sharply, reaching some constant limit referred to as the limit of continuous strength. If the load acting on the soil is below this limit, the deformations of frozen soils with time will appear as a typical curve of damped creep (Fig. 6d, curve 1) determined by the compression of the frozen soil (Fig. 6c) and the creep of its skeleton. If the load exceeds the limit of continuous strength, undamped creep is observed, and the rate of creep may be taken as proportional to the excess (over and above the limit of continuous strength) pressure. Shown in Fig. 6 are: (*a*) curves of non-frozen water for clay (1) and for sand (2); (*b*) instantaneous (σ_{inst}) and continuous (σ_c) resistance for clay (1) and for sand (2); (*c*) compression curve for high-temperature clay; (*d*) deformation curves of frozen clay: 1, of extinguishing creep; 2, of non-extinguishing creep (plastic flow).

When selecting design resistances for permafrost at structure bases it is necessary to proceed from their continuous resistances, which are 5–15 times smaller than their instantaneous ones, but still several times greater than the resistances of non-frozen soils. Here, as well as in the erection and use of structures, one must not forget the instability of the mechanical properties of frozen soils (Tsytovich, 1963) owing to the increase in their temperature, even in the region of subzero temperature, the non-uniformity of deformation

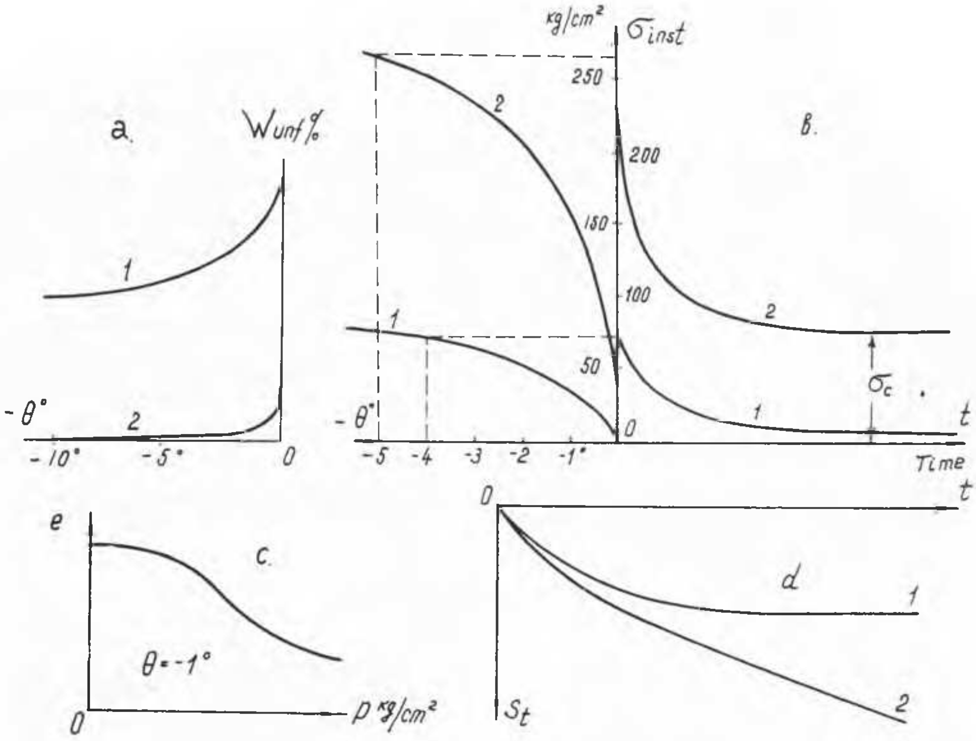

FIG. 6. Physico-mechanical properties of frozen soils.

in depth caused by the compaction of the upper layers of the soil, and so forth.

At present, definite values of standard resistances for permafrost at structure bases have been adopted in the U.S.S.R., and their design subzero temperature must absolutely be maintained (*Technical Specifications*, 1960).

In a general way, standard resistances can be characterised by the following values:

1. For sand and gravelly-sandy soils: (*a*) high temperature ($\Theta = -0.1$ to -0.3 C), resistance, $R^{II} = 4$–6 kg/sq.cm.; (*b*) low temperature ($\Theta = -2$ to -3 C); $R^{II} = 7$–12 kg/sq.cm.
2. For colloidal-clayey soils: (*a*) high temperature ($\Theta = -0.5$ to -1 C), $R^{II} = 2$–4 kg/sq.cm.; (*b*) low temperature ($\Theta = -2$ to -4 C), $R^{II} = 4$– 8 kg/sq.cm.
3. For peaty-marshy soils ($\Theta = -2$ to -4 C), $R^{II} = 1$–2 kg/sq.cm.

The latest research results indicate that the above values of design resistances can be increased by a factor of approximately 1.2–1.8.

Until recently builders usually thought that when a structure was erected on permafrost retaining its subzero temperature the deformation of the base would be negligible and could be ignored in the design. However, our investigations (Tsytovich, 1959) and those of our co-workers (Brodskaya, 1962) have shown that high-temperature soils, particularly clayey ones, are highly compressible (with the compressibility coefficient m_v being equal to 0.02–0.07 sq.cm./kg, and sometimes even more) due to the recrystallization and repacking of pore and excess ice under the effect of external pressure and the migration of non-frozen water under pressure with some squeezing out. The latter necessitates designing foundations that are erected on permanently frozen, high-temperature clayey soils in accordance with the limiting settlements of bases (Tsytovich, 1960; Ushkalov, 1962) in order to guarantee against impermissible deformations of the structures.

When frozen and permanently frozen soils thaw, the crystallization bonds of the ice break down and the soil texture is drastically impaired. Partial thawing of permafrost (which is taken to mean an increase in the water content in the liquid phase) takes place upon any increase in the subzero temperature, and, as has been shown before (Tsytovich, 1945), is always accompanied by a decrease in soil strength since part of the cementation bonds break down. On the other hand, at the temperature of soil thawing, an abrupt (avalanche-like) breakdown of structural bonds occurs, compaction settlements are observed, and for high-ice soils under a definite load, which is sometimes quite small, a local, very rapid settlement attended by the squeezing out of liquefied thawed soil masses, or subsidence, takes place. When structures are erected on permanently frozen soils, their subsidences upon thawing are especially dangerous, because it is impossible to build structures on such soils without taking special measures for compacting the soils. Settlements of non-subsiding thawing soils (observed mainly in gravel, sandy, and semi-hard clayey soils) should be taken into consideration in design.

The problem of determining the settlement of a thawing soil has been under investigation for quite some time (Tsytovich, 1937; Ushkalov, 1962), but in recent years new investigations into the consolidation of thawing soils make it possible to predict the rate of settlement of thawing high-ice soils (Tsytovich, *et al.*, 1965), and this is essential for designing structures to be built on thawing bases.

The typical compression curve of thawing soils is shown in Fig. 7a. At least three characteristic portions can be discerned in this curve: portion 1′–1, compaction of the soil

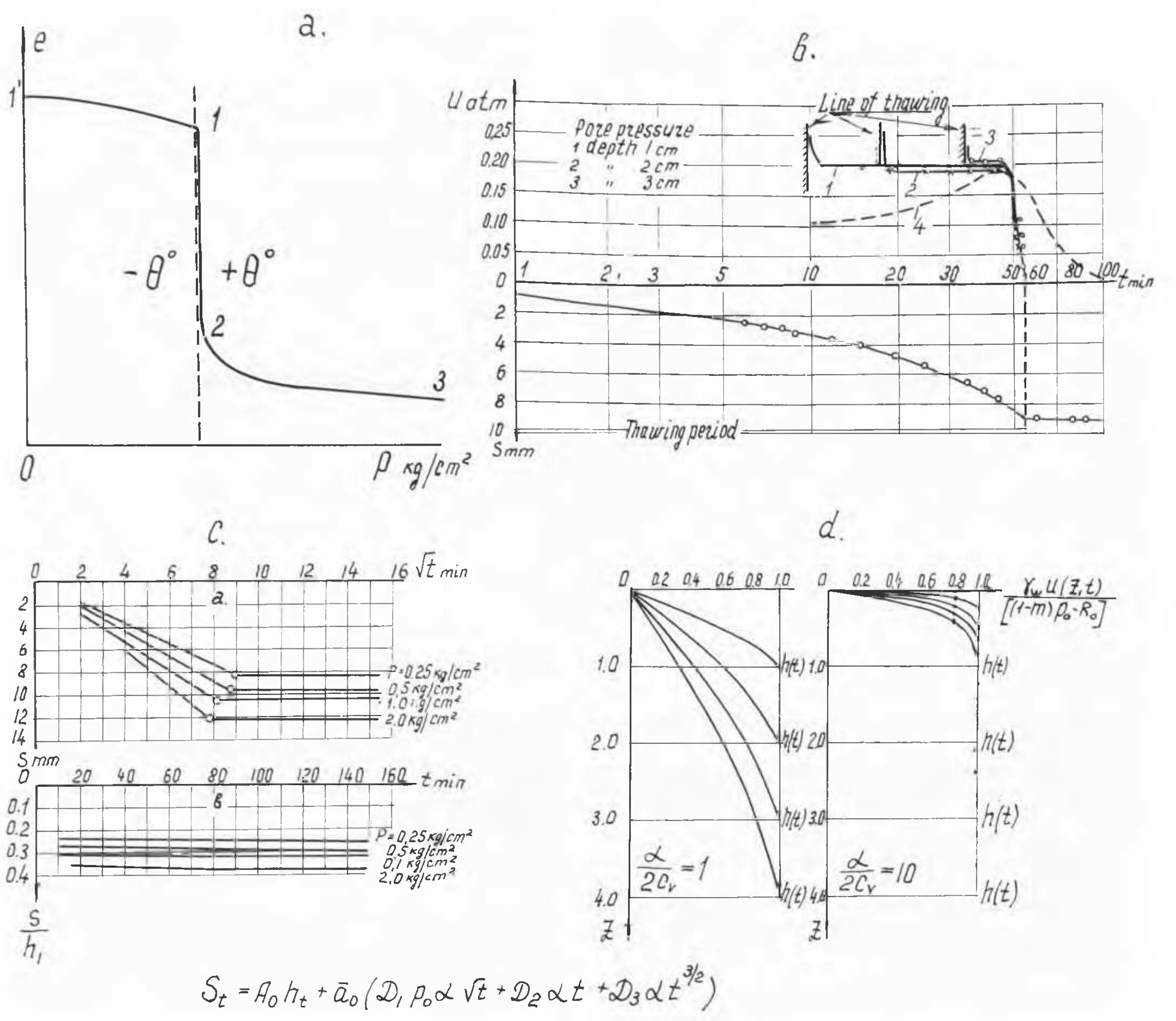

FIG. 7. Settlements of high-ice frozen soils during thawing.

(change in its void coefficient, e) in a frozen state, conditioned by migration-viscous deformation; portion 1–2 corresponding to abrupt settlement in the course of thawing (which, for high-ice soils, is well defined by the theory of filtration consolidation, allowing for the specific features of the thawing soils); and, finally, portion 2–3, the first half (immediately after thawing) being due to both residual filtration consolidation and skeleton creep, the second half, only to the creep of the soil skeleton.

Experimental research has demonstrated (Tsytovich, *et al.*, 1965) that consolidation of high-ice soils during thawing (with simultaneous loading) proceeds at a practically constant pore pressure (Fig. 7b). The initial value of the pore pressure is below the external pressure ($U = \beta_0 p_0$, where $\beta_0 < 1$), however, and the settlement until the moment of complete thawing is proportional to the square root of time (Fig. 7c). In this case the relative settlement (ratio of the settlement S_t to the depth of thawing h_t, i.e. S_t/h) is close to a constant. After thawing, the process slows down, and for some time the settlement is proportional to the logarithm of the time.

It is well known from the author's works (*Geocryology*, 1959; Tsytovich, 1941 and 1960) that the total settlement of thawing soils at uniform loading (one-dimensional problem) consists of two phases: thermal settlement, which is independent of the value of the external pressure and proportional to the depth of thawing (h), and compaction settlement upon thawing ($a_0 h_t p$, where $a_0 = m_{vs}$, the compaction coefficient, that is,

$$S_t = A_0 h_t + a_0 h_t p. \tag{5}$$

Settlement for any time interval has been obtained by rigorous solution of the differential equation of compaction of high-ice thawing soil (Tsytovich *et al.*, 1965):

$$c_v(\partial^2 \bar{\sigma} / \partial z^2) = \partial \bar{\sigma} / \partial t \tag{6}$$

TABLE I. VALUES OF D_1 AND D_3

$\xi = \frac{\alpha}{2\sqrt{c_v}}$	D_1				D_3			
	$\beta = 0$	$\beta = 0.2$	$\beta = 0.3$	$\beta = 0.5$	$\beta = 0$	$\beta = 0.2$	$\beta = 0.3$	$\beta = 0.5$
0	0.5	0.6	0.65	0.75	0.25	0.3	0.325	0.375
0.5	0.48	0.584	0.636	0.74	0.222	0.271	0.295	0.343
1	0.423	0.539	0.593	0.712	0.168	0.213	0.285	0.279
3	0.188	0.351	0.432	0.594	0.063	0.099	0.117	0.153
5	0.113	0.290	0.379	0.556	0.038	0.072	0.089	0.124
10	0.056	0.245	0.339	0.528	0.019	0.052	0.069	0.103

Under conditions of linear increase in loading $p_t = (p_0 + nt)$, at an initial void pressure of a fraction of the external load ($U_0 < \beta_0 p_t$), and when the thawing boundary moves proportionally to the square root of the time ($h_t = \alpha\sqrt{t}$) Tsytovich, *et al.*, 1965),

$$S_t = A_0 h_t + \bar{a}_0(D_1 p_0 \alpha\sqrt{t} + D_2 \alpha t + D_3 \alpha t^{3/2}) \quad (7)$$

where $D_2 = \frac{1}{2}\gamma^1\beta_0$, γ^1 being bulk weight of the soil in water; and $\bar{a}_0 = D_1 a_0$, a_0 being the relative compaction coefficient of the soil upon complete thawing. D_1 and D_2 are given in Table I as a function of the parameter $\xi = \alpha/2\sqrt{c_v}$, where c_v is the consolidation coefficient of the thawing soil. An expression has also been obtained for the distribution of pore pressure in the thawing soil along the depth (see Tsytovich, *et al.*, 1965, Eq 7). The curves of time variation of pore pressure (U_t) are presented in Fig. 7d for the case $\xi = 1$ and $\xi = 10$.

Fig. 7 illustrates research on the consolidation on thawing soils: (*a*) total compression curve of frozen soil upon thawing; (*b*) change in void pressure and in settlements of high-ice soil upon thawing (time is plotted on a logarithmic scale); (*c*) change in settlements of thawing soils with time (the circle denotes the termination of thawing); (*d*) calculated curves of void pressure in a thawing soil for different thawing depth h_t and different values of the parameter ξ.

It is worth noting that for a three-dimensional problem on thawing settlements an approximate engineering solution has been obtained by the author's equivalent-layer method both for the total stabilized settlement of foundations and for the rate of settlement with time (Tsytovich, 1941 and 1960).

The data quoted clearly indicate the complexity of the processes of deformation in freezing, frozen, and thawing soils which the builder must take into account in building structures on permafrost. Let us consider the basic principles governing the design of bases and foundations built on permafrost, principles which have been established on the basis of experimental research and observations under natural conditions, as well as by the use of discoveries in thermal physics and the mechanics of frozen soils.

Depending on the geological and geophysical conditions at the site of construction, the properties of permanently frozen soils, and the temperature conditions of the structures and their sensitivity to non-uniform settlements, one of the following principal methods for erecting structures on permafrost may be selected: (1) on bases which, practically, do not change their strain-strength properties upon thawing; (2) by the method of conservation of the frozen state of the base; (3) by erecting structures allowing the frozen soil of the base to thaw; (4) by the method of pre-construction thawing and compaction of the base soils.

Method 1 is used only when building on non-fissured rocks. Even in this case, however, the method has its peculiarities in permafrost regions, since the effect of frost heaving forces on the foundations must be taken into consideration. Consequently, under permafrost conditions it is essential, even on rock, to use a foundation that exceeds in depth the thickness of the active layer (the layer of annually recurring freezing and thawing) or else to use the appropriate thermal insulation of the surface. Also, in erecting structures under the conditions of permafrost by any of the methods, the amount of heave must be calculated (Saltykov, 1959; Tsytovich, 1957 and 1960).

At a thickness of heaving soils up to 1 m the rated heaving forces (*Technical Specifications*, 1960) are taken as equal to 60 to 90 kg per cm. length of the foundation perimeter (depending on the severity of the climate); at a depth of freezing up to 2 m or more, the heaving forces are 100 to 150 kg/cm. It is worth recording that recent experiments carried out in the Igarka region by Orlov (1962) showed somewhat larger average values of heave (up to 170 kg/sq.cm., and under especially unfavourable conditions up to 350 kg/cm), with the greatest forces arising at the ground surface; the heaving forces diminish with depth (by a factor of 2 or 3 at a depth of 2.5 to 3 m).

Method 2, conservation of the frozen state of the base soils, is at present the most widely applied in practice and is provided with an adequately elaborated method of foundation design. This method is based on the suggestion of Russian engineers borrowed by them from experience with some old structures in the Trans-Baikal region which had basements that were ventilated during the winter. The engineering substantiation of this method (Tsytovich, 1928; Tsytovich and Soumgin, 1937) and expert advice on the building of the first capital structure with a considerable evolution of heat (the Yakut Heat and Electric Power Station) by the method of conservation of the frozen state of the base soil were supplied by the present author (Tsytovich, *et al.*, 1947). The main feature of this method is the removal of the heat produced by the floor of the structure through ventilation of the basement during the winter. The designer should allow for the thermal effect of the building on the temperature conditions in the frozen soils and for the variation in their bearing capacity and deformability. Thermal design of permafrost bases is discussed in the works of Saltykov (1959), Porkhayev (*Geocryology*, 1959; *Conference on Geocryology*, 1963; *Thermal Physics*, 1964), and Ushkalov (1962), as well as by Tomirdiaro (1963) and others.

Referring all those interested in this matter to the relevant literature, we will restrict ourselves to presenting a schematic drawing of a building designed to have basement ventilation in winter (Fig. 8).

Method 3, erecting structures allowing the thawing of

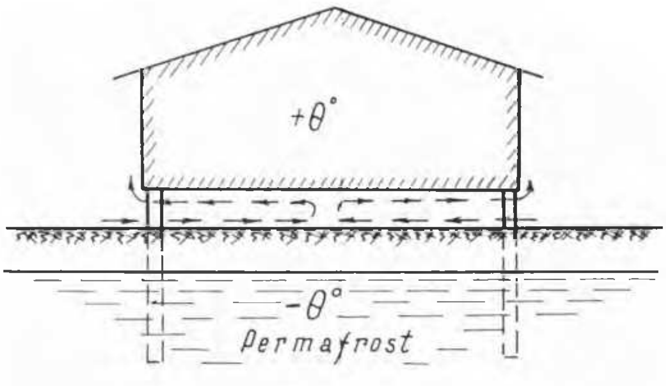

FIG. 8. Schematic drawing of building with basement ventilated in winter.

frozen bases, as indicated by the building practice, can be used only for soils showing small settlements upon thawing, for instance, sandy, hard-clayey, and shale-clayey frozen soils. According to Saltykov (1959), the conditions for the applicability of this method are:

$$A_0/a_0 \leqslant 3 \text{ and } \Delta S \leqslant \tfrac{1}{4}S_{av}, \qquad (8)$$

where A_0 = relative thawing coefficient, a_0 = relative coefficient of compaction upon thawing, ΔS difference in settlement, and S_{av} = average settlement upon thawing. The procedure for determining the thawing coefficient (A_0) and coefficient of compaction upon thawing (a_0) is set out in a number of works (Ushkalov, 1962; Tsytovich, *et al.*, 1961).

In designing foundations by the Method 3 it is essential to predict the formation of the thawing area (this problem found its solution in the work of the V. A. Obruchev Permafrost Institute, the Institute of Bases, and other research establishments (*Geocryology*, 1959; *Thermal Physics*, 1964; Saltyvov, 1959; Tamirdiaro, 1963; Velli, *et al.*, 1963; Pchelkin, 1963; Ushkalov, 1962) and the prediction of foundation settlements on thawing soils (Tsytovich, 1941; Saltykov, 1959; Ushkalov, 1962).

The foundations of structures erected with an allowance for the thawing of frozen soils should be designed in keeping with the limit settlements of the bases (Tsytovich, 1960; Ushkalov, 1962). In this case the total design (predicted)

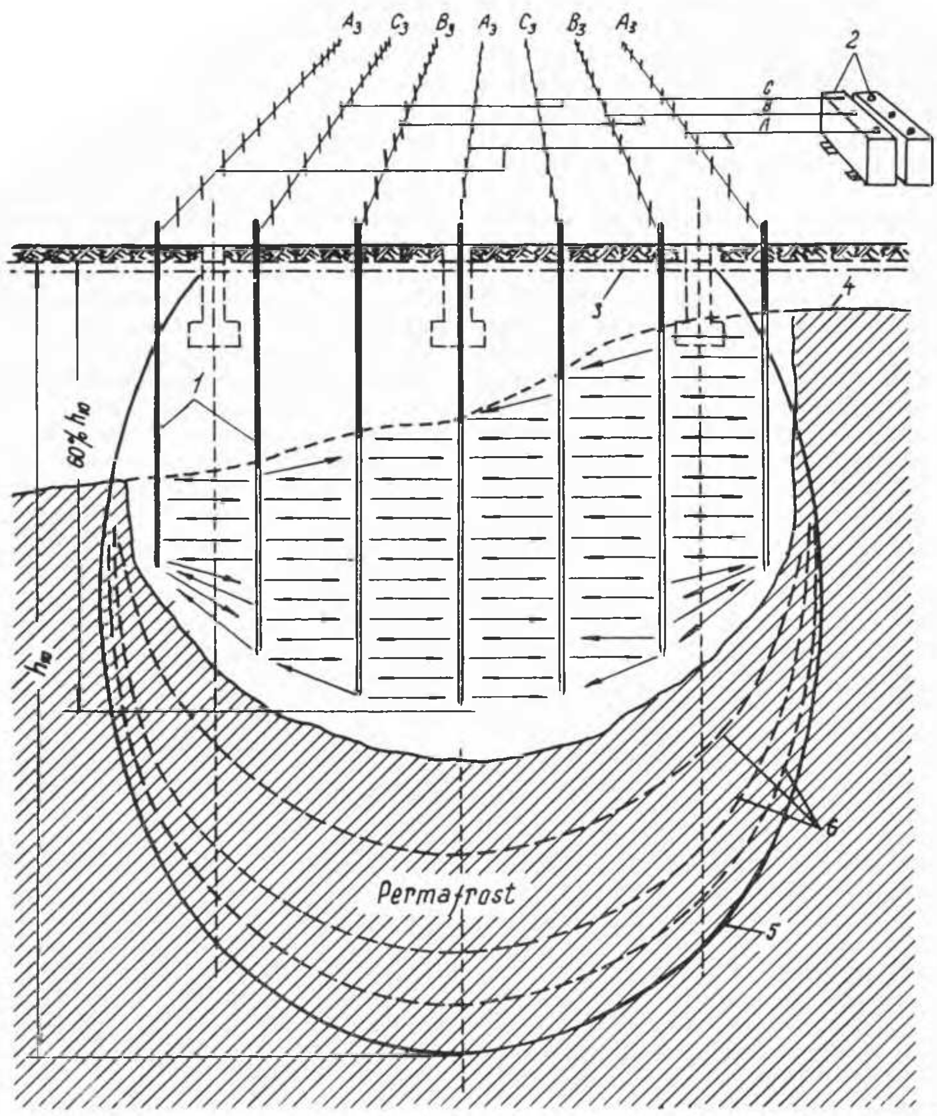

FIG. 9. Schematic drawing of pre-construction thawing.

settlement of the base and the difference in the settlement of the foundations should be below the limit values for the particular type of structures, which are based on observations of structure deformations and are standardized accordingly (*Technical Specifications*, 1960).

Method 4, pre-construction thawing and compaction of thawing soils, is used mainly when rock occurs at a shallow depth (about 5 to 15 m) or when the soil mass substantially affecting foundation settlements can be strengthened and compacted after thawing. In the latter case special investigations and heat engineering calculations are made to establish the thickness of the frozen soils which should be subjected to preliminary thawing, proceeding from the rule that the levels which are the highest in ice content and the most deformable (usually these are the upper layers of permafrost down to a depth of 5 to 10 m) should be previously thawed and compacted.

In recent years (1960–64) the electrical method of preliminary thawing and compaction of soils has been used successfully. First the permafrost is thawed with alternating current down to the depth required according to the prediction, attended by the simultaneous compaction of the thawed soil under gravity, and second, electro-osmotic dehydratation

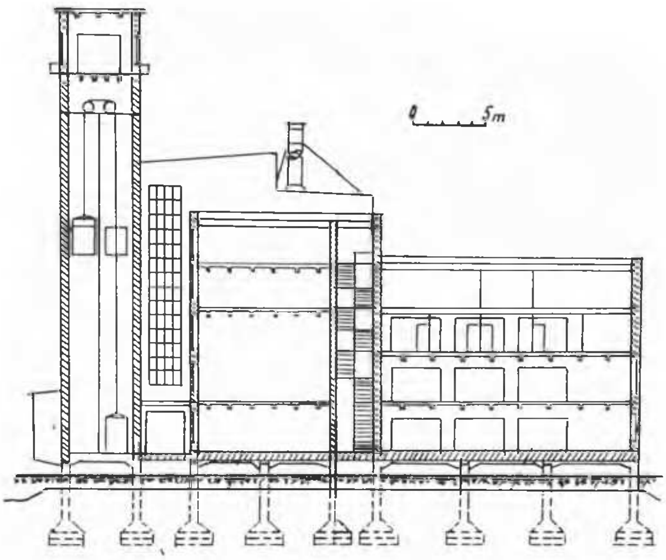

FIG. 10. First building designed by method of conservation of frozen state of base soils.

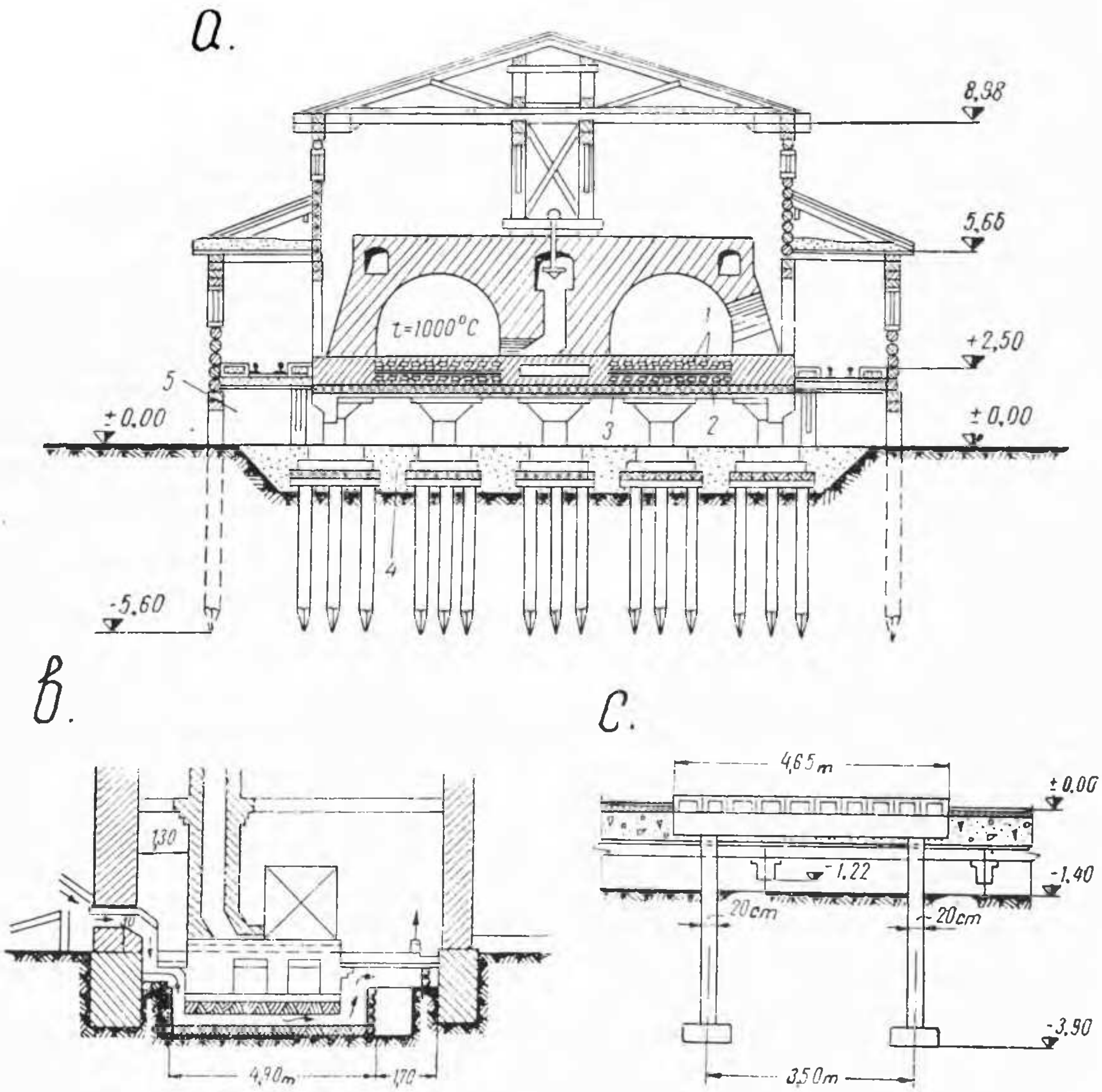

FIG. 11. Examples of structure erection by method of conservation of permafrost.

and strengthening of the thawed soil is effected under the action of direct current.* The latter operation is used to advantage in the case of clayey and other finely dispersed frozen soils.

A scheme of pre-construction preparation of permanently frozen bases by the method of electrical thawing and electro-osmotic compaction, which has been worked out by the Research Institute of Bases, is given in Fig. 9 where the symbol h_{10} denotes the depth of the prediction thawing area (for a period of 10 years). The numbers have the following meaning: 1, electrodes which simultaneously serve as vertical drains; 2, current transformers; 3, groundwater table; 4, boundary of the permafrost surface after thawing; 5, outline of a limiting (after a long period of time) thawing area under the structure; 6, development of the thawing area with time.

The present author believes that of all the methods of structure erection on permafrost the most expedient and promising (provided they are further improved) are those that permit of nearly complete mechanization of foundation work and, consequently, provide the highest economy, namely: erection of structures by the method of conservation of the frozen state of the base soil (Method 2), and erection of structures by the method of pre-construction thawing and subsequent compaction of the base soils (Method 4).

I wish to point out that with regard to erecting structures by Method 2 (conserving the frozen state of the bases), there are now available not only systems of foundations, the most common types being pile foundations and deep spread footings (*Geocryology*, 1959; Velli, *et al.*, 1963), but also services such as water and heat supply and sewerage (Pchelkin, 1963) which have made possible the successful construction of capital structures and modern towns on permafrost.

Fig. 10 represents a cross-section of the first industrial building erected by the method of conservation of the frozen state of the bases (the Yakutsk Central Power Station) which was designed according to formulas worked out by the author (Tsytovich, 1928) and built with his advisory participation (Tsytovich, *et al.*, 1947). This building has been in use since 1933 and shows no impermissible deformation (the temperature inside the building being sufficiently high). The level of the permafrost boundary under the building has not lowered, but, on the contrary, has risen by 0.8 to 1.2 m compared to the initial level (according to the survey data of the Yakutsk Research Station of the USSR Academy of Sciences).

Fig. 11 shows other examples of successful erection of structures by the method of conservation of the frozen state of the soil bases (Velli, *et al.*, 1963): (*a*) foundations for the ovens of a brick works using cavity brick lying under the bottom of the stove with the provision of a ventilated basement under the entire building; (*b*) a boiler house in Yakutsk with a base cooled by ventilation channels (after P. I. Melnikov); (*c*) a boiler foundation with full-width basement ventilated (the town of Amderma). Finally, Fig. 12 is a photo of the main street of the town of Norilsk with modern buildings erected by the method of conservation of the frozen state of the base soils. These buildings are provided with central heating, hot and cold water, and all modern conveniences. As can be seen, they are in good condition even in the severe environment of Arctic permafrost.

In conclusion the author wishes to point out that he deems it highly desirable to establish international co-operation in the field of improving methods for building various types of structures on permafrost along the following lines:

1. Exchange of the results of experimental building of structures on permafrost, establishing the laws of interaction of permafrost and the structures.
2. Improving the procedure of engineering-geocryological investigations using precision methods of modern physical experiment.
3. Exchange of experience in the mechanization of foundation and earth-moving work under the conditions of permafrost, and the automation of operations ensuring the preservation of structures erected on permafrost.

*See the paper by B. A. Rzhanitsyn, V. F. Zhukov, and others in the *Proceedings* of the International Permafrost Conference (1963).

FIG. 12. Main street in city of Norilsk, built on permafrost.

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