Principles of Geocryology (Permafrost Studies). Part II, Engineering Geocryology. Chapter VI, Bases and Foundations. p. 140-218

Saltykov, N. I.; Porkhaev, G. V.

Publisher's version / Version de l'éditeur:
https://doi.org/10.4224/20386778
Technical Translation (National Research Council of Canada), 1967

PREFACE

This translation is the tenth from the Russian permafrost publication "Principles of Geocryology", Part II (Engineering Geocryology). Chapters of Part II that have already been translated and the TT numbers in the NRC series of technical translations are (in the order of their translation) as follows:

Chapter I	Principal aspects of engineering geocryology by N.I. Saltykov (TT-1215)
Chapter VII	Particular aspects of mining in thick permafrost by V.P. Bakakin (TT-1217)
Chapter II	Deformation of structures resulting from freezing and thawing by A.I. Dement'ev (TT-1219)
Chapter VIII	Beds for roads and airfields by G.V. Porkhaev and A.V. Sadovskii (TT-1220)
Chapter IX	Underground utility lines by G.V. Porkhaev (TT-1221)
Chapter XI	Specific features of the maintenance of structures in permafrost conditions by A.I. Dement'ev (TT-1232)
Chapter III	Basic mechanics of freezing, frozen and thawing soils by N.A. Tsytovich et al. (TT-1239)
Chapter IV	Thermal physical principles of controlling the interaction between structures and frozen soil by G.V. Porkhaev (TT-1249)
Chapter V	Principal methods of moisture-thermal amelioration of the ground over large areas by V.P. Bakakin and G.V. Porkhaev (TT-1250)

This translation of Chapter VI by N.I. Saltykov and G. V. Porkhaev reviews the design, construction and maintenance of foundations in permafrost regions and engineering aspects of the perennially frozen foundation soils. The various methods of contructing foundations in permafrost are: designing for preservation of the permafrost, designing for gradual thawing of the permafrost with possible differential settlement of the foundation soils, designing for preconstruction thawing of the permafrost. Measures to counteract the effects of heaving in the seasonally thawed layer above the permafrost are also discussed.

The Division of Building Research is grateful to Mr. G. Belkov, Translations Section, National Research Council, for translating this Chapter and to Dr. R.J.E. Brown of this Division who checked the translation.

Ottawa

March, 1967

N.B. Hutcheon

Assistant Director

NATIONAL RESEARCH COUNCIL OF CANADA

Technical Translation 1266

Title: Bases and foundations
(Osnovaniya i fundamenty)

Authors: N.I. Saltykov and G.V. Porkhaev

Reference: Principles of geocryology (permafrost studies), Part II, Engineering geocryology, Chapter VI. Academy of Sciences of the U.S.S.R. Moscow 1959. p. 140-218

(Osnovy geokriologii (merzlotovedeniya), Chast' vtoraya, Inzhenernaya geokriologiya, Glava VI. Akademiya Nauk SSSR. Moskva 1959. s.140-218)

Translator: G. Belkov, Translations Section, National Science Library

BASES AND FOUNDATIONS*

1. The performance of a foundation depending on where it is located with respect to the permafrost. 2. General requirements of bases and foundation. 3. Bases and foundations constructed with preservation of the permafrost. 4. Calculations for bases and foundations on erection of structures with preservation of the permafrost (supplementary calculations for checking the general stability of structures). 5. Bases and foundations of structures designed for differential settlement during gradual thawing of the foundation soils. 6. Bases and foundations of structures erected with preconstruction thawing of the foundation soils. 7. Measures to prevent heaving and calculation of the stability of foundations on heaving. 8. Foundation work carried out in the permafrost region.

1. The Performance of a Foundation Depending on Where it is Located with Respect to the Permafrost

Under ordinary building conditions, i.e. in regions where there is no permafrost, it is usually assumed that the entire load of the structure is transferred to the foundation soil by the footing; the soil in contact with the sides of the structure assumes vertical loads only in particular cases (piles and deep foundations).

This simple mechanism for the transfer of load, being rather well founded for such cases cannot be extended to the interaction between the foundation and the foundation soil in regions where permafrost and deep frost penetration occur. Here the load is transferred to the soil by the entire contact surface between the soil and the foundation; the value and even the direction of the effect on individual parts of the foundation surface vary in relation to the position of the foundation with respect to the frozen soil.

Typical diagrams of the position of the foundation with respect to the frozen soil. Figure 33 shows five basic diagrams for the positioning of simple post or wall foundations. Uniform geocryological conditions are assumed.

The first three diagrams are for foundations laid below the layer of seasonal freezing and thawing and the influence of annual heaving and settlement of the soil applies only to the lateral surfaces of the foundation. The last two diagrams represent foundations laid in the layer of seasonal freezing

* Translator's Note: The Russian term "osnovaniye" (base) actually means the material on which the foundation bears, not the bottom of the structure. Thus a more accurate rendering in English is either "foundation soil" or "bearing medium". "Base" is used in some places for ease of wording.

and thawing soil undergoing the influence of heaving and settlement not only applied to its lateral surface but also to the bottom of the footing.

Diagrams I, II, and III are differentiated by the location of the footing with respect to the permafrost table, namely: in diagram I the footing is located on the permafrost table or below it; in diagram II the foundation is placed above the permafrost table but deeper than the depth of seasonal freezing; in diagram III the position of the footing with respect to the permafrost table is subject to variation after the building is put to use, namely: during construction the permafrost table was above the footing and after thawing it is lower.

At times the typical cases of the placing of foundations shown in Fig. 33 may become complicated, for example, under one part of the structure the footing may be placed in the permafrost and the footing under another part may lie above it.

Phenomena taking place at the contact between the foundation and the frozen soil. In the layer of soil adjacent to the surface of the foundation there are a number of complex physical phenomena of moisture and heat exchange, changes in the phase composition of the soil and mositure, etc. Free moisture accumulates at the surface of contact between the foundation and the soil, which on freezing facilitates the formation of a frozen bond between the foundation surface and the soil. A strong bond is formed by means of which tangential and normal forces are transferred from the soil to the foundation and from the foundation to the soil, and the value of the forces transmitted is limited by the strength of the bond. The strength of the frozen bond depends on temperature and other factors as explained in Chapter III.

The direction of forces generated by the interaction between the foundation and the soil varies mainly in relationship to the time of year. In the freezing and thawing layer the tangential forces transmitted by the soil to the foundation are directed upwards during the period of heaving and, as shown by A.M. Pchelintsev (1956), downwards during settlement. Within the limit of the perennially frozen soil at the point of contact with the lateral surface of the foundation, tangential forces may also develop; during the warm season the frozen soil receives from the foundation part of the load and part of its dead weight; during the cold season it increases the resistance of the foundation heaving (it becomes an "anchor"). The same can be said with respect to forces normal to the footing of the foundation. Depending on the overall effect of these forces, the foundation can be either stable or unstable (owing to heaving or settlement) and the uneven displacement either upwards or downwards may be the cause of deformation of the structure. Thus the freezing and thawing of the soil should be regarded as a factor influencing the distribution of external forces acting on the foundation.

Forces acting on the foundation. The basic purpose of the foundation under permafrost conditions, as under ordinary conditions, is to transfer the external load to the foundation soil, i.e. the dead weight of the structure, its useful and various additional load.

Usually the total resultant force of these loads in the plane of the foundation edge is expressed by the components N_1, Q_1 and M_1, the value of which in the general case can vary in passing from one part of the foundation to another, depending on the variation in the external load or the method in which it is applied.

Under ordinary conditions when freezing and thawing of the soil occurs rather slowly and does not exert any important influence on the performance of the foundation it is assumed that the entire external load is fully transmitted to the bearing medium in the plane of the foundation footing in the form of the component $N = N_1 + N_f$; $Q = Q_1 + Q_f$ and $M = M_1 + M_f$ where N_f, Q_f and M_f correspond to the dead weight of the foundation and the pressure of the soil on it. In the plane of contact between the foundation and the bearing medium reactive forces develop N_r, Q_r and M_r, which are equal in value but of opposite sign to N, Q and M.

The action of freezing and thawing leads to different mechanism for the distribution of external forces determined by: (1) partial or complete unloading of the foundation soil owing to the external load being taken by the frozen seasonally freezing layer; (2) unloading of some portions of the bearing medium owing to the overloading of others.

Let us consider the relationship between the performance of the foundation and processes of freezing and thawing of the soil. We will denote by T_h the tangential force transmitted to the foundation by the soil that is being frozen to the foundation and rising gradually.

This force is not constant and depends on many factors. At the place of contact between the frozen layer of soil and the side of the foundation there occur plastic displacements which decrease the value of the tangential force transmitted to the foundation. As long as the tangential force T_h remains less than the load on the foundation, its dead weight and the friction against the underlying unfrozen soil combined in equation (6.1) by the symbol N, the foundation occupies its initial position and to some, although reduced, extent transmits the external load to the foundation soil.

Here the relation between the value of the load N, the reaction of the bearing medium N_r and the forces T_h received by the freezing layer is expressed by the equation

$$N = N_r + T_h. \tag{6.1}$$

At the moment when the force T_h becomes equal to the load N the reaction of the foundation soil is reduced to 0 and the entire load is transmitted to the soil through the layer frozen to the lateral sides of the foundation, and if up to this time the process of heaving has not finished, the foundation begins to rise, being lifted by the frozen soil.

In this sense the forces of frost adhesion between the foundation and the soil must be considered as reactive forces. It can be said that during the freezing of the soil the external load is balanced by two forms of reactive forces: normal forces applied to the footing of the foundation and tangential forces applied to its lateral surface.

In diagram I (Fig. 33) the lateral surface of the foundation freezes not only to the soil of the seasonal freezing layer but also to the perennially frozen soil. In the latter case tangential frost adhesion forces T_s are generated and the equilibrium condition (6.1) of the foundation with respect to vertical force components will have the form

$$N = N_r + T_h - T_s. \tag{6.2}$$

Analogously one can write equations of equilibrium for displacement forces (6.3) and moment (6.4). Let us denote the shear forces generated at the plane of the foundation footing by the external load and by soil heaving by Q_{ex} and Q_h. The effect of the shear forces caused by heaving can be added to that of the shear forces of the external load if both are acting in the same direction. In this case the equilibrium conditions can be written in the form

$$Q_{ex} + Q_h \leqslant H, \tag{6.3}$$

where H denotes the possible shear strength of the frozen soil in which the foundation is frozen and the friction of the footing against the soil.

If the heaving forces act in a direction opposite to that of the external load they do not participate in shear nor do they resist shear since by their very nature they can only bring about compression but not expansion of the soil. Then the equilibrium conditions will be written in the form:

$$Q_{ex} \leqslant H. \tag{6.3a}$$

The construction of an equilibrium equation when a tipping moment M is generated in the plane of the foundation footing presents some difficulty because the distribution of reactive forces resisting tipping are not sufficiently known. For a reserve of stability we assume that:

(1) the rotation of the foundation under the influence of the moment takes place around the edge as it would around an axis (Fig. 34);

(2) the resistance to rotation is due to normal reaction forces N acting on the foundation footing ae and to the tangential forces $T = \tau_{max} h_3$ along the surface de of frost adhesion between the foundation and the perennially frozen soil with the force h_3. Then the condition of equilibrium for the summertime for a unit of foundation length is written in the form of the equation

$$M \leqslant b\left(\frac{bn}{3} + h_3\tau_{max}\right), \qquad (6.4)$$

where n is the maximum pressure compressing the foundation soil;

τ_{max} - the maximum tangential pressure of the frost adhesion between the lateral surface of the foundation and the soil;

b - the width of the foundation in the direction of the moment M.

During the period of heaving additional forces are generated along the lateral surfaces of frost adhesion between the foundation and the freezing soil. From the formula it can be seen that heaving at the surface bf has no influence on the stability of the foundation. Heaving forces acting on the section ng cause a moment that is inverse to the moment M. However, the total effect of heaving forces on the surfaces bf and ng, acting to unload the footing, reduces the reserve stability of the foundation. Thus conditions of tipping during the period of heaving of the foundation soil will be less favourable.

The particular features of the performance of foundations erected on perennially frozen soil consist in continuous variation of external forces, with respect to time, acting on the foundation. In summer the reaction of the bearing medium on the foundation footing N_r reaches the highest values but is still less than it would be in the absence of perennially frozen soil since $T_s \neq 0$. In winter it decreases and may be reduced to 0 when all of the load of the structure will maintained by tangential forces of frost adhesion applied to the lateral surface of the foundation.

In areas where there is no permafrost the equilibrium condition of the foundation in the absence of tipping and shear forces is expressed for winter by the expression $N = N_r + T_h$ and in the summer by the equation $N = N_r$ which are usually used in design.

The performance of a foundation wall under non-uniform conditions along its length. In the simplest case of a post foundation or wall foundation under uniform geocryological conditions, equation (6.2) remains valid for each section of the foundation. If, however, the conditions of soil freezing at one section differs from that of other sections, the forces used in equation (6.2) are redistributed along the length of the foundation. The redistribution of external forces acting on the foundation (N, T_h) consisting in reducing the load from one section and applying an additional load on others, may occur during freezing or thawing.

As a result of non-uniform lifting of separate portions of the foundation soil of the structure during freezing, the load of the structure is transmitted primarily to portions subject to the greatest amount of heaving, and conversely the least load is applied to portions undergoing the least heaving. Analogously, as a result of differential settlement on thawing, the portions that have settled the greatest bear the least load, at times zero load, whereas portions undergoing small settlement bear loads which reach the maximum bearing capacity of the soil. Correspondingly, there is a redistribution in the reaction between the bearing medium and the foundation. As a result of this type of redistribution of reaction, which is considered in more detail in section 5, substantial additional forces are generated in the foundations and connected parts of the structure as in a three-dimensional system.

Denoting the unit area of the base by $\Delta\omega$ and the maximum load, the normal forces of reaction, tangential reaction forces and frost adhesion forces between the foundation and the perennially frozen soil by N, N_r, τ_h, τ_s, we obtain for vertical forces the equilibrium condition in the form

$$\Sigma n\Delta\omega = \Sigma n_r\Delta\omega + \Sigma\tau_h\Delta\omega - \Sigma\tau_s\Delta\omega. \tag{6.5}$$

Performance of the foundation. The geocryological conditions on which the performance of a foundation depends are not constant in time. Consequently the conditions under which a foundation performs are subject to change during the period of construction and when the structure is put to use. We will consider briefly the performance of a foundation from this point of view using the diagrams shown in Fig. 33 and assuming that at the moment the foundation is laid the geocryological conditions are uniform over the entire building site.

Diagram I. The foundation footing is placed at a depth h_f below the ground surface and at a depth of h_f-H from the permafrost table, and during the entire time the structure is in use the depth of the permafrost table does not decrease and the condition h_f > H is preserved. Diagram I corresponds to construction with preservation of the foundation soil in a frozen state, at which the simplest and most favourable conditions of the performance of the structure occur (Chapter I). Resistance to heaving is provided by observing conditions (6.2) by means of design measures indicated below in section 7. The reactions of the bearing medium at the footing are determined just as they would be if the foundation rested on firm soil.

At the time of construction of the foundation H > h, the condition H < h occurs only the second or third year after the structure has been in use; in other words, after one or two winters the permafrost table rises. In this case, during the first year the performance of the foundation is complicated by the fact that the soil under the footing may be subject to heaving under normal heaving forces which are still insufficiently known and apparently are rather high.

Diagram II. The foundation is laid deeper than the layer of winter freezing and above the permafrost table; the soil at the bottom of the foundation is unfrozen at the moment of construction, as well as throughout the entire period the structure is in use.

Here two cases are possible that are different in principle: first, when the thawing of deep lying permafrost has no influence on the stability of the structure, and second, when the thawing may have an influence on the stability and the engineer must take it into account. The particular features of the performance of the foundation under conditions corresponding to diagram II have not been investigated and one can only consider that they depend on many factors: the distance from the footing to the permafrost table, the dimensions of the foundation, the heat emission from the structure, the physico-mechanical properties of the soil, etc. Practice has shown that with average heat emission from the structure under average compaction and ice content of the soil, thawing of frozen soil lying 6 - 8 m below the footing has no influence on the stability of the structure. In particular cases there may be no permafrost under the structure whatsoever - for example, in erecting a building on a talik extending right through the permafrost.

Since in the case under consideration $T_s = 0$ equilibrium conditions of the foundation for diagram II, when the permafrost table is sufficiently deep, are expressed by the equations:

for winter $$N = T_h + N_r, \qquad (6.6)$$

for summer $$N = N_r, \qquad (6.7)$$

which must be put at the basis of calculating the strength of the foundations and bases. In designing foundations outside the permafrost zone only the second equation is usually considered. The unloading of some sections of the bearing medium at the expense of others and the associated generation of bending moments in parts of the structure when the position of the foundation with respect to frozen soil is according to diagram II, may occur mainly because of non-uniform tangential heaving forces T_h for relatively light structures.

Diagram III. The condition illustrated in diagram III is characterised by local thawing of the permafrost under a structure already placed to a depth H > h. This case is particularly unfavourable.

As a rule, the thawing of the foundation soil occurs unevenly over the area of the building site which results in non-uniform distribution of the reaction of the foundation soil. Therefore the equilibrium condition of the foundation corresponding to diagram III must be analysed specifically for the entire foundation as well as for individual sections as applicable to section 5. By artificially heating the foundation soil to a certain depth before the foundation is laid, one can bring the geocryological conditions of the building site to conditions similar to diagram II. This idea was developed by Z.F. Zhukov and is presented below in section 6 of this chapter.

Diagrams IV and V, differing only in the depth of the permafrost table, correspond to the case of laying the foundation in a layer of soil that thaws during the summer. In this case the equilibrium conditions of the foundation differ from the foregoing in the following manner:

(a) instead of tangential forces arising from the freezing of the foundation to the perennially frozen soil T_s which counteract heaving of the foundation in diagram I, there is the force arising from the compression resistance of the thawed soil T_t (Porkhaev, 1957);

(b) normal heaving forces N_h acting on the footing are added which along with tangential heaving forces T_h can exceed the load N. In this case stability conditions of the foundation are expressed by equation (6.6a):

$$T_h + N_h \leq N. \qquad (6.6a)$$

If $N < T_h + N_h$ the foundation is raised by heaving of the soil.

Between diagrams I and V their may be transitions. For example, when at one section of the foundation there is a combination of conditions corresponding to the different diagrams, one frequently observes in practice cases one part of the structure resting on soil where the seasonal freezing reaches down to the permafrost (diagrams I, III, IV), and another part of the structure on soil a permanently unfrozen stratum of soil above the permafrost (diagrams II, V). In view of the extreme variations in conditions of this type of combination they should not be considered together but the attention of designers and surveyors should be drawn to the fact that such cases are exceedingly complex and should be approached on an ad hoc basis.

Concerning the performance of foundations in relationship to the presence of frozen soil and the development of freezing and thawing, the following supplements should be added to what has been given above.

1. In selecting building sites it is recommended that sites with highly variable geocryological conditions including variable depths to the permafrost table should be avoided, since construction and utilization of a

structure under such conditions is complicated.

At times it is possible and expedient to have the frozen soil at a uniform depth; this can be accomplished by applying heat before construction begins.

2. Keeping in mind that the diagrams in Fig. 33 determining the performance of a foundation depend not only on the natural features of the site but also to a great extent on the method of contruction and the application of heat to the frozen soil, it is not recommended to use the contrasting schemes I and III in one building site.

For different buildings erected on the same building site the contrasting schemes I and III are permitted if the distance between the buildings is sufficiently large and the possibility of heat transfer through the foundation soil is eliminated. In practice, when a building is located on the ground and subject to moderate heat emission this distance is taken to be 10 - 15 m. In the construction of a structure with high heat emission, and particularly when the basement space is heated, one must either increase this distance or provide for special cooling-ventilating systems to prevent the foundation soil from thawing.

3. For large buildings schemes I and II are preferred over schemes IV and V, with the exception of foundation soils which are not subject to heaving on freezing or settlement on thawing.

4. The most favourable schemes for the performance of a foundation are I and II which are characterized by a constant physical state of the foundation soil.

5. Conditions characterized by scheme III are the most complex for designers and builders. Construction by this scheme is carried out usually only when absolutely necessary or where it is exceedingly difficult to preserve the frozen state of the soil.

2. General Requirements of Bases and Foundations

Experience in erecting engineering structures under permafrost conditions has shown that their strength and stability can be ensured by different methods listed in Chapter I and in the corresponding standards (NiTU 118-54, etc.).

Comparing these methods with the schemes in Fig. 33 it can be seen that scheme I corresponds to preservation of the frozen state of the soil, scheme II - to the preliminary thawing of the soil and scheme III to the design of the structure to withstand settlement. The engineering economics of the choice depends on the following:

(a) on the nature of the perennially frozen soils (the physico-mechanical composition in the frozen and thawed state, depth of the permafrost table, temperature regime and thickness of the permafrost, etc.);

(b) on the climatic conditions of the region (mean annual air temperature duration of the frost and frost-free period, time and amount of precipitation and wind conditions);

(c) on the type of structure, its thermal regime and condition under which it is used.

The choice of the type of foundation is of great importance in designing the structure for permafrost conditions.

Because of the variety of conditions occurring in areas where permafrost is present, all types of foundation known in building practice can be used if they are modified and adapted to local conditions. However, it would be incorrect to think that the choice of the type of foundation under these conditions is irrelevant. On the contrary, practice and theory in the design of foundations have shown that the types of foundations which have proved satisfactory under some conditions (conditions and not regions) where perennially frozen soil occurs, may be unsuitable for others.

The shape, construction and design of foundations are determined by the particular features of their performance, external load, method of construct and the geocryological properties of the foundation soil. We are considering only the influence of the last two factors with the assumption of a static vertical load, and the data given below is applicable to such a load.

However, this limitation refers only to structural details and design diagrams. The principle of the problem does not change with the nature of the external load.

3. Bases and Foundations Constructed with Preservation of the Permafrost

The frozen state of the foundation soil is maintained when the depth of summer thawing under the building is regularly less than the depth of winter freezing. This method of construction, as noted above, has been used widely.

Construction with preservation of the permafrost at the base of structures in most cases involves regulating the temperature of the perennially frozen soil and layers of soil subject to summer thawing. For example, when the temperature of the perennially frozen soil is relatively high (close to 0°C) such measures can result not only in preserving the frozen state of the soil but even in decreasing the temperature and thus increasing its bearing capacity. Reducing the depth of summer thawing is accompanied by reducing the area of the frozen bond between the lateral surface of the foundation and the layer subject to heaving; as a result the heaving forces are reduced.

In regions where the temperature of the permafrost is low, preservation of the foundation soil in the frozen state may involve an increase in its mean temperature in comparison with that of the surrounding soil in its natural state, where reducing heat losses of the building is an important consideration.

The general natural trend in the formation of perennially frozen soils in the vicinity of the building site is not particularly important in preserving the frozen state of the soil at the base of the structure if provision is made for sufficiently powerful refrigeration devices. Even in the case of a highly unstable state of the frozen soils at the base when even small distubances in the natural conditions of heat exchange, for example the removal of vegetation cover, results in a rapid disappearance of the permafrost, one can preserve the frozen state of the foundation soil and even improve its bearing capacity by decreasing the temperature.

To preserve the frozen state of the foundation soil two basic types of cooling devices are used.

1. Cooling devices operating on the principle of ventilation with cold outside air (with natural or forced ventilation).

2. Cooling devices with the use of artificial methods of refrigeration (refrigeration machines, ice-salt cooling, etc.)

As a supplementary measure, heat insulating covers and spacers are used which reduce the quantity of heat penetrating from the heat-evolving structure to the ground.

Systems operating with artificial refrigeration have not been widely used in the North since the climatic conditions in most cases permit the preservation of the frozen state of the soil under structures which evolve large quantities of heat by the proper design and operation of cooling devices using cold winter air. Therefore, we consider below only devices for cooling the soil using outside air.

Design features of heated structures erected with the intention of preserving the frozen state of the foundation soil. In designing heated structures to preserve the frozen state of the soil one should take the following measures beneath them: (a) make provisions for and develop cooling devices of the required size and provided for proper insulation; (b) ensure the proper construction of enclosures: floors, perimeter walls and for buildings set in frozen soil the walls must also be considered; (c) to design decking over the ground which would protect the structure from the inflow of surface water and, where necessary, also from groundwater if it is present at the building site.

Cooling devices designed to use outside air at low temperature in the design and operation are divided into two types: surface and underground. When possible preference should be given to the surface type.

Surface cooling arrangements. Surface cooling arrangements are made in the form of open crawl spaces, crawl spaces with openings in the perimeter wall of the building and with draft ducts. The rather rarely used system of ventilation conduits placed directly on the ground surface is also included in the surface cooling system.

The open crawl space method without foundation walls is rather antiquated construction; it causes excessive heat losses and at the present time can be recommended only for exceptional cases where structures have a very high heat emission and in regions with a relatively high mean winter temperature, for example in the Transbaikal region.

In most cases the frozen state of the bearing medium under a heated building can be ensured by ventilating the crawl space through openings in the perimeter wall. This type of design (Porkhaev, 1952) has the advantage over an open crawl space in that ventilation of the crawl space can be controlled and the opening may be closed during the coldest part of the winter to reduce heat losses from the building, and in wind swept regions during strong blizzards the openings can be closed to prevent the crawl space being filled in with snow.

In regions with frequent and strong blizzards, and also where buildings are very close together, the use of open crawl spaces and crawl spaces with ventilation through openings in the perimeter wall does not always ensure a temperature regime necessary to preserve the frozen state of the soil. In these cases it is best to ventilate the crawl space by using draft ducts which discharge above the high point of the roof. To increase the draft, and consequently to decrease the number of ducts and their diameter, they may be located inside the building in staircases, corridors, etc. In such cases the thermal resistance of walls of the ducts should be such as to prevent condensation of moisture on the surface.

The system of ventilating the crawl space, the size of the openings, the distance between the wall-supporting beam and the ground surface in the case of an open crawl space, the number and diameter of draft ducts, etc., are determined either from experience in erecting similar structures under similar climatic and cryological conditions or by calculation. Some current methods of calculating the thermal regime of the crawl space are those of N.I. Saltykov and N.N. Saltykova (1950) and G.V. Porkhaev (1952).

The height of the crawl space is not based on cooling considerations but is determined by design considerations based on the minimum permissible height of the ceiling of the crawl space above the outside decking expressed by the condition

$$H_c \geqslant H_d + a + b + c,$$

where H_c - height of the crawl space ceiling;

H_d - height of the surface of the decking near the foundation wall;

a - minimum height of the opening in the perimeter wall above the surface of the decking determined by the quantity and type of precipitation at 15 - 25 cm so that the precipitation falling on the decking in the form of rain or snow will not fall into the opening of the crawl space;

b - heights of the opening 25 - 30 cm;

c - distance from the top of the opening to the ceiling of the crawl space 10 - 20 cm.

Thus the design height of the crawl space ceiling above the decking varies from 0.5 to 0.75 m. The ground in the crawl space is levelled and where required is insulated with rot-resistant insulating material such as slag or slag concrete. If plumbing is installed in the crawl space the height of the crawl space is increased to 1.5 - 2.0 m to facilitate inspection and repair (Maksimov, 1956).

In the ventilated crawl space there should be no structure interfering with the free movement of air such as blind perimeter walls or other walls separating the crawl space into sections that do not provide for free air movement. Experience has shown that interior load-bearing foundation walls should have at least 25% of their above-ground area open.

Ventilated crawl space is also provided under heavy equipment which therefore must be placed not on solid foundations but on slabs supported by post foundations (Fig. 35).

Underground cooling devices. Underground cooling devices such as pipes and ducts (Fig. 36) have to used in buildings with a large load on the floor of the first storey when it is impossible to provide for a crawl space. In such cases silty soil subject to heaving must be replaced to the depth of summer thawing by soil that is not subject to heaving to avoid deformation of the cooling ducts and the floor of the first storey. The cooling ducts are located near the foundation of the building and can be single or in groups.

Underground cooling devices should have the following requirements: (a) accessibility for inspection, cleaning and repair, for which the diameter of underground ventilation ducts should be not less than 0.6 m; (b) unconditionally reliable waterproofing; (c) they should be located above the foundation footing.

Ventilation of the underground cooling pipes is either by draft ducts with deflectors or by mechanical ventilation. The design of the inlets should be such as to keep them free of snow in winter and water in summer.

Of the many factors connected with the preservation of permafrost at the base of heated buildings, the most important are the floor of the first storey, the perimeter wall and decking, and for buildings that are partially underground one must also consider lateral enclosures.

The floor of the first storey of apartments and industrial buildings should have a thermal resistance of not less than 3 m^2 hour x degree/kcal and must be draftproof which is provided for by tar paper or other roll material which covers cracks. Particular care must be given to protecting the dwellings and working space from draft at the point of contact between the floor and the outside walls and the load-bearing beams.

The perimeter walls of a building erected with the intention of preserving the permafrost can be either load bearing or not; if the wall is not load bearing it serves the purpose of protecting the floor space from wind, snow and rain as well as for decorative purposes. In masonry buildings the protective wall is built of brick set on edge and for wooden buildings it is made of horizontal board siding.

The careful construction of decking in permafrost areas is of great importance since water accumulating around the foundation can on melting facilitate the thawing of the frozen soil; the thawed soil settles and around the building a depression is formed into which more surface water flows, causing heaving and settlement of the foundation. In building construction on permafrost the construction of decking is essential not only where it is necessary to preserve the permafrost but should also be used in conjunction with all other methods of ensuring stability of the structure. In building construction with preservation of the permafrost the decking may serve not only as a protection from surface water but also as protection against the influx of groundwater above the permafrost. In such cases an insulating decking is constructed from slag or other material possessing low heat conductivity.

An important part of the insulating deck (Fig. 37) is a covering of planks or reinforced concrete flags raised above the surface of the slag prism by 5 - 10 cm which protects the slag during the summer from direct solar radiation and in winter from snow. The deck covering can serve as a sidewalk. If it is undesirable to use the deck covering as a sidewalk the deck should be raised above the level of the sidewalk by at least 5 - 10 cm to provide runoff for surface water and between the deck and the sidewalk a runoff trough should be provided. It is desirable to have the top of the foundation footing at the same level as the deck covering so that the deck will not overlap the perimeter wall. Experience in the performance of buildings in Yakutsk has shown that a properly made insulated deck reliably protects not only the foundation and perimeter wall but also the basement, where applicable, from groundwater above the permafrost table.

Types of foundation used in construction with preservation of the frozen state of the bearing medium. As a bearing medium, frozen soil must be considered if not absolutely rigid, at least subject to little compression and under load usually does not settle more than several millimetres.

This simplifies the mechansim of the transfer of pressure from the structure to the bearing medium and one can in the first approximation eliminate from consideration the question of redistribution of the reaction of the bearing medium; if the pressure of the foundation on the frozen soil does not exceed the permissible long-term load (Chapter III), in designing foundations for large-scale construction such frozen soil can be considered practically incompressible. On erecting the foundation with preservation of the bearing medium in the frozen state, post type foundations are usually used; foundation walls are less common.

Post-type foundations connected across the top by a load bearing beam are the most convenient type. In addition to the generally known advantages such as economy of material and a small amount of earth work, such foundations have a number of special advantages: (a) they fit well with the construction of a cooled crawl space under heated buildings; (b) being anchored in permafrost they resist tangential heaving forces better than other types of foundations; (c) they conduct a small amount of heat to the frozen soil; (d) they are subject to the influence of underground services less than any other type of foundation.

When the reaction to heaving forces does not apply a load on the foundation, i.e. when equilibrium conditions are expressed by the equation

$$T_s = T_h - N > 0, \tag{6.7a}$$

the foundation posts should be made of reinforced concrete or wood capable of withstanding a tension force of T_s. Stone and concrete can be used only where the soil of the seasonally freezing and thawing layer is not subject to heaving (sand, gravel) or under heavy structures when it is evident that $N > T_h$ and when equilibrium conditions are determined by the equation $N = T_h + N_r$ and T_s is either equal to 0 or act in the same direction as T_h.

Until recently, foundation posts under buildings erected with preservation of the permafrost were made primarly in the form of columns with footings on the end. In more recent times in connection with investigations of S.S. Vyalov

K.E. Egerev, M.V. Kim and T.I. Mel'nikov, the possibility has arisen of substantially increasing the pressure exerted on permafrost (Chapter III) and also the possibility of taking into account the reactive tangential forces of adfreezing between the lateral surface of the foundation and the soil in calculating the bearing capacity of foundations. These new factors have brought about a tendency to decrease the bearing area of the footings and to a transition from foundation posts with footings to those without footings which substantially decreases the cost and time spent on foundation work. Simultaneously K.E. Egerev (1956) suggested a reinforced concrete pile of a double T cross-section having an increased bearing capacity owing to a substantial increase in the reactive tangential forces of adfreezing with the permafrost (Fig. 38).

The footing of a column serves not only as a bearing surface but also as an anchoring device and in some cases, for example, when there is a large quantity of rock-like inclusions in the soil making it difficult to drive the pile, columns with footings are still widely used as foundations for building with preservation of the permafrost and it would therefore be useful to make the following remark regarding their use.

1. The most suitable type of foundation consists of sectional posts in conjunction with a precast reinforced concrete footing. The placement of precast footings in the foundation trench can be carried out during any time of the year in a layer of dry sand of sufficient thickness to even out irregularities in the bearing surface.

2. When the concrete footings are poured in place in the foundation trench, a row of wooden beams, with cross-sections of 20 x 20 to 25 x 25 cm, and in northern regions two such rows of beams, should be placed at the bottom of the trench.

In these cases the row of beams is not so much to preserve the foundation soils in a frozen state as to provide normal hardening of the concrete (Chernigov, 1957).

Individual wooden posts used under wooden buildings are installed in the following way: (a) in the form of smooth posts buried in the foundation trench, (b) in the form of piles, (c) in the form of posts with footings of various types (Yakovlev, 1952). When there are horizontal forces acting on the foundation one must of course take them into account, which is reflected in the design of the foundation but does not introduce any important changes in the above procedures.

Foundation wall. In some cases to eliminate the horizontal load-bearing beam the foundation is constructed in the form of a wall. This type of foundation under the conditions considered here has a number of defects which limit its application: it has a large adfreezing area along the lateral surface of the foundation which increases the heaving forces; it involves a larger amount of excavation and is subject to relatively large heat conductivity.

Because of their weak resistance to heaving and because of their complex anchoring in frozen soil, foundation walls can be permitted only for buildings erected on dry non-heaving soils. Under such conditions they can be erected in the usual manner employed outside the permafrost zone. Such foundations are usually made of rubble-stone concrete with careful parging of the walls. Foundation walls should not be built of stone masonory which are subject to rapid destruction because of water freezing in the joints.

Massive foundations. Massive foundations are most frequently used to support large engineering structures such as bridges, furnaces and heavy industrial machinery. The main defect of this type of foundation is the difficulty in preserving the soil underneath them in a frozen state. If a massive foundation supports a structure or machinery unit emitting heat, or if such equipment is located in a heated room, the only way to preserve the ground in the frozen state under the foundation is by periodic cooling of the foundation by means of cooling ducts running through the foundation. An example of this type of procedure is the foundation under a brick kiln furnace in Pokrovsk (Hofman furnace - Fig. 39 and a floor-mounted furnace - Fig. 40) which have been in use for several years. Efforts to counteract heat penetration into the frozen ground by installing insulated spacers were unsuccessful.

It should be noted that the wall and massive types of foundation are not typical of large-scale construction with preservation of the frozen state of the foundation soils but are used for some structures.

4. Calculation for Bases and Foundations on Erection of Structures with Preservation of the Permafrost (Supplementary Calculations for Checking the General Stability of Structures)

In designing foundations with preservation of the frozen state of the bearing medium the following calculations are required:

(a) Thermal engineering calculations by which one can establish the possibility of preserving the foundation soil in its frozen state under specific conditions; the degree of reliability of such a state is expressed by the excess of winter cooling of the soil over its heating under the most unfavourable circumstances; thermal engineering calculations include also the computation of the greatest possible depth of summer thawing on which is based the depth at which the foundation is laid; the temperature of the soil and its variation are determined to establish the data required for statistical calculations of the long-term compression and shear strength of the frozen soil;

(b) Statistical calculations for checking the strength and stability of bearing media and foundations for given dimensions and depth of laying of the foundations, material, design load and, taking into account local conditions, the temperature of the soil, its ice content, texture, unit weight, etc.

1. Thermal Engineering Calculations of Surface Cooling Arrangements

General Considerations. The most widely used type of surface cooling arrangement is the ventilated crawl space. The calculation of the thermal regime of a ventilated crawl space should take into account its heat exchange with the building, with the ground and the surrounding atmosphere. Fig. 41 shows diagrammatically the most typical combination of heat flow between a heated building, cooling crawl space, the ground and the outside air in winter and in summer (Porkhaev, 1952).

Following from this diagram the thermal balance of the crawl space can include the following heat exchange components.

1. The heat flowing from the building to the crawl space through the floor (Q_1).

2. Heat lost to the atmosphere through the perimeter wall of the building (Q_2).

3. Heat flowing from the bearing medium during freezing of the active layer under the structure (Q_3).

4. Heat going to thaw the seasonally freezing layer and to warm up the permafrost (Q_4).

5. Heat eliminated from the crawl space by a particular ventilating system (Q_5).

6. Heat lost to the bearing medium owing to the difference in temperature between the permafrost and the ground surface in the crawl space and also heat lost to the atmosphere through the soil owing to lateral heat flow (Q_6).

It should be noted that the heat balance component and the calculation formulae given below are valid for surface cooling arrangements of any type.

Calculations are carried out in the following manner.

1. The estimated (lowest) air temperature is determined for the crawl space during the summer without permitting condensation of water vapour in the crawl space.

2. The thermal balance of the crawl space is constructed, from which is found the air exchange required to maintain the required air temperature in the crawl space during the summer.

3. The depth of soil thawing under the structure is determined for the summer period.

4. The air temperature in the crawl space during winter required to freeze the layer of ground thawed during the summer is determined.

5. The thermal balance of the crawl space is constructed from which is found the required air exchange during the winter; the possibility of preserving the frozen state of the soil at the base of the building without ventilating the crawl space is determined.

6. A calculations is made of the ventilation arrangements if ventilation turns out to be necessary according to item 5.

<u>Determination of the estimated air temperature in the crawl space during the summer</u>. Ventilation of the crawl space is carried out during the summer to protect the building from dampness while keeping the thawing of the foundation soils to a minimum.

For buildings with large thermal emission the air temperature in the crawl space may be higher than the temperature of the outside air, and consequently ventilation of the crawl space by the cooler outside air facilitates preservation of the foundation soil in a frozen state.

The air temperature in the crawl space of ordinary apartment buildings can be found with the formulae:

1. In regions with relatively high ($\geqslant -2°C$) mean annual ground temperatures

$$\vartheta_c = \frac{\vartheta_1 - \alpha_n R_{fl} \vartheta_{dp}}{1 - \alpha_n R_{fl}} + \vartheta. \qquad (6.8)$$

2. For low mean annual ground temperatures (below $-2°C$) when at the beginning of the warm period there may be no thawing of the soil and the entire heat evolved by the building goes to increase the temperature of the frozen stratum

$$\vartheta_c = \frac{\vartheta_1 \sqrt{\pi a t} + 2\lambda_{fr} R_{fl} \vartheta_o}{\sqrt{\pi a t} + 2\lambda_{fr} R_{fl}} + \vartheta. \qquad (6.9)$$

3. In regions with a continental climate, where low humidity of the outside air is characteristic for the beginning and end of the summer, the air temperature in the crawl space for the first and last months for the period of above freezing mean monthly air temperatures, and for buildings with high thermal emission for all of the summer months, is found with the formual:

$$\vartheta_c = m - \sqrt{m^2 - \left(m - \frac{80\lambda_T R_{fl}^2 \omega}{t}\right)^2 + \left[(\lambda_T R + h_o)\frac{80\omega R_{fl}}{t}\right]^2} + \vartheta. \qquad (6.10)$$

The conventional notations used in formulae (6.8), (6.9) and (6.10) are given below:

- ϑ_1 - air temperature in the first storey of building in degrees,
- ϑ_c - air temperature in the crawl space in degrees,
- ϑ_{dp} - dew point of the outside air, in degrees,
- ϑ_o - temperature of the perennially frozen strata at the depth of zero annual amplitude, in degrees
- R_{fl} - thermal resistance of the floor above the crawl space, m^2 x hours x degrees/kcal,
- R - thermal resistance to heat transfer from the ground surface to the air in the crawl space, m^2 x hours x degrees/kcal,
- λ_T, λ_{fr} - heat conductivity coefficient of thawed and frozen ground, kcal/m x hours x degrees,
- α - diffusivity coefficient of frozen ground m^2/hours
- ω - quantity of frozen water in the soil, kg/m^3,
- α_n - coefficient of heat transfer from the air in the crawl space to the floor, kcal/m^2 x hours x degrees,
- h_o - depth of thaw of the ground before the beginning of the calculation time interval t, m (initial depth of thaw),
- t - calculation of time interval in hours,

$$m = \vartheta_1 + (\lambda_T R + h_o + \lambda_T R_{fl}) \frac{80\omega R_{fl}}{t}.$$

Supplementary changes in air temperature in the crawl space due to ventilation and heat flow to and from the crawl space through the perimeter wall of the building is taken into account by corrections ϑ in the formulae. For the crawl space of apartment buildings this correction ϑ, should be taken as 0.5 - 1.5°C. For crawl spaces of buildings with large heat emission where accelerated ventilation of the crawl space by cool outside air reduces the depth of thaw under the building, the value of ϑ should be taken within the range of -0.5 to -1.5°C. The limits of these corrections were derived by considering the minimum increase in air exchange with simultaneous minimum increase in depth of thaw. The accuracy in choosing the calculation formula for determing the air temperature in the crawl space is further checked by the thermal balance from which the air exchange between the outside air and the air in the crawl space is determined. If according to calculations the quantity of air required to maintain the air temperature in the crawl space as calculated with formulae 6.8, 6.9 and 6.10 has a negative value the formula is incorrect and the air temperature should be recalculated with another formula.

Thermal balance of the crawl space in summer and the depth of thaw under a building. The thermal balance components of the crawl space are determined with the usual formulae of construction thermal engineering. The quantity of heat flowing to the crawl space through the floor and the quantity of heat flowing in or out of the crawl space through the perimeter wall with a standard type of crawl space are:

$$Q_1 = \frac{F_{fl}}{R_{fl}} (\vartheta_1 - \vartheta_c)\ t; \tag{6.11}$$

$$Q_2 = \frac{F_p}{R_p} (\vartheta_{oa} - \vartheta_c)\ t, \tag{6.12}$$

where F_{fl} - the area of the floor above a ventilated crawl space,
F_p - area of the perimeter wall of the building measured from the inner perimeter,
R_p - thermal resistance of the protective wall,
ϑ_{oa} - temperature of the outside air.

Denoting the temperature of freezing-melting of the ground moisture by ϑ_3, we obtain the quantity of heat going to thaw the soil under the building during the time interval t according to the formula (Chapter IV):

$$Q_4 = -80\omega F_{fl} \left[-h_o - \lambda_T R + \sqrt{(h_o + \lambda_T R)^2 + \lambda_T \frac{\vartheta_c - \vartheta_3}{40\omega} t} \right], \tag{6.13}$$

and in some regions having low soil temperatures where at the beginning of summer when the outside air temperature is above 0°C the temperature of the air in the crawl space is below 0°C,

$$Q_4' = 2\lambda_{fr} (\vartheta_o - \vartheta_c) \sqrt{\frac{t}{\pi a}}\ F_{fl}. \tag{6.14}$$

The quantity of heat that is eliminated or introduced by some ventilating system (in winter or summer) is determined by the formula

$$Q_5 = 864 (\vartheta_{oa} - \vartheta_c)\ Mt, \tag{6.15}$$

where M is the amount of air moved by the ventilating system (kg/sec).

For the period with above 0°C mean monthly air temperatures when the ground is thawing the heat balance of the ventilated crawl space has the form

$$Q_1 + Q_2 + Q_4 + Q_5 = 0. \tag{6.16}$$

In regions with low soil temperatures at the beginning of the summer period all of the heat flowing through the floor and that introduced by the outside air ventilating the crawl space is consumed only on increasing the soil temperature which remains in the frozen state. Then the heat balance equation has the form.

$$Q_1 + Q_2 + Q_4' + Q_5 = 0. \tag{6.17}$$

The depth of thaw of the foundation soil under the building for any calculation time interval t is determined by the formula

$$h = -\lambda_T R + \sqrt{(\lambda_T R + h_o)^2 + \lambda_T \frac{\vartheta_c - \vartheta_3}{40\omega} t + \lambda_{fr} \frac{\vartheta_o - \vartheta_3}{40\omega \sqrt{\pi a}} (\sqrt{t_o + t} - \sqrt{t_o})}, \tag{6.18}$$

where t_o - the time from the beginning of thawing to the beginning of the calculation time interval t and corresponding to a depth of thaw of h_o.

Determination of air temperature in the crawl space during winter. The thermal regime in the crawl space during winter in each particular case should be such as to freeze the layer under the building thawed during the summer with a minimum heat loss from the building itself.

Thus the highest rate of ventilation of the crawl space and the consequently highest rate of freezing of the thawed layer under the building should be assigned to the initial period of stable below freezing air temperatures. To reduce heat losses from the building when the outside air temperature is relatively low, the ventilating system should be either completely closed down (openings in the perimeter wall covered, etc.) if calculations show that the ground under the building will not thaw, or ventilation reduced to a minimum that will maintain a temperature of minus 2 - 3°C in the crawl space.

The number of below 0 degree-hours $\Sigma(\vartheta_c - \vartheta_3)t$, required to freeze the layer of soil thawed during the summer is determined by the formula

$$-\sum(\vartheta_c - \vartheta_3)\ t = \frac{40\omega}{\lambda_{fr}} (2h\lambda_{fr}R + h^2). \tag{6.19}$$

Comparing the value of (6.19) with data on the temperature of the outside air during the winter, taking into account the considerations given above makes it possible to establish an effective time period for cooling the crawl space and the mean air temperature in the crawl space for this time interval.

For regions with a severe temperature regime, the air temperature in the crawl space determined in this way will be somewhat too low because equation (6.19) does not take into account freezing of the thawed layer from below due to heat losses to the permafrost. Nevertheless the error in determining the air temperature in the crawl space has no influence on the accuracy of the subsequent calculations and simply allows some reduction in the duration of the intensive ventilation of the crawl space.

On the basis of general considerations concerning the choice of air temperature in the crawl space given above, the most favourable time can be established for freezing the ground and correspondingly the value of the air temperature in the crawl space can be refined.

When the ground begins to freeze there is intensive heat liberation from the ground into the crawl space. Therefore during the first months of freezing, if the calculation is carried out monthly, a higher air temperature for the crawl space should be used than that used in subsequent months. Otherwise extremely large ventilating devices would have to be used (large openings in the perimeter wall, air ducts and deflectors, etc.).

Thermal balance of the crawl space in winter and freezing of the ground under the building. The thermal balance of the crawl space in winter includes the heat flows Q_3 and Q_6 in addition to the above-mentioned components Q_1, Q_2, Q_4, Q_4' and Q_5.

The quantity of heat flowing into the crawl space from the freezing ground for any given time interval of freezing t is found by the formula:

$$Q_3 = 80\omega F_{fl} \left[\sqrt{(\lambda_{fr}R + h_o')^2 - \lambda_{fr} \frac{\vartheta_c - \vartheta_3}{40\omega} t} - \lambda_{fr}R - h_o'\right], \qquad (6.20)$$

where h_o' - depth of freezing of the foundation soil from above during the time interval preceeding time t.

After complete freezing of the thawed layer around the building, the transfer of heat into the atmosphere through the frozen ground Q_6 begins.

The quantity of this heat is calculated by the formula:

$$Q_6 = \lambda_{fr} (\vartheta_{ng} - \vartheta_{pg}) \left\{ 4\sqrt{\frac{t}{\pi a}} (dl + sb - 2sd) - \frac{2(d+s)}{\pi} t \left[3.576 + \ln \frac{(l - s)^2}{4at}\right]\right\} + \frac{3\lambda_{fr} sd}{\sqrt{\pi a}} (\vartheta_o - \vartheta_3)(\sqrt{t_p + t} - \sqrt{t_p}), \qquad (6.21)$$

where t_p - time period from the beginning of thawing to the moment of complete freezing of the thawed layer,

l - half the length of the building,

b - half the width of the building,

s - distance from the centre of the building to the centre of the perimeter wall in the longitudinal direction, d - distance from the centre of the building to the middle of the perimeter wall in the transverse direction,

ϑ_{ng} - temperature of the ground surface outside the crawl space,

ϑ_{pg} - temperature of the ground surface in the crawl space.

The thermal balance of the crawl space at the beginning of freezing of the thawed layer has the form:

$$Q_1 + Q_2 + Q_3 + Q_5 = 0. \tag{6.22}$$

Freezing of the layer of soil at the base of the building thawed during the summer, taking into account freezing from above as well as from below due to heat losses into the permafrost for any time interval t, is determined by the formula:

$$h = -\lambda_{fr}R + \sqrt{(\lambda_{fr}R + h_o')^2 - \lambda_{fr}\frac{\vartheta_c - \vartheta_3}{40\omega}t} -$$

$$- \lambda_{fr}\frac{\vartheta_o - \vartheta_3}{40\omega\sqrt{\pi a}}\left(\sqrt{t_o + t} - \sqrt{t_o}\right), \tag{6.23}$$

where t_o - time from the beginning of thawing up to the calculation time.

In formula (6.23) the first two components express the value of freezing of the thawed layer in the crawl space from above and the last component expressed freezing from below.

After freezing of the seasonally thawed layer near the structure and its base, the thermal balance takes on the form:

$$Q_1 + Q_2 + Q_5 + Q_6 = 0. \tag{6.24}$$

During this time ventilation can be substantially decreased or even stopped if the following relation is satisfied:

$$Q_1 \leqslant | Q_2 + Q_6 | . \tag{6.25}$$

For a rough consideration of heat losses Q_6, Table XXIX can be used in which the maximum dimensions of the heated building (in metres) are given under which in specific circumstances the frozen state of the foundation soil can be preserved without ventilation and even without a crawl space exclusively because of heat flow through the frozen soil. As introductory values in Table XXIX we have:

mean temperature of the ground surface (in practive the mean temperature of the outside air) for the four coldest months of the winter - ϑ_{ng};

ground temperature at the depth of zero amplitude - ϑ_o
temperature in the building ϑ_1,
thermal resistance of the floor R_{fl}.

Experience has shown that air exchange in open crawl spaces and crawl spaces ventilated by openings in the perimeter wall is due mainly to wind pressure. In ventilated crawl spaces, short term increases or decreases in temperature and the quantity of air exchanged due to temporary low or short-term strong winds lasting for several hours or even days, does not have any influence on the thermal regime of the crawl space averaged over a relatively long period of time. In calculating air exchange this makes it possible to use average air temperatures in the crawl space and also average wind velocity and average outside air temperatures.

In calculating the ventilation of the crawl space, two types of problems may be encountered:

1) With respect to the calculated mean air exchange M (resulting from thermal engineering computation) during the calculation time interval t and averaged meterological data, the required area of the ventilation openings has to be determined;

2) from the given area of the openings and mean meterological data the mean air exchange M for the calculation time interval has to be determined.

These two modifications of the problem do not differ in principle.

The calculations are based on the connection between the area of the ventilating openings F and the quantity of air M passing through them in a unit of time, expressed by the equation

$$F = \frac{M}{\mu v \sum\limits_{i=1}^{i=m} \sqrt{k_i - z}}, \tag{6.26}$$

where μ - coefficient of air flux through the opening 0.65 - 0.70,
v - mean velocity of air flow (wind) for the calculation time interval in m/sec,
k - aerodynamic coefficient showing what part of the kinetic energy of the wind is transformed into potential pressure energy,
k_i - aerodynamic coefficient near the i-th opening functioning as an air inlet taken at 0.4 to 0.8,
k_n - aerodynamic coefficient near the n-th opening functioning as a draft taken at -0.4 to -0.6,
m, p - the number of openings functioning as inlets and outlets, respectively,
z - a dimensionless coefficient determined on the assumption that the

area of all openings is equal, from the expression:

$$\sum_{i=1}^{i=m} \sqrt{k_i - z} = \sum_{n=1}^{n=p} \sqrt{-k_n + z}. \qquad (6.27)$$

If it is desirable to refine the calculations, one should use tables of aerodynamic coefficients (Baturin and Kucheruk, 1937).

Figure 42 shows diagrams of wind pressures on buildings corresponding to three basic wind directions indicated by solid line arrows. Winds blowing from the direction of the dashed-line arrows at different velocity create the same air exchange as winds from the three basic directions.

Thus the effect of winds from all eight points of the compass can be reduced to three calculation cases.

Having taken a given number of openings and established from the diagrams the approximate aerodynamic coefficients of the direction of air flow, we determine graphically or by selection for each calculation case a supplementary dimensionless value z according to equation (6.27) which does not depend on wind velocity.

Let us assume that we are solving a problem of the first type, i.e. we are trying to find the area of openings functioning as inlets proceeding from a given air exchange M.

If during the calculation time interval the direction of the wind remains constant for the corresponding values of k_i and z, the area of the openings functioning as inlets could be determined from equation (6.26).

If the direction of the wind changes the denominator in the fraction of equation (6.26) will be $\frac{\mu v}{100}[n_1 f_1 + n_2 f_2 + n_3 f_3 + \ldots]$ instead of

$$\mu v \sum_{i=1}^{i=m} \sqrt{k_i - z},$$

where n_1, n_2, n_3 ... express in percent the repetition of winds of a given direction during the calculation time period and the values f_1, f_2, f_3 ... respectively represent

$$\sum_{i=1}^{i=m} \sqrt{k_{i_1} - z_1};$$

$$\sum_{i=1}^{i=m} \sqrt{k_{1_2} - z_2}$$

etc., and k_{1_1}, k_{1_2}, k_{1_3} denote the aerodynamic coefficients, and z_1, z_2, z_3 ... are dimensionless coefficients obtained from equation (6.27) for a specific wind direction. Then equation (6.26) can be written in the form

$$F = \frac{100\ M}{\mu v \Sigma nf} = \frac{100\ M}{\mu v [\, n_1 f_1 + n_2 f_2 + n_3 f_3 + \dots]},$$

or

$$F\,\frac{n_1 f_1 \mu v}{100} + F\,\frac{n_2 f_2 \mu v}{100} + F\,\frac{n_3 f_3 \mu v}{100} + \dots = M. \qquad (6.26a)$$

Each component of the left-hand part of this equation expresses that average quantity of air which under specific conditions passes through the ventilation openings with the appropriate wind direction.

An example is given below of a thermal engineering calculation of a ventilated crawl space applicable to the mechanism given above.

Example. It is required to design a ventilated crawl space which would ensure preservation of the foundation soil of a heated building in the frozen state based on the following data:

Basic dimension of the crawl space in plan view; l = 10 m, s = 9.5 m, b = 5 m, d = 4.5 m (Fig. 43).

The thermal resistance of the floor R_{fl} 3.0 m² hour x degree/kcal.

Thermal resistance of the perimeter wall R_p 1.6 m² hours x degree/kcal.

Thickness of the perimeter wall is equal to 1 metre, the height of the crawl space - 0.45 metres.

The temperature of the building at floor level: in winter $\vartheta_1 = 16°C$, in summer when $\vartheta_{oa} \leq 16°C$, $\vartheta_1 = 16°C$, when $\vartheta_{oa} > 16°C$, $\vartheta_1 = \vartheta_{oa}$.

The heat emission coefficient for the ground surface when the foundation soil is melting is

$$\alpha_n = 10 \text{ kcal/m}^2 \times \text{hour} \times \text{degree}.$$

The coefficient of heat emission of the ground surface during freezing of the foundation soil is

$$\alpha_n = 15 \text{ kcal/m}^2 \times \text{hour} \times \text{degree}.$$

The thermal physical indices of the soil are: λ_T = 1.2 kcal/m x hour x degree, λ_{fr} = 1.8 kcal/m x hour x degree, a_n = 0.004 m²/hour, w = 200 kg/m³, ϑ_o = -6°C, ϑ_3 = -0.5°C.

Calculation. The air temperature in the crawl space determined by formula (6.8) is: in May ϑ_c = -1.6°C, in June +7.2°C, in July +11.7°C, August +9.8°C, in September +2.5°C.

Substituting in the heat balance equation (6.16) $Q_1 + Q_2 + Q_4' + Q_5 = 0$, the numerical value of Q_1, Q_2, Q_4', Q_5 from formulae (6.11), (6.12), (6.14) and (6.15) we obtain for May

$$\frac{18\cdot 8}{3}(16+1{,}6)\cdot 720+\frac{52\cdot 0{,}45}{1{,}6}\cdot(5{,}6+1{,}6)\,720+$$
$$+2\cdot 1{,}8\cdot(-6+1{,}6)\cdot\sqrt{\frac{720}{3{,}14\cdot 0{,}004}}\cdot 18\cdot 8+864\,(5{,}6+1{,}6)\cdot 720\ M=0,$$

hence M = 0.0295 kg/sec.

The minus sign standing before the value of air exchange M indicates that formula (6.8) is not applicable to the determination of air temperature in the crawl space and a recalculation should be made with formula (6.9) which gives

$$\vartheta_c=\frac{16\sqrt{3{,}14\cdot 0{,}004\cdot 720}+2\cdot 1{,}8\cdot 3\cdot(-6)}{\sqrt{3{,}14\cdot 0{,}004\cdot 720}+2\cdot 1{,}8\cdot 3}+0{,}5=-0{,}7°;$$

thus in May there is no thawing in the crawl space.

Constructing a new equation for the thermal balance for May we obtain

$$\frac{18\cdot 8}{3}(16+0{,}7)\cdot 720+\frac{52\cdot 0{,}45}{1{,}6}(5{,}6+0{,}7)\,720+$$
$$+2\cdot 1{,}8\,(-6+0{,}7)\cdot 18\cdot 8\sqrt{\frac{720}{3{,}14\cdot 0{,}004}}+864\cdot(5{,}6+0{,}7)\cdot 720\ M=0,$$

hence M = 0.0032 kg/sec = 0.0025 m²/sec.

For June (ϑ_c = 7.2°C, t_o = 720 hours, h_o = 0) the depth of thaw is determined by formula (6.18) assuming R = 1: a_n:

$$h=-1{,}2\cdot 0{,}1+\sqrt{(1{,}2\cdot 0{,}1+0)^2+1{,}2\frac{7{,}2+0{,}5}{40\cdot 200}\,720+}$$
$$\overline{+1{,}8\frac{-6+0{,}5}{40\cdot 200\cdot\sqrt{3{,}14\cdot 0{,}004}}\left(\sqrt{720+720}-\sqrt{720}\right)}=0{,}68\ \text{м}.$$

After constructing the equation for the thermal balance of the crawl space for June from formula (6.16) and substituting the numerical values in formulae (6.11), (6.12), (6.13) and (6.14) we obtain:

$$\frac{18\cdot 8}{3}(16-7{,}2)\cdot 720+\frac{52\cdot 0{,}45}{1{,}6}(15{,}5-7{,}2)\cdot 720-80\cdot 200\cdot 8\cdot 18\times$$
$$\times\left(\sqrt{(0+1{,}2\cdot 0{,}1)^2+1{,}2\frac{7{,}2+0{,}5}{40\cdot 200}\cdot 720}-\right.$$
$$\left.-0{,}12-0\right)+864(15{,}5-7{,}2)\cdot 720\ M=0,$$

hence M = 0.282 kg/sec = 0.229 m^3/sec.

Similarly we obtain:

in July (ϑ_c = 11.7°C, h_o = 0.68 m, t_o = 1440 hours):
h = 1.19 m, M = 0.236 kg/sec = 0.196 m^3/sec;

in August (ϑ_c = 9.8°C, h_o = 1.19 m, t_o = 2160 hours):
h = 1.48 m, M = 0.193 kg/sec = 0.158 m^3/sec;

in September (ϑ_c = 2.5°C, h_o = 1.48 m, t_o = 2880 hours):
h = 1.59 m, M = -0.123 kg/sec = -0.096 m^3/sec.

The negative value of M indicates that in September the air temperature in the crawl space should be recalculated with formula (6.10)

$$m=16+(1{,}2\cdot 0{,}1+1{,}48+1{,}2\cdot 3)\frac{80\cdot 200\cdot 3}{720}=363^\circ$$

$$\vartheta_c=363-\sqrt{363^2-\left(363-\frac{80\cdot 1{,}2\cdot 3^2\cdot 200}{720}\right)^2+\left[(1{,}2\cdot 0{,}1+1{,}48)\times\frac{80\cdot 200\cdot 3}{720}\right]^2}+$$
$$+0{,}5=363-358{,}5+0{,}5=5^\circ.$$

For a crawl space temperature of ϑ_c = 5°C we obtain m = 0.05 kg/sec = 0.039 m^3/sec.

The total depth of thaw in September, i.e. for the entire summer period, is h = 1.59 m; the number of degrees centigrade below freezing required to freeze the layer of soil thawed during the summer is found from equation (6.19)

$$-\Sigma(\vartheta_c-\vartheta_3)\,t=\frac{40\cdot 200}{1{,}8}\left(2\cdot 1{,}59\cdot 1{,}8\cdot\frac{1}{15}+1{,}59^2\right)=12\ 950\ \text{deg. hour.}$$

Taking the calculation time interval t = 720 hours we obtain:

$$\sum(\vartheta_c+0{,}5)=-\frac{12\,950}{720}=-18^\circ.$$

Comparing this value with the total of the winter average monthly below freezing temperatures $\Sigma\vartheta_{oa}$ = -184°C, we see that the soil can be slightly frozen. Taking the time of thawing to be 3 months t = 2160 hours, and, consequently, t_o = 3600 hours, the refined value of ϑ_c according to formula (6.23)

$$1{,}59 = -1{,}8\cdot\frac{1}{15}+\sqrt{\left(1{,}8\cdot\frac{1}{15}\right)^2-1{,}8\,\frac{\Sigma(\vartheta_c+0{,}5)\cdot 720}{40\cdot 200}-}$$
$$\overline{-1{,}8\,\frac{-6{,}0+0{,}5}{40\cdot 200\cdot\sqrt{3{,}14\cdot 0{,}004}}\cdot(\sqrt{3600+2160}-\sqrt{3600})},$$

hence

$$\sum(\vartheta_c+0{,}5) = -14{,}5^\circ \text{ and } \sum\vartheta_c = \sum(\vartheta_c-\vartheta_з)+n\vartheta_з = -14{,}5-3\cdot 0{,}5 = -16^\circ,$$

where n - the number of months during which the ground is frozen.

To obtain a total of -16°C for 3 months we assume the following temperatures: in October ϑ_c = -1°C, in November ϑ_c = -8°C, in December ϑ_c = -7°C. Then in October h_o = 0, ϑ_c = -1°C, t_o = 3600. The depth of freezing (6.23) is

$$h=-1{,}8\cdot\frac{1}{15}+\sqrt{\left(1{,}8\cdot\frac{1}{15}\right)^2-1{,}8\,\frac{-1+0{,}5}{40\cdot 200}720-}$$
$$\overline{-1{,}8\,\frac{-6+0{,}5}{40\cdot 200\sqrt{3{,}14\cdot 0{,}004}}(\sqrt{3600+720}-\sqrt{3600})} = 0{,}19+0{,}06=0{,}25 \text{ м}.$$

The heat balance equation for October in correspondance with equations (6.11), (6.12), (6.20) and (6.15):

$$\frac{18\cdot 8}{3}\cdot(16+1)\cdot 720+\frac{52\cdot 0{,}45}{1{,}6}\cdot(-7{,}9+1)\cdot 720+200\cdot 80\cdot 18\cdot 8\times$$
$$\times\left(\sqrt{\left(0+\frac{1{,}8}{15}\right)^2-1{,}8\cdot\frac{-1+0{,}5}{40\cdot 200}\cdot 720}-1{,}8\cdot\frac{1}{15}-0\right)+$$
$$+864(-7{,}9+1)720\cdot M=0,$$

hence M = 0.240 kg/sec or 0.18 m^3/sec.

Similarly we obtain:

in November (ϑ_c = -8°C, h_o = 0.19, t_o = 4320):

h = 1.14, M = 0.238 kg/sec = 0.173 m^3/sec;

in December (ϑ_c = -7°C, h_o = 1.02 m, t_o = 5040 hours);

h = 1.59 m, M = 0.06 kg/sec = 0.041 m^3/sec, i.e. by January the soil will be completely frozen. Under the condition $Q_1 \leqslant |Q_2 + Q_6|$ the openings in the perimeter wall can be closed for January through March when the mean temperature of the outside air is

$$\vartheta_{oa} = \frac{-43.2-35.6-22.4}{3} = -33.7^\circ.$$

The mean ground surface temperature outside the crawl space during this time is

$$\vartheta_{ng} = \frac{-42-36-23}{3} = -33.8^\circ.$$

The ground surface temperature in the crawl space as a safety factor is taken to be

$$\vartheta_{ng} = \vartheta_3 = -0.5°.$$

Then according to formula (6.11)

$$Q_1 = \frac{16 + 0.5}{3} \cdot 8 \cdot 18 \cdot 2160 = 1710{,}000 \text{ kcal};$$

according to formula (6.12)

$$Q_2 = \frac{-33.7 + 0.5}{1.6} \cdot 52 \cdot 0.45 \cdot 2160 = -1{,}080{,}000 \text{ kcal};$$

according to formula (6.21)

$$Q_6 = 1{,}8(-33{,}8 + 0{,}5)\left\{4\sqrt{\frac{2160}{3{,}14 \cdot 0{,}004}} \times (4{,}5 \cdot 10 + 9{,}5 \times\right.$$

$$\times 5 - 2 \cdot 9{,}5 \cdot 4{,}5) - \frac{2(4{,}5 + 9{,}5)}{3{,}14} 2160 \times \left.\left[3{,}576 + \ln\frac{(10 - 9{,}5)^2}{4 \cdot 0{,}004 \cdot 2160}\right]\right\} +$$

$$+ \frac{8 \cdot 1{,}8 \cdot 9{,}5 \cdot 4{,}5}{\sqrt{3{,}14 \cdot 0{,}004}}(-6{,}0 + 0{,}5) \cdot (\sqrt{5760 + 2160} - \sqrt{5760}) = -2\,656\,000 \text{ kcal}$$

$$|1\,080\,000 + 2\,656\,000| > 1\,710\,000,$$

i.e. condition (6.25) is fulfilled and with the openings closed there is no thawing under the structure.

Determining the area of the openings. The calculation of the ventilated crawl space consists in determining the area of the ventilated openings based on the air exchange M calculated from the heat balance equation.

For buildings having the plan shape of an extended rectangle it is advisable to locate the openings along the long sides of the building. Taking into account the dimensions of the plan of the building, let us locate 5 openings on each side.

The effect of winds from any direction is reduced to three calculation diagrams (Fig. 44). After determining the aeordynamic coefficient from diagram 1 and substituting them in equation (6.27) we obtain: $3 \cdot \sqrt{0.7 - z} + 2\sqrt{0.4 - z} = 5 \cdot \sqrt{0.35 + z}$, from which we determine the value of z = 0.11 and

$$\sum_{i=1}^{i=m} \sqrt{k_{i_1} - z_1} = 3 \cdot \sqrt{0{,}7 - 0{,}11} + 2 \cdot \sqrt{0{,}4 - 0{,}11} = 3{,}38.$$

Analogously for diagram 2 from the same equation

$$2\sqrt{0{,}6 - z} + \sqrt{0{,}4 - z} + 2\sqrt{0{,}2 - z} = 2\sqrt{0{,}45 + z} + 3\sqrt{0{,}5 + z}$$

we obtain z = -0.05 and

$$\sum_{i=1}^{i=m} \sqrt{k_{i_2} - z_2} = 3{,}28.$$

In diagram 3 the flux of air into the crawl space is due to differences in negative pressures along the length of the building. The openings for which the relation $|k_n| < |z|$ is fulfilled, function as inlets; their number is taken approximately and refined during the calculation. Let us assume that four openings operate as inlets, then

$$2\sqrt{-0.1-z}+2\sqrt{-0.2-z}=2\sqrt{0.4+z}+4\sqrt{0.7+z}.$$

It turns out that $|z| > 0.4$ and consequently the opening for which $|k_n| = 0.4$ also functions as an inlet.

Then

$$2\sqrt{-0.1-z}+2\sqrt{-0.2-z}+2\sqrt{-0.4-z}=4\sqrt{0.7+z},$$

hence

$$z=-0.41 \text{ and } \sum_{i=1}^{i=m}\sqrt{k_{i_3}-z_3}=2.125$$

Taking into account the frequency of winds from various directions, we determine the area of a single opening using equation (6.26a), assuming that for November the frequency of winds corresponding to diagram 1 is $n_1 = 34.9\%$, corresponding to diagram 2 $n_2 = 44.5\%$ and corresponding to diagram 3 $n_3 = 20.6\%$ and the required $M = 0.173$ m^3/sec.

$$F=\frac{100\cdot 0.173}{0.7\cdot 1.5}\left[\frac{1}{34.9\cdot 3.38+44.5\cdot 3.28+20.67\cdot 2.125}\right]=0.0535 \text{ м}^2.$$

Openings with area $F = 0.0535$ m^2 can be readily constructed in the perimeter wall of the building. If F turns out to very large the calculation should be repeated with a larger number of openings.

Such a calculation is carried out for each calculation time interval.

Similarly, with respect to a given area of the opening F and values of z one can calculate air exchange M.

The calculation time interval t varies with variation in meteorological factors. If during the course of several months these factors change little, the wind velocity and air temperature for these months should be averaged and used for the calculation time interval.

The calculation can make provision for changing the dimension of the opening for different periods of the year; for example, during a very cold winter the area may be reduced to cut heat losses from a building and during a very hot summer to decrease the depth of thaw, etc.

In some cases for convenience, the area of the opening can be left constant. If the soil has a high moisture content, and if there is a uniform variation in the mean monthly wind velocity and air temperature during the winter or summer period, decreasing or increasing the area of the openings during some months even by 40- 50% has little influence on the depth of thaw.

The above calculation plan extends also to the construction of open crawl spaces where the perimeter of the crawl space is divided into separate equal portions within which the aerodynamic coefficients can be assumed constant. Furthermore, as for crawl space ventilated by openings in the perimeter wall, a determination is made either of the area of the ventilation opening (in this case the height of the load bearing beam above the ground surface) or air exchange.

For a rough determination of the area of ventilation openings one can use Table XXXI in which are given ventilation moduli, i.e. the ratio of the total area of the openings to the area of the floor above the crawl space for the appropriate permafrost zones of the map by I.Ya. Baranov (see appendix). In addition the table includes as initial values the width of the building in metres, the thermal resistance of the floor of the first storey and the temperature inside the heated building.

2. Static Calculations

In considering the essential point and the methodology of static calculations relating to the design of foundations with preservation of the foundation soils in the frozen state, we deal only with features connected specifically with the given conditions and construction method, which are supplemental to the generally accepted features.

As noted above, the equilibrium of external forces acting on a foundation under the conditions considered is expressed by the equation

$$N = N_r + T_s + T_h.$$

T_s can have a positive or negative value depending on the action of adfreezing tangential forces between the foundation and the permafrost being directed upwards or downwards.

The strength of the bearing part is calculated for the most unfavourable condition existing during the warm part of the year when there is no heaving, i.e. when $T_s = 0$.

Then the load N is distributed between the reactions: N_r - normal to the footing, and T_s - tangential to the lateral surface of the foundation.

The permissable maximum pressures on perennially frozen soil are given in Chapter III.

Since the bearing capacity of frozen soil decreases with an increase in temperature, the highest temperature of the soil at the depth of the foundation is taken for the calculation. Thus, if the foundation is laid at a depth of 3 - 6 m from the ground surface the highest temperature of the perennially frozen soil under the footing is observed in the fall or at the beginning of winter.

Depending on the permafrost conditions at the building site, two cases should be differentiated:

a) when the mean annual temperature of the permafrost is close to 0°C and

b) when it is substantially below 0 (for example -3°C and below).

In the first case, increasing the depth at which the foundation is placed does not provide for a reduction in the calculation temperature of the soil at the level of the footing. It is therefore advisable to strive towards reducing the depth of the foundation designing it for stability with respect to heaving (see Section 7) and according to local experience. According to current standards (N and TU 118-54), the least depth of laying a foundation below the calculated horizon of summer thawing, i.e. taking to account the change in depth of summer thawing of the soil under the structure, are as follows:

for foundations of masonry and reinforced concrete structures - 0.75 - 1 m,

for foundations of wooden structures - 0.5 - 0.75 m.

In this case the bearing capacity of the frozen foundation soil should be calculated for a temperature close to 0°C (-0.1; -0.2°C).

In the second case the calculation, i.e. the highest temperature of the perennially frozen soil decreases noticeably with depth; correspondingly there is an increase in the long-term strength of the frozen soil and in the adfreezing tangential forces. In this case, questions concerning the depth of laying the foundation and the value of the permissible pressure on the soil n_{per} as well as the tangential pressure τ are closely interconnected. As the depth at which the foundation is laid increases, the length of the columns and the depth of the trench increases, but there is a reduction in the size of the load-bearing part and in the size of the trench since there is a substantial increase in the bearing capacity of the soil. When the foundation is laid at greater depth in the frozen soil, 3 m and more, one can get by without footings and use posts only. The problem of establishing the best depth for laying the foundation or driving piles is regional, and in erecting very large structures in the Far North foundations that are laid deep in the permafrost can be cheaper than shallow foundations.

In other respects the static calculation of foundations erected with the consideration of preserving the foundation soil in the frozen state do not differ from the usual practices.

5. Bases and Foundations of Structures Designed for Differential Settlement during Gradual Thawing of the Foundation Soils

In the literature on engineering geocryology and in building practice, this method of providing for stability in the normal performance of a structure is called "konstruktivnyi" (structure designed for settlement). The essential point of this method is that during the operation of the structure the foundation soil gradually thaws and compacts, and, owing to special measures taken in the design, the structure adapts itself to the displacement (settlement) of the foundation soil and performs the task for which it was designed. This method is intended mainly for construction in the southern portion of the permafrost region and for other cases when preservation of the permafrost requires complex and expensive measures.

In the erection of buildings with gradual thawing of the foundation soils while the building is in operation, the foundation initially lies on frozen soil which only after some time, usually not before one or two years after completion of the structure, begins to thaw.

As the soil thaws it is compacted and there is a gradual, usually differential, settlement of the structure. The settlement regime of the structure is connected with the thawing regime of the foundation soils and settlement of the structure to some extent can be controlled by changing the rate of thaw.

An advantage of this method is the elimination of the necessity of additional expenditure of thawing the foundation soil. At the same time designing becomes much more complex and the bearing components of the structure become more expensive. The use of this construction method is usually limited by the ice content of the frozen soil; when there is a large quantity of ice in the soil the cost of construction using the method of adapting the structure to differential settlement increases substantially.

Design features. In designing structures where the foundation soil is expected to thaw under them the following specifications should be maintained:

a) reduction in additional forces in parts of the structure arising from differential settlement of the foundation soil;

b) provision for resistance of the structure to these additional forces;

c) obtaining uniform settlement of the structure as a whole and of its separate units bounded by settlement joints and also of individual units of heavy equipment.

The reduction in additional forces in parts of the structure is achieved by dividing the structure into separate units by settlement joints, initial thawing and to some extent controlling thawing of the foundation soil.

The task of the designer is to develop an effective diagram of the building, and to locate correctly the settlement joints and hinge couplings. Excessive use of joints and hinges can greatly decrease the additional forces in parts of the building but complicates the operation of equipment. Some indications of the subdivision of buildings can be expressed schematically in the following ways.

1. In designing the layout of a building and its division by joints one should calculate the depth of thaw of the soil under the building. The best thing is to represent the thawing surface graphically with contours.

2. As a rule when the foundation is placed at a normal depth, the interior walls of the building rigidly connected to the exterior walls can transfer their load only to the outer wall. It is recommended that the interior main walls and their foundations be separated from the exterior walls by joints; secondary interior walls should be as light as possible and supported on load-bearing beams attached to the foundation of the outer wall.

3. The subdivision of a building by joints should not decrease the stability of individual units. In particular, one should not permit the design of walls which are not tied in with other walls or at least they should be constructed with buttresses. When a wall is divided by joints resulting in an unattached structure in the plan view, the wall sections should be connected with specially constructed rigid beams.

Resistance of a structure to additional forces, resulting from differential settlement of the foundation soil is provided for by the following measures.

1. Sufficiently large though shallow foundations of the type described below.

2. The tie-in of the foundation with the superstructure into a single three-dimentional system bounded by the settlement joint.

3. The construction of rigid reinforced concrete floors between the storeys.

4. The inclusion in brick walls and rubble stone foundations of reinforced concrete interlayers. This measure is sufficient when the foundation soils are sufficiently favourable and there is relatively little differential settlement of the foundation.

Some supplementary remarks follow:

a) for constructing the floors of the first storey it is desirable to use material of low thermal conductivity to decrease the rate of thawing of the soil; floors laid directly on the ground should be made of slag concrete, foam concrete; etc.; the construction of such a floor should at the same time be as cheap as possible, since after the building has been put to use it will be necessary to renew the floor two or three times as the foundation soil settles.

b) Foundations should be designed in such a way that there is no rotating moment in the cross-section of the foundation. The footings of the foundations of external walls should be extended in the direction of greater depth of thaw, i.e. towards the centre of a heated building, as shown in Fig. 45.

c) In places where plastic deformation of the foundation soil may occur, the bearing part of the foundation should be designed for the action of friction forces between the foundation footing and the underlying soil being squeezed out (Fig. 46).

When excessive tilting occurs as the result of differential thawing of the foundation soil under individual parts of the building, some thought should be given to the possibility of reestablishing the appropriate level by special heating of those places where there would otherwise be insufficient thawing of the foundation soil. For this purpose when the foundation is being laid, pipes can be installed at appropriate places near the footing, along which hot water may be passed periodically. In the design a detailed diagram should be worked out for the distribution of valves and taps permitting selective heating of specific parts of the soil under the foundation. Because of the novelty of this type of construction, if the above measures turn out to be insufficient, it is possible to install heavy mechanical equipment on regulating devices of the type used on loess in Krivorozh'e; in installing underground pipes one then has to use flexible hoses, etc., for pumping liquids.

In the present paper no design solution is given for the listed difficulties but it should be an object of attention for engineers carrying out building construction with adaptation of the structure to thawing of the foundation soil.

<u>Types of foundations used in construction with adaptation of the structure to thawing of the foundation soil</u>. The basic characteristic of the performance of foundations beneath which the soil begins to thaw after the construction is completed is the necessity of counteracting a complex combination of various external forces in the form of bending, twisting and shearing. Bending moments and shear forces in this case reach values which exceed the moments and forces to which foundations are subjected when built on ordinary thawed soil.

This unique performance of foundations predetermines the choice of the most suitable type of foundation for the given conditions. Foundations using individual supports connected along the top by a load-bearing beam must be rejected since individual supports cannot counteract bending forces and all forces in the form of moments and transverse forces are applied to the beam.

The most suitable types of foundations in this case are strip foundations, systems of intersecting strips and slabs. The basic material apparently should be precast reinforced concrete which does not exclude the use of wood concrete (woodcrete) and wood. Since the theoretical basis for this method of construction was developed by the Permafrost Institute only recently (Saltykov, 1952b, 1953), most of the examples cited below are in fact diagrams illustrating possible methods of construction and only partly refer to structures actually erected.

A foundation wall can be used for supporting a wall and also rows of columns. The thickness of the wall varies depending on the value of forces applied. Figure 47a and b shows a rubble stone foundation with a single strip of reinforced concrete which is satisfactory for an average settlement of a building of 40 - 50 mm during the first year after construction; Fig. 47c shows a similar foundation with two strips of reinforced concrete. A building having one reinforced concrete strip in the foundation and one under the windows satisfactorily withstood settlement of 50 - 70 mm in a year.

For more unfavourable permafrost conditions and thawing conditions one can use a larger strip foundation of the frame type used in Vorkuta (Fig. 48).

Foundations of the slab type are used for structures occupying a relatively small area and at the same time are under complex conditions, i.e. when there is deep differential thawing of the foundation soil and when the ice content of the soil is high.

Experience has shown that a reinforced slab 8 x 8 m in area and 75 cm thick under a brick water tower 20 m high successfully withstood extensive tilting when the foundation soil thawed without damage to the slab or to the tower. One can think that in this case, as in the construction shown in Fig. 48, there is an excess safety factor which could be eliminated by more careful calculation.

Foundations in the form of intersecting strips can be used for unfavourable soil conditions and for structures with a complex configuration of the foundation where on thawing of the foundation soil high three-dimensional rigidity of the foundation must be provided. With a building covering a large area a lattice of intersecting strips can give substantial economy with respect to reinforced concrete as compared with a solid slab.

The rigidity and resistance of a building to bending forces on thawing of the frozen foundation soil can be even more greatly increased by constructing the foundation and frame of the building in the form of a single three-dimensional rigid system.

The depth of placing a foundation where the foundation soil is expected to thaw depends on two factors: on the depth of seasonal freezing and on the ice content of the frozen soil that is expected to thaw. The dependence of the depth of the foundation on the depth of seasonal freezing is established in the same way as in regions where permafrost is absent.

It is desirable to design the footings of foundations below the level of soils containing large quantities of ice which are subject to substantial thaw settlement. In most cases layers of soil with high ice content lie within 1 - 2 m of the permafrost table and with increased depth the ice content of the frozen soil decreases.

Construction on soil with a small amount of thaw settlement can be regarded as a particular case of construction with adaptation to thawing. As the thermal settlement of the soil decreases, determined by multiplying the depth of thaw by the value of the thawing coefficient, design measures and types of foundations approach those usually used in regions where there is no permafrost. The adaptation of a structure to a thawing foundation soil involves the laying of a reinforced concrete strip in the plane of the floors separating the storeys and in the foundation. Large foundations of the frame type, intersecting strips and slab types in this case are not necessary.

Calculations Required in Designing Bases and Foundations Erected with Adaptation of the Structure to Thaw Settlement of the Foundation Soil

The calculations in this section are based on the performance of foundations on a thawing foundation soil corresponding to diagram III of the location of foundations with respect to frozen soil (Fig. 33).

As in previous cases the calculations are divided into two categories: (a) thermal engineering calculation with which the depth and configuration of the thaw basin, which occurs under a structure during erection, are established, and (b) static calculations of the reaction of the thawing foundation soil on the footing of the foundation.

Thermal engineering calculations. A reliable solution of the depth, configuration and rate of development of the thaw basin can be obtained with the hydrointegrator of the system of Professor V.S. Luk'yanov. In addition to the hydrointegrator at present one can recommend the method developed by G.V. Prokhaev (Chap. IV). An example is given below of the numerical calculation of a thaw basin by this method.

Example. It is required to calculate the limiting configuration of the thaw basin under a heated building consisting of two rooms.

The building: dimensions of the rooms are shown in the cross-section (Fig. 49), the temperature inside the rooms $\vartheta_1 = 12°C$, $\vartheta_2 = 20°C$; the thermal resistance of the floor $R_1 = 1.5$ m^{2}hours · degrees/kcal, $R_2 = 2$ m^{2}hours · degrees/kcal; the temperature of the ground surface outside the building $\Theta_1 = -5°C$, $\Theta_2 = -3°C$; the heat conductivity coefficient of thawed and frozen soil $\lambda_T = 1.2$ kcal/m · hour · degree, $\lambda_{fr} = 1.6$ kcal/m · hour · degree; the temperature gradient within the layer having seasonal variations in temperature

$G = 0.02$ degrees/m; the freezing temperature of the soil $\vartheta_3 = 0°C$.

To calculate the configuration of the thaw basin we use equation (4.36); for this case

$$\vartheta\ (x, y) = \vartheta_3 = 0.$$

Making the notations in equation (4.36)

$$\frac{\lambda_{fr}}{\lambda_T} \Theta_1 f\ (\xi_1) = A_1; \tag{a}$$

$$\frac{\lambda_{fr}}{\lambda_T} \Theta_2 f\ (\xi_2) = A_2; \tag{b}$$

$$\vartheta_c F\ (X_1, Y) = B_1; \tag{c}$$

$$\vartheta_c F\ (X_2, Y) = B_2; \tag{d}$$

we reduce it to the form

$$\vartheta_3 - \frac{\lambda_{fr}}{\lambda_T} Gy = A_1 + A_2 - \sum_{i=1}^{n} (B_1 - B_2). \tag{e}$$

In the above expressions a, b, c, d, and e

$$f\ (\xi_1) = \frac{1}{2} - \frac{1}{\pi} \operatorname{arctg} \xi_1;\ \xi_1 = \frac{x}{y}; \tag{f}$$

$$f\ (\xi_2) = \frac{1}{2} - \frac{1}{\pi} \operatorname{arctg} \xi_2;\ \xi_2 = \frac{l_n - x}{y}; \tag{g}$$

$$X_1 = \frac{x - l_n}{\lambda_T R_{fl}}; \tag{h}$$

$$X_2 = \frac{x - l_{n-1}}{\lambda_T R_{fl}}; \tag{i}$$

$$Y = \frac{y}{\lambda_T R_{fl}}, \tag{j}$$

where x and y are coordinates and the index n is the order number of the room.

The depth of thaw for any value of x is determined graphically as the point of intersection of the lines

$$F_1 = \vartheta_3 - \frac{\lambda_{fr}}{\lambda_T} G \cdot y \tag{k}$$

and

$$F_2 = A_1 + A_2 - \sum_{i=1}^{i=n} (B_1 - B_2). \qquad (l)$$

According to formula (k) we determine:

$$F_1 = f\ (y) = 0 - \frac{1.6}{1.2}\, 0.02\ y = -0.026y,$$

then construct the line $F_1 = -0.266\ y$ on a supplementary graph (Fig. 49).

The function $F_2 = f\ (x,\ y)$ is constructed with respect to 2 - 3 points, i.e. for any value of x, 2 - 3 values of y have to be given as explained below.

Note that at the very beginning of the calculation it is exceedingly difficult at times to define the interface between thawed and frozen soil and to construct the points of intersection of the lines F_1 and F_2 one has to use 4 - 5 values of y.

Let us determine, for example, the depth of thaw under the centre of the building, i.e. for x = 9.

Let us arbitrarily take y = 13 m.

From nomogram 1 (Fig. 50) we determine:

$f\ (\xi_1) = f\ (\frac{9}{13}) = 0.35$ and further according to the nomogram in Fig. 51 or formula (a) for $\frac{\lambda_{fr}}{\lambda_T} = 1.33$ or $\Theta_1 = -5°C$ we find $A_1 = -2.35$.

Analogously for $f\ (\xi_2) = f\ (\frac{18-9}{18}) = 0.35$

$$\frac{\lambda_{fr}}{\lambda_T} = 1.33 \text{ and } \Theta_2 = -3°C \text{ we determine } A_2 = -1.4.$$

The values of B_1 and B_2 for each room are determined from the nomogram (Fig. 52). For the first room according to formulae (h, i, j) we determine:

$$X_1 = \frac{9 - 8}{1.2 \cdot 1.5} = 0.55;\ Y = \frac{18}{1.2 \cdot 1.5} = 10;\ \text{for } \vartheta_c = 12°;$$

from this nomogram we find $B_1 = 0.195$.

$$X_2 = \frac{9 - 0}{1.2 \cdot 1.5} = 5.0;\ Y = \frac{13}{1.2 \cdot 1.5} = 10 \text{ and } B_2 = 1.65.$$

For the second room analogously we find

$$X_1 = \frac{9 - 18}{1.2 \cdot 2} = -3.75;\ X_2 = \frac{9 - 8}{1.2 \cdot 2} = 0.416;\ Y = \frac{13}{1.2 \cdot 2} = 7.5;$$

according to the nomogram we find $B_1 = -2.8$, $B_2 = 0.32$. The computed values of A and B are substituted in formula (1), then

$$F_2' = -2.35 - 1.4 - (0.195 - 1.65) - (-2.8 - 0.32) = +0.82.$$

In Table XXXII the first line represents the computations carried out above. Analogously, the second and third lines are written:

when $y = 20$ m $F_2'' = +0.27$;

when $y = 22$ m $F_2''' = -0.42$.

Plotting the points obtained on the auxiliary graph $F = f(y)$ (Fig. 49) and connecting them with a line, we see that it intersects the line F_1 at the point for which $y = 22$ m, i.e. the depth of thaw for $x = 9$ is 22 m. On the left-hand side of Fig. 49 is shown the configuration of the thaw basin obtained as the result of this calculation.

Similar calculations are carried out for other values of x. In Table XXXII the form which is recommended is part of the calculation carried out in solving the above example. It should be noted that readings from the nomogram should be made accurately.

Static calculations. A characteristic feature of diagram III is the gradual long-term lowering of the permafrost table under a structure. As a rule the soil does not thaw uniformly under a building; under heated buildings of circular shape the interface between the thawed and frozen soil has the shape similar to that of a segment of a sphere; under buildings having the shape of an extended rectangle, the form is that of an elongated trough and so on. In addition to the differential thawing there is always some non-homogeneity in the properties of the soil. These two reasons along with uneven distribution of the external load result in differential settlement of the foundation soil.

Since deformation of a structure within the limits of conservation of its continuity is much less than deformation of thawing soil, differential settlement of the soil leads to redistribution of these actions between the foundation soil and the foundation footing. The greatest reaction of the foundation soil is observed where there is the least settlement of the soil, i.e. where the depth of thaw and compressibility of the soil is small; the least reactions of the foundation soil occur where there is the greatest settlement of the soil, i.e. where there is the greatest depth of thaw and compressibility of the thawed soil under load.

In sections where the reaction of the foundation soil is concentrated, the forces may be very large but they will not exceed a certain value corresponding to the maximum strength of the soil, since the soil will begin to deform and be pressed out of the areas with higher pressure to areas with lower pressure. In places where the reaction is at a low level the limit is zero. If at some place the reaction of the foundation soil reaches zero, further thawing of the soil will result in an expansion of the area with zero reaction.

If a structure is of sufficient rigidity and strength it may be assumed that it settles as a rigid body and its displacement in space consists of its own proper settlement s_o and tilts in two mutually perpendicular directions $tg\alpha_x$ and $tg\alpha_y$, as shown in Figure 53a. Further calculations are based on this mechanism and where necessary it can be refined and adapted for expressing the settlement of a structure having some flexibility.

The presence of settlement at times amounting to several tens of centimetres of tilt and substantial bending and turning moments, as well as transverse forces resulting from redistribtuion of the reaction of the foundation soil, determine the general design in calculation of the structure for the purpose of adapting it to a thawing foundation soil, namely:

(1) the design of the structure in the form of a rigid three-dimensional system;

(2) the subdivision of a structure into units dividied by settlement joints and these units are to be considered as independent structures, i.e. designed likewise as rigid three-dimensional systems;

(3) design arrangements permitting planned distribution and interrelationship of heavy machinery while the building is settling.

In this chapter we are considering only the permafrost basis of the design of structures with adaption to thawing of the foundation soil, i.e. calculation of settlement redistribution of the reaction between the foundation soil and the footing.

Other calculations and design questions are not considered here since they are solved by standard means of statics of the structure and design.

In considering the redistribution of reactions of the thawing foundation soil on the footing, let us imagine a diagram in which a building erected on frozen soil is replaced by a solid slab taking position I in Fig. 53 perpendicular to the force of gravity. For simplicity in calculation let us consider that slab I before thawing lies in the plane of the foundation footing. On thawing of the foundation soil and followed by differential settlement, the building changes to position II and equilibrium is always maintained between the load and reaction forces of the foundation soil. Let us assume the origin of the coordinates to be at point O, the intersection of an equivalent load and the dead weight of the foundation with slab I and the axis of the coordinates x and y lying in the plane of this slab.

Then the displacement of any point m with coordinates x and y belonging to the footing of the foundation will be represented by the expression

$$s_{x,y} = s_o - x \text{ tg } \alpha_x - y \text{ tg } \alpha_y, \tag{6.28}$$

where s_o - settlement of point O;

where α_x, α_y - angles expressing tilt of the structure in directions of the x and y axes. The sign of the angles of tilt is assumed to be positive if on increase in the coordinates the angle moves upwards, and negative in the converse case.

Figure 53b shown settlement and tilt of a structure. We denote the entire area of the foundation by the letter Ω and we will divide it into unit areas $\Delta\omega$. We will consider that within the limit of the given unit area the specific reaction of the foundation soil has the value q_1, and the complete reaction has the value $q_1\Delta\omega$. Part of the area of the foundation within the range of which the reaction q = 0, i.e. the area of sections with zero reaction, is denoted by Ω_o, the area of sections undergoing plastic deformation by Ω_{pl} and the area where on thawing there is compaction of the thawed soil by Ω_{ycomp}, then:

$$\Omega = \Omega_o + \Omega_{pl} + \Omega_{ycomp}. \tag{6.29}$$

Static conditions of equilibrium of the structure under the influence of load and reaction forces in position II are expressed by the system of equations

$$\left.\begin{aligned} \sum_{0}^{\Omega_{ycomp}} q_{x,y}\Delta\omega + \Omega_{pl}\cdot q_{pl} &= P; \\ \sum_{0}^{\Omega_{ycomp}} q_{x,y}x\Delta\omega + \Omega_{pl}q_{pl}x_1 &= 0; \\ \sum_{0}^{\Omega_{ycomp}} q_{x,y}y\Delta\omega + \Omega_{pl}q_{pl}y_1 &= 0, \end{aligned}\right\} \tag{6.30}$$

where x and y - coordinates of equivalent reaction forces acting within the limits of the area Ω_{pl}.

p - equivalent weight of the building including the weight of the foundation and load.

Reaction $q_{x,y}$ is a function of the coordinates and the nature of its relationship to x and y varies for different parts of the foundation soil area. For sections of zero reaction q = 0 and for sections of plastic deformation $q = q_{pl}$.

Expressing the relationship between the specific load $q_{x,y}$ at the point m (x,y) and the value of settlement s_o of soil thawed to the depth $h_{x,y}$ in the form $q_{x,y} = f\left(\frac{s}{h}\right)_{x,y}$, one can write

$$q_{x,y} = f\left[\frac{1}{h_{x,y}}(s_o - x \text{ tg } \alpha_x - y \text{ tg } \alpha_y)\right]. \qquad (6.31)$$

Substituting the value of $q_{x,y}$ from equation (6.31) in the system of equations (6.30) and solving them with respect to the values of s_o, tg α_x, and tg α_y, one can compute the reaction of $q_{x,y}$ for any point of the foundation footing.

Further development of this calculation system is determined by the form of the function f in the relationship

$$q_{x,y} = f\left(\frac{s}{h}\right)_{x,y}.$$

Assuming that this relationship is approximately linear and using the N.A. Tsytovich formula one can write the expression for settlement s in relationship to the local load in the form

$$s = \frac{ha\left(2h_s - \frac{h}{s}\right)}{2h_s} q + Ah,$$

hence taking into account equation (6.28)

$$q_{x,y} = \frac{2h_s}{ha\left(2h_s - \frac{h}{s}\right)}[s_o - x \text{ tg } \alpha_x - y \text{ tg } \alpha_y - Ah], \qquad (6.32)$$

where h_s - thickness of the equivalent layer;

h - depth of thaw at point m (x,y);

A - thawing coefficient, i.e. variation in the porosity coefficient on thawing under load of the building's dead weight;

a - compaction coefficient or the relation to the value of p of part of the change in the porosity coefficient depending on external load p.

After substitution of the values of s and $q_{x,y}$ from equation (6.28), (6.32) in the system of equation (6.27), the latter takes on the form of (6.33)

$$\left.\begin{aligned}
&s_o \sum_{0}^{\Omega_{ycomp}} u - \mathrm{tg}\,\alpha_x \sum_{0}^{\Omega_{ycomp}} ux - \mathrm{tg}\,\alpha_y \sum_{0}^{\Omega_{ycomp}} uy - L = 0;\\
&s_o \sum_{0}^{\Omega_{ycomp}} ux - \mathrm{tg}\,\alpha_x \sum_{0}^{\Omega_{ycomp}} ux^2 - \mathrm{tg}\,\alpha_y \sum_{0}^{\Omega_{ycomp}} uxy - M = 0;\\
&s_o \sum_{0}^{\Omega_{ycomp}} uy - \mathrm{tg}\,\alpha_x \sum_{0}^{\Omega_{ycomp}} uxy - \mathrm{tg}\,\alpha_y \sum_{0}^{\Omega_{ycomp}} uy^2 - N = 0,
\end{aligned}\right\} \qquad (6.33)$$

where

$$u = \frac{2h_s}{ah\left(2h_s - \frac{h}{2}\right)} \Delta\omega \quad \text{kg/cm};$$

$$L = \sum_{0}^{\Omega_{ycomp}} Auh + h_\varphi\gamma \sum_{0}^{\Omega_{ycomp}} auh + \frac{\gamma}{2} \sum_{0}^{\Omega_{ycomp}} auh^2 - q_{pl}\Omega_{pl} + P \ (\text{kg});$$

$$M = \sum_{0}^{\Omega_{ycomp}} Auhx + h_\varphi\gamma \sum_{0}^{\Omega_{ycomp}} auhx + \frac{\gamma}{2} \sum_{0}^{\Omega_{ycomp}} auh^2x - q_{pl}\Omega_{pl}x_1 \ (\text{kg}\cdot\text{cm}),$$

$$N = \sum_{0}^{\Omega_{ycomp}} Auhy + h_\varphi\gamma \sum_{0}^{\Omega_{ycomp}} auhy + \frac{\gamma}{2} \sum_{0}^{\Omega_{ycomp}} auh^2y - q_{pl}\Omega_{pl}y_1 \ (\text{kg}\cdot\text{cm}).$$

The boundaries between the regions of varying states of the soil are established from the assumption that at the boundary between the region of soil compaction and the region of plastic deformation there is the condition

$$q_{x,y} = q_{pl}, \qquad (6.34)$$

and at the boundary between the region of soil compaction and the region of zero reaction there is the condition

$$q_{x,y} = 0. \qquad (6.35)$$

Equations (6.33, 6.34, 6.35) are written in the general form suitable for calculating the settlement and tilt of any rigid structure for any given configuration in plan, distribution of external load, depth and shape of the thaw basin and soil properties expressed by any given graph.

The solution of these equations gives numerical values of settlement and tilt of the structure s_o, $tg\ \alpha_x$ and $tg\ \alpha_y$ and the boundaries separating the sections where the soil is compacted from those of zero reaction and plastic deformation. After substituting these values in equation (6.32) one can obtain the expression for reactions of the foundation soil at any point of its contact with the foundation footing.

Using some method of thawing the soil, for example the contact pressure method (Lapkin, 1943), to express settlement of thawing soil one can construct a system of calculation equations analogous to the system (6.33) differing from the latter only in details.

The result of calculation is greatly affected by inaccurate estimate of the configuration of the thaw basin under the building and the establishment of q_{pl} - the calculation value of the maximum pressure on the soil corresponding to the state of the limiting equilibrium of the soil.

Example 1. It is required to calculate the distribution of reactions of the thawing foundation soil under a foundation in the form of a rectangular plate ABCD having dimensions of 800 x 600 cm. The plate is loaded taking into account the dead weight of the continuous load and along the edge AB $q_{AB} = 1$ kg/cm^2, and along the edge CD - 4 kg/cm^2.

The calculated depths of thaw are given in Fig. 54 graphically by the solid line contours in centimetres. The foundation soil is considered to be unvaried over the area $A = 0.015$, $a = 0.005$ cm^2/kg, $\gamma = 0.0018$ kg/cm^2, $h_s = 300$ cm, $q_{pl} = 6$ kg/cm^2. The depth of the foundation is taken to be $h_f = 250$ cm.

We carry out preliminary calculations. The total load on the soil: $P = 800 \cdot 600\ \frac{1 + 4}{2} = 120 \cdot 10^4$ kg. The distance from the centre of gravity of the load to the edge AB

$$A_1O = \frac{800}{3} \cdot \frac{2.4 + 1}{4 + 1} = 480 \text{ cm.}$$

In the direction of the shorter side of the plate the load does not change and therefore the centre of gravity lies on the axis A_1C_1. For a unit area we take a square 200 x 200 cm; then the plate is subdivided into 12 unit areas.

For most practical cases the solution can be found by summing and it is recommended to make the calculations in the form of Tables XXXIII - XXXVI.

Using the sums obtained from Table XXXIV we get

$$\sum Auh = A\sum uh = 0{,}015\cdot 3326{,}39 = 49{,}896 \text{ kg};$$

$$\sum auh = a\sum uh = 0{,}005\cdot 3326{,}39 = 16{,}632 \text{ см}^2;$$

$$h_\Phi\gamma\sum auh = h_\Phi\gamma a\sum uh = 250\cdot 0{,}0018\cdot 16{,}632 = 7{,}484 \text{ kg};$$

$$\sum auh^2 = a\sum uh^2 = 0{,}005\cdot 1111{,}673 = 5558{,}37 \text{ см}^3;$$

$$\frac{\gamma}{2}\sum auh^2 = \frac{\gamma}{2}a\sum uh^2 = \frac{0{,}0018}{2}\cdot 5558{,}37 = 5{,}00 \text{ kg}.$$

As a first approximation we assume $\Omega_{pl} = 0$, i.e. we assume that there are no plastic deformationsin the foundation soil. Substituting the calculated values in formual (6.33) we determine L, M, N.

$$L = (49.896 + 7.484 + 5.00)\cdot 4\cdot 10^4 + 120\cdot 10^4 = 92.38\cdot(4\cdot 10^4)\text{ kg}.$$

To find M also using Table XXXIV we make the calculation,

$$A\sum uhx = 0.015\ (-308{,}374) = -4625.61 \text{ kg}\cdot\text{cm};$$

$$h_\varphi\gamma\alpha\sum uhx = 250\cdot 0.0018\cdot 0.005\ (-308{,}374) = -693.84 \text{ kg}\cdot\text{cm};$$

$$\frac{\gamma}{2}\alpha\sum uh^2x = \frac{0.0018}{2}\,0.005\ (-139{,}652{,}000) = -628.43 \text{ kg}\cdot\text{cm};$$

$$M = -(4625.61 + 693.84 + 628.43)\cdot 4\cdot 10^4 = -5947.88\ (4\cdot 10^4)\text{ kg}\cdot\text{cm};$$

and finally for the expression N we determine:

$$A\sum uhy = 0.015\ (-41{,}842) = -627.63 \text{ kg}\cdot\text{cm};$$

$$h_\varphi\gamma\alpha\sum uhy = 250\cdot 0.0018\cdot 0.005\ (-41{,}842) = -94.14 \text{ kg}\cdot\text{cm};$$

$$\frac{\gamma}{2}\alpha\sum uh^2y = \frac{0.0018}{2}\cdot 0.005\ (-50{,}211{,}000) = -225.95 \text{ kg}\cdot\text{cm};$$

$$N = -(627.63 + 94.14 + 225.95)\cdot 4\cdot 10^4 = -947.72\ (4\cdot 10^4)\text{ kg}\cdot\text{cm}.$$

Substituting the coefficient obtained from Table XXXV multiplied by $4\cdot 10^4$ and also the previously calculated values of L, M and N in the system of equations (6.33) and reducing all terms by $4\cdot 10^4$ we obtain:

$$10.618\ s_o + 637.76\ \text{tg}\ \alpha_x - 192.38\ \text{tg}\ \alpha_y - 92.38 = 0; \qquad (a)$$

$$-637.76\ s_o - 578{,}936\ \text{tg}\ \alpha_x + 20{,}500\ \text{tg}\ \alpha_y + 5947.88 = 0, \qquad (b)$$

$$192.32\ s_o + 20{,}500\ \text{tg}\ \alpha_x - 285{,}470\ \text{tg}\ \alpha_y + 947.72 = 0. \qquad (c)$$

Eliminating in turn form equations (a) and (b) all terms containing $tg\ \alpha_x$ and $tg\ \alpha_y$ we obtain

$$tg\ \alpha_x = +\ 0.000966\ s_o - 0.00762;$$

$$tg\ \alpha_y = 0.05839\ s_o - 0.5055.$$

Substituting these values in equation (c) we obtain

$$s_o = 8.94\ cm;$$

$$tg\ \alpha_x = 0.000966 \cdot 8.94 - 0.00762 = 0.00102,$$

$$tg\ \alpha_y = 0.05839 \cdot 8.94 - 0.5055 = 0.0165.$$

Now we have all the data required to calculate the reactions of the foundation soil for each unit of area of the foundation footing (according to formula (6.32)). For convenience these calculations are arranged in the form of Table XXXVI.

Comparing the sum of reactions on unit areas of the base obtained by calculation with the total external load we see that the difference between them comprises

$$P - \Sigma Q = 1{,}200{,}000 - 1{,}200{,}830 = -830\ kg,\ or\ 0.07\%.$$

Thus the calculations obtained can be considered sufficiently accurate.

Example 2. The planned building is rectangular in shape 500 x 400 cm (Fig. 55). The walls rest on strip foundation 100 cm wide. The equivalent force to the external load P = 150 T passes through point O, located on the longitudinal axis of the building at a distance of 25 cm from the point of intersection of the diagonals. The depth of the foundation h_f = 250 cm; the thickness of the equivalent layer

$h_s = 350$ cm.
$A = 0.02$,
$a = 0.005\ cm^2/kg$;
$\gamma = 0.0018\ kg/cm^3$.

For this example, tables are constructed for the auxiliary values and coefficients analogous to Tables XXXIV and XXXV. We do not give the tables here but restrict ourselves to giving the sum for each column

$uh = 3{,}632.7$ kg;
$uh^2 = 1{,}166{,}051\ kg \cdot cm$;
$uhx = 91{,}836\ kg \cdot cm$;
$uhy = 34{,}145\ kg \cdot cm$;
$uh^2x = 33{,}207{,}700\ kg \cdot cm^2$;
$uh^2y = 47{,}817{,}350\ kg \cdot cm^2$.

We determine the values of expressions required to calculate L, M and N.

$$\sum Auh = 0.02 \cdot 3632 = 72.65 \text{ kg};$$

$$\sum Auhx = 0.02\ (-91{,}836) = -1836.7 \text{ kg} \cdot \text{cm};$$

$$\sum Auhy = 0.02\ (-34{,}145) = -692.2 \text{ kg} \cdot \text{cm};$$

$$\sum auh = 0.005 \cdot 3632.7 = 18.16 \text{ cm}^2;$$

$$\sum auhx = 0.005\ (-91{,}836) = -459.18 \text{ cm}^3;$$

$$\sum auhy = 0.005\ (-34{,}145) = -170.72 \text{ cm}^3;$$

$$\sum auh^2 = 0.005 \cdot 1{,}166{,}051 = 5830.26 \text{ cm}^3;$$

$$\sum auh^2x = 0.005\ (-33{,}207{,}700) = -166038.5 \text{ cm}^4;$$

$$\sum auh^2y = 0.005\ (-47{,}817{,}350) = -239{,}087 \text{ cm}^4.$$

Using formulae (6.33) we obtain

$$L = (72.65 + 250 \cdot 0.0018 \cdot 18.16 + \frac{0.0018}{2} 5830.26 + 15) \cdot 10^4 = 101.07 \cdot 10^4 \text{ kg};$$

$$M = -(1836.7 + 250 \cdot 0.0018 \cdot 459.18 + \frac{0.0018}{2} 166038.5) \cdot 10^4 =$$

$$= -2192.77 \cdot 10^4 \text{ kg} \cdot \text{cm};$$

$$N = -(682.9 + 250 \cdot 0.0018 \cdot 170.72 + \frac{0.0018}{2} 239{,}087) \cdot 10^4 =$$

$$= -974.90 \cdot 10^4 \text{ kg} \cdot \text{cm}.$$

The coefficients with s_o, tg α_x, tg α_y have the following values (for the sake of brevity the calculations are not given):

$$\sum u = 12.346 \text{ kg/cm}; \qquad \sum ux^2 = 330{,}518 \text{ kg} \cdot \text{cm};$$

$$\sum ux = -382.1 \text{ kg}; \qquad \sum uy^2 = 207{,}617 \text{ kg} \cdot \text{cm};$$

$$\sum uy = +275.0 \text{ kg}; \qquad \sum uxy = -36{,}255 \text{ kg} \cdot \text{cm}.$$

Substituting the coefficients obtained in the system of equations (6.33) and reducing by 10^4, we write these equations in the following form:

$$12.346\ s_o + 382.1\ tg\ \alpha_x - 275\ tg\ \alpha_y - 101.07 = 0;$$

$$-382.1\ s_o - 330{,}518\ tg\ \alpha_x + 36{,}255\ tg\ \alpha_y + 2192.77 = 0;$$

$$+ 275.0\ s_o + 36{,}255\ tg\ \alpha_x - 207{,}617\ tg\ \alpha_y + 974.9 = 0;$$

as a result of solving these equations we obtain:

$$s_o = 8.58\ \text{cm};$$

$$tg\ \alpha_x = -0.00156;$$

$$tg\ \alpha_y = 0.0158.$$

Substituting these values in equation (6.32), we find the reaction of the foundation soil summarized in Table XXXVIII.

Comparing ΣQ with the given load P = 150 tons, we see that the diversions between them is expressed by the value $\Sigma Q - P = 150{,}004 - 150{,}000 = 4$ kg. Regardless of the good agreement of the calculations the distribution of reactions of the foundation soil which were obtained cannot be considered final, since under the foundation footing negative pressures cannot occur at sections 5, 10 and 11.

The calculation should be repeated in the order of the subsequent approximation, eliminating from the table of auxiliary values the coefficients at $tg\ \alpha_x$ and $tg\ \alpha_y$ values corresponding to areas with negative reactions.

As a result of these calculations the basic equations (6.33) are written as follows:

$$10.258\ s_o + 431.91\ tg\ \alpha_x - 339.61\ tg\ \alpha_y - 79.99 = 0;$$

$$- 431.91\ s_o - 294{,}795\ tg\ \alpha_x + 72{,}289\ tg\ \alpha_y + 2144.73 = 0;$$

$$+ 339.61\ s_o + 72{,}289\ tg\ \alpha_x - 160{,}636\ tg\ \alpha_y - 364.545 = 0,$$

hence

$$s_o = 8.34\ \text{cm};$$

$$tg\ \alpha_x = -0.0013;$$

$$tg\ \alpha_y = 0.0148.$$

There remains to construct a table of reactions of the foundation soil at the unit areas (Table XXXIX).

The sum of reactions of the foundation soil differs from the load by 150,000 - 148,231 = 1769 kg, i.e. by 1.15% < 5%. This diversion can be expanded proportionally to the reaction values multiplying the latter by the coefficient

$$\frac{150{,}000}{148{,}231} = 1.012.$$

During the calculations, pressures may be encountered which exceed permissible values, i.e. plastic deformation will be observed resulting in squeezing out of the soil from beneath the foundation. During calculation, pressure will decrease to the value of q_{pl} and even lower. This must also be expressed in the calculation as shown in formulae (6.33).

Example 3. It is required to calculate the reactions of the foundation soil under a building resting on a strip foundation. All the initial data are taken from the previous example with the exception of the values of A and a, which are taken to be variable; they are shown in Table XL and graphically in Fig. 56. The calculations differ from examples 1 and 2 by the impossibility of carrying A and a outside the summation sign.

Omitting for the sake of brevity the table of auxiliary values, we write only the sums derived from its columns:

$$\sum uh = 3819.5 \text{ kg}; \qquad \sum auhx = -459.95 \text{ cm}^3;$$

$$\sum uh^2 = 1{,}181{,}480 \text{ kg} \cdot \text{cm}; \qquad \sum auh^2x = -155305.7 \text{ cm}^4;$$

$$\sum auh = 18.162 \text{ cm}^2; \qquad \sum Auhy = -456.8 \text{ kg} \cdot \text{cm};$$

$$\sum Auh = 72.9 \text{ kg}; \qquad \sum auhy = -172.6 \text{ kg} \cdot \text{cm}^3;$$

$$\sum Auh^2 = 5828 \text{ kg} \cdot \text{cm}; \qquad \sum auh^2y = -238{,}934 \text{ cm}^4;$$

$$\sum Auhx = -2290.6 \text{ kg} \cdot \text{cm};$$

according to which we determine:

$$L = 101.34 \cdot 10^4 \text{ kg};$$

$$M = -2637.37 \cdot 10^4 \text{ kg} \cdot \text{cm};$$

$$N = -749.51 \cdot 10^4 \text{ kg} \cdot \text{cm}.$$

Constructing Table XL for coefficients analogous to Table XXXIV in Example 1, we obtain

$$\sum u = 13.55 \text{ kg} \cdot \text{cm};$$

$$\sum ux = -737.9 \text{ kg};$$

$$\sum uy = 486.5 \text{ kg};$$

$$\sum ux^2 = 383{,}840 \text{ kg} \cdot \text{cm};$$

$$\sum uy^2 = 220{,}505 \text{ kg} \cdot \text{cm};$$

$$\sum uxy = -74{,}626 \text{ kg} \cdot \text{cm}.$$

Substituting the values obtained in equation (6.33) we obtain:

$$13.55\ s_o + 737.9 \text{ tg } \alpha_x - 486.5 \text{ tg } \alpha_y - 101.34 = 0;$$

$$-737.9\ s_o - 383{,}840 \text{ tg } \alpha_x + 74{,}626 \text{ tg } \alpha_y + 2637.37 = 0;$$

$$486.5\ s_o + 74{,}626 \text{ tg } \alpha_x - 220{,}515 \text{ tg } \alpha_y + 749.51 = 0.$$

Solving these equations we obtain:

$$s_o = 8.50 \text{ cm}, \text{ tg } \alpha_x = -0.0055, \text{ tg } \alpha_y = 0.0202.$$

The calculated reactions of the foundation soil are shown in Table XLI. Eliminating, as in Example 1, the auxiliary calculations of areas with negative reactions we construct a new system of equations (6.33)

$$11.682\ s_o + 769.95 \text{ tg } \alpha_x - 560.3 \text{ tg } \alpha_y - 81.87 = 0;$$

$$-769.90\ s_o - 353{,}105 \text{ tg } \alpha_x + 106{,}037 \text{ tg } \alpha_y + 2492.46 = 0;$$

$$560.3\ s_o + 106{,}037 \text{ tg } \alpha_x - 178{,}395 \text{ tg } \alpha_y - 651.46 = 0,$$

after solving which we obtain

$$s_o = 8.24, \text{ tg } \alpha_x = -0.0052, \text{ tg } \alpha_y = 0.0192.$$

Calculated reactions of the foundation soil in the second approximation are given in Table XLII.

The inconsistancy between the reactions of the foundation soil and the given load P comprises

$$150{,}318 - 150{,}000 = 318 \text{ kg, or } 0.2\% < 5\%.$$

The result is corrected by multiplying the value of Q by the coefficient

$$\frac{150{,}000}{150{,}383} = 0.998.$$

The reactions of the foundation soil in kg/cm^2 is shown by the curves in Figure 56.

6. Bases and Foundations of Structures Erected with Preconstruction Thawing of the Foundation Soils

The essential point of preconstruction thawing as a method of presenting extensive and rapid settlement of the foundation soil, and resulting deformation of the structure, consists in an artificial approximation of the performance of the foundation to that normally occuring in regions where permafrost is absent. An important advantage of this method over that described in Section 5 is that it can be used for construction on any unfavourable soils.

As long ago as the last century it was noted that the thawing of frozen soil under the foundation of a structure results in differential settlement and deformation of the structure. Current investigations have shown that deformation of the structure depends not only on the amount of settlement but also on the rate of settlement. The more rapid the settlement, the more unfavourable is the stress deformational state of the material of which the structure is made. At the present time the relationship between the strength of a structure and the rate of settlement has not yet been sufficiently investigated but from observations of existing buildings one can see that attenuating settlement occurring during the first years after construction at the rate of 0.1 mm per day in most cases does not result in failure of masonry foundations (Abelev, 1948, Zhukov 1957 and others).

The compaction of frozen foundation soils on thawing under the influence of the dead weight of a structure comprises a substantial portion of the total compression of the soil under load. By thawing and compacting the foundation soil before the building is erected one can greatly improve the conditions of its stability and decrease the non-uniformity and rate of settlement. This question has always been raised by builders who have frequently expressed a desire in the literature to carry out measures before construction to improve the bearing capacity* of the foundation soil for structures erected on permafrost. However up to the present time little has been done about this problem. One of the examples of the practical application of this method is the Bayangol Power Station erected in 1948. (N.I. Saltykov, 1952-b), where on foreseeing that considerable settlement of the foundation soil under the boilers would be unavoidable after the expected thawing of the foundation soil, the builder undertook accelerated artificial thawing of the soil and compacting it by means of a special temporary load before the boilers were installed. The absolute value of nonuniformity and rate of settlement of a structure reach the highest values when the foundation soil thaws after the building has been occupied. With preconstruction preparation of the foundation soil settlement is much less and, moreover, it occurs partially during the period of construction and partially during the initial period of the operation of the building, when the masonry foundation is less subject to damage from displacement.

Knowing the structure of the frozen soil, its ice content, composition, etc., from permafrost data, one can determine the amount and rate of settlement of each layer of soil on thawing during any period when the structure is in operation. From this one can decide whether the construction should be adapted to thawing of the foundation soil when the building is in operation or whether it is necessary to thaw the foundation soil before construction. In this way the sphere of application of these two methods can be defined.

The adaptation of the structure to thawing of the foundation soil is applicable when the rate of settlement (compaction) of the foundation soil is less than that permitted for the structure, depending on the type of structure and the nature of the material, and when the final value of settlement does not exceed building standards. If the rate of settlement is greater than that permissible, the foundation soil should be improved, i.e. it should be thawed and compacted. As a rule, in both cases there is no need to place the foundation deeper than required considering the depth of seasonal freezing of the soil, but an important difference is that in the presence of soil strata with high ice content, if the construction is adapted to differential settlement, the foundation should be extended below the strata; if the foundation soil is to be thawed and compacted before construction this requirement is not always necessary.

If with given structural features the final configuration of the thaw

---

* By "improving the bearing capacity of the base soil" is meant some measures to reduce the pliability, i.e. the compaction, shear and plastic flow etc., of foundation soils under load.

basin under the structure is known, and the rate of formation is known, then, knowing the compactibility of the soil on thawing and the maximum permissible rate of settlement of the structure v_{max}, one can determine the minimum required depth of preconstruction thawing of the soil (Zhukov, 1957). Fig. 57 shows the maximum configuration of the thaw basin, the curve of the increase in thawing of the foundation soil $h = f(t)$, the curve of the expected increase in settlement during the course of time $s_h = \varphi(t)$ and the broken curve of the rate of settlement $v_s = \frac{ds}{dt}$, calculated layer by layer using the curve $s_h = \varphi(t)$ taking into account the compactibility of each thawed layer.

From Fig. 57 it can be seen that at the beginning of thawing (with small values of h) the settlement rate v_s calculated with the formula

$v_s = \frac{s_{i+1} - s_i}{t_{i+1} - t_i}$, has a high value and may be higher than the permissible value for the settlement rate v_{max}. To avoid the formation of cracks in the structure, the foundation soil should be thawed before construction to a depth h, at which the settlement rate $v_s < v_{max}$. Thus, having on the blueprint the relationship $v_s = \Psi(h)$, and knowing the maximum permissible settlement of the building v_{max}, one can determine graphically the required depth of preconstruction thawing. In Fig. 57 this depth of thaw is noted by the letter M. Since for various values of the thermal resistance of the floor R_{fl} and various values of the maximum rate of settlement v_{max} different depths of preconstruction thawing can be obtained, the design engineer can increase the maximum permissible rate of settlement of the structure by introducing rigid belts in the walls, provide settlement joints, increase the strength of the foundation, etc.; if the permissible rate of settlement is changed there is a change also in the maximum required depth of thaw. On the other hand, there may be also a variation in the thermal resistance of the floor R_{fl} on which the rate of thawing $v_s = \Psi(h)$ depends. Thus the amount of preconstruction work on improving the foundation soil required to ensure normal stability of the structure can be regulated. An example of practical calculation is given below.

Example. It is required to calculate the depth of preconstruction thawing of the foundation soil under a building 40 m long and 16 m wide proceeding from a maximum permissible rate of settlement of the structure $v_{max} = 0.1$ mm/day.

Data:

temperature in the building $\vartheta_1 = 16°C$;

temperature of the frozen soil in the region of zero amplitude $\vartheta_0 = -2°C$;

thermal resistance of the floor $R_{fl} = 2$ m²/hours · degrees/kcal;

the foundation soil consists of clay loam 3 m thick, under which there is a layer of sand 3.5 m thick, below which there is clay loam of unknown thickness.

Soil characteristics. Clay loam: $w = 300$ kg/m^3; $A = 0.013$; $a = 0.0027$ cm^2/kg; $\gamma = 1500$ kg/m^3; $\lambda_T = 1.2$ kcal/m · hour · degree; $C_T = 540$ kcal/m^3; sand: $w = 100$ kg/m^3; $\gamma = 1800$ kg/m^3; $A = 0.010$; $a = 0.0015$; $\lambda_T = 1.5$ kcal/hour · degree; $C_T = 400$ kcal/m^3; load of structure $P = 3$ kg/cm^2; $\lambda_{fr} = 1.8$ kcal/m · hour · degree.

The solution of the problem is divided into four parts.

1. The depth of thaw of the soil depending on time is determined.
2. Settlement of the structure depending on depth of thaw is determined.
3. The rate of settlement is determined.
4. The depth of preconstruction thawing based on results obtained and the maximum rate of settlement are established.

The formation of a thaw basin under the building as a function of time can be calculated with formula (4.34):

$$h_o = \frac{h}{1 + k\sqrt{t}}, \tag{a}$$

in which h can be determined with formula (4.26):

$$h = -S + \sqrt{S^2 + \frac{2\,(\lambda_T\vartheta_1 + m\lambda_{fr}\vartheta_o)\,t}{q_o\omega + \dfrac{C_T\vartheta_1}{2}}}, \tag{b}$$

where $m = 0.5 \div 0.6$.

Let us clarify the meaning of the value S in formula (b).

In calculating the thawing of each successive layer of soil, all the previously thawed layers provide thermal resistance which must be taken into account. For example, if at the moment of time t the depth of thaw reaches h, when $h < h_{\substack{\text{clay}\\\text{loam}}}$

$$S = \lambda_{T_1}\left(R_{f1} + \frac{h}{\lambda_{T_1}}\right), \tag{c}$$

and when $h_{\substack{\text{clay}\\\text{loam}}} < h < h_{\substack{\text{clay}\\\text{loam}}} + h_{\text{sand}}$

$$S = \lambda_{T_2}\left(R_{f1} + \frac{h_{\substack{\text{clay}\\\text{loam}}}}{\lambda_{T_1}} + \frac{h - h_{\substack{\text{clay}\\\text{loam}}}}{\lambda_{T_2}}\right), \tag{d}$$

where R_{f1} - the thermal resistance of insulation on the ground surface;

λ_{T_1}; λ_{T_2}- heat conductivity coefficients of clay loam and sand in the thawed state.

In calculations for constructing the curve $h_o = f(t)$, the reading of the depth of thaw is made from the top boundary of each layer and the reading of the time is made from the beginning of thawing of this layer.

The settlement of the structure as a function of the depth of thaw is determined by the formula

$$s_h = Ah_o + aph_o,$$

under the condition that the compaction of the soil is much more rapid than thawing.

The rate of settlement is defined as the increase in settlement in a unit of time

$$v_s = \frac{s_2 - s_1}{t_2 - t_1} = \frac{\Delta s}{\Delta t}. \qquad \text{(e)}$$

Calculation. The coefficient k in formula (a) determined with the empirical formula (4.34a) in relation to the dimensions of the structure (l, b), ice content in the soil (w) and heat conductivity of the soil in the thawed state.

Substituting in (4.34a) the values of l, b, λ_T and w we obtain

for sand $$k = 0.14\left(1 + \sqrt{\frac{8^3}{20^3}}\right)\frac{2 \cdot 1.5}{8^3 \cdot 100} = 0.0013;$$

for clay loam $$k = 0.14\left(1 + \sqrt{\frac{8^3}{20^3}}\right)\frac{2 \cdot 1.2}{8^3 \cdot 300} = 0.0007.$$

Let us determine the time of thawing of the upper layer of clay loam to the depth of h = 3 m, using formulae (a), (b) and (c):

$$h_{\substack{clay\\loam}} = 3 = \frac{-1.2 \cdot 2 + \sqrt{(1.2 \cdot 2)^2 + \dfrac{2\,(1.2 \cdot 16 - 0.5 \cdot 1.8 \cdot 2)\,t_1}{80 \cdot 300 + 540 \cdot \dfrac{16}{2}}}}{1 + 0.0007\sqrt{t_1}},$$

hence graphically or by trial we obtain t_1 = 22,000 hrs = 916 days. The time required to thaw the layer of sand

$$h_{\substack{clay\\loam}} = 3.5 = \frac{-1.5\left(2 + \dfrac{3}{1.2}\right) + \sqrt{\left[-1.5\left(2 + \dfrac{3}{1.2}\right)\right]^2 + \dfrac{2\,(1.5 \cdot 16 - 1.8 \cdot 0.5 \cdot 2)t_2}{80 \cdot 100 + 400 \cdot \dfrac{16}{2}}}}{1 + 0.0013\sqrt{t_2}},$$

hence t_2 = 18,000 hrs = 750 days.

For constructing the graph $h = f(t)$, the time of thawing of each layer is subdivided into intervals of 100 - 200 days. Then

$$\Delta h_1 = \delta_1 = -1.2 \cdot 2 + \sqrt{(-1.2 \cdot 2)^2 + \frac{2\,(1.2 \cdot 16 - 0.5 \cdot 1.8 \cdot 2)\,2400}{80 \cdot 300 + \frac{540 \cdot 16}{2}}} = 0.6 \text{ m.}$$

Taking into account the layer $\delta_1 = 0.6$ m as a thermal insulator we obtain

$$\Delta h_2 = \delta_2 = -1.2\left(2 + \frac{0.6}{1.2}\right) +$$

$$+ \sqrt{\left[-1.2\left(2 + \frac{0.6}{1.2}\right)\right]^2 + \frac{2\,(1.2 \cdot 16 - 0.5 \cdot 1.8 \cdot 2)\,2400}{80 \cdot 300 + \frac{540 \cdot 16}{2}}} = 0.47 \text{ m.}$$

Further calculations of the values of h_o, s_h and v_s do not present any difficulties and we do not give them here reducing all the calculations to the form of Table XLIII.

The calculation may be terminated here since the rate of settlement is considerably less than $v_{max} = 0.1$ mm/day.

On the basis of calculations obtained we construct the curves of $h_o = f(t)$, $s_h = \varphi(h)$ and $v_s = \Psi(s)$. In Fig. 57 it is seen that the curve of the actual rate of settlement intersects the line of maximum rate at points A and B and after point B the rate of settlement remains less than $v_{max} = 0.1$ mm/day. At this moment of time (t_o) the depth of thaw reaches $h = 4.3$ m. Thus to avoid deformation of the structure the foundation soil should be thawed to a depth of $h = MN = 4.3$ m. We note that formula (4.34) includes the value R_{fl} - the thermal resistance of the floor. By increasing R_{fl} we decrease simultaneously the rate of thawing and the rate of settlement and consequently the depth of preconstruction thawing will be less. This is very important when the depth of preconstruction thawing reaches high values, up to 10 or more metres.

If in our example we take $R_{fl} = 3$ m^2 hour · degree/kcal then on constructing new graphs of $h_o = f_1(t)$ and $v_s = \Psi_1(s)$ we find that the rate of settlement will exceed the maximum permissible rate only during the first 100 - 150 days during thawing of the clay loam layer. The required depth of preconstruction thawing at which the rate of settlement will be less than $v_{max} = 0.1$ mm/day for $R_{fl} = 3$ m^2 hour · degree/kcal will be only 0.7 - 0.8 m.

Economy may be achieved in the volume of soil subject to preconstruction thawing, in addition to limiting the depth of thaw, by decreasing the area applicable to the configuration of the foundation in plan (under strip foundation, separate posts, etc.), but this question is still in the development stage. If preconstruction thawing is carried out to only part of the required depth of width of the expected thaw basin, the preconstruction settlement of the foundation soil from its dead weight will also be incomplete and will continue during the time when the building is put into operation. However, this additional part of the settlement will be much less and slower than that which would occur without preconstruction preparation.

Preconstruction thawing should likewise not exceed the limits of the contour of the thaw basin since the soil thawed before construction will again freeze with possible heaving.

Preconstruction Thawing and Methods of Artificial Strengthening of Thawed Soil at the Base of Structures

The quality of the foundation soil prepared to take a foundation depends not only on the depth and spatial contour of the preconstruction thawing of the frozen soil but also on the methods and conditions of carrying out these operations, and also on methods of improving the soil properties. The methods of thawing used in the permafrost region are known. The choice of the method of strengthening the soil depends to some extent on the method of thawing.

The methods of thawing basically are subdivided into three groups depending on the type of heat used. Solar heat is used for thawing the soil by artificially changing the thermal balance at the earth surface; the heat contained in water applied under pressure, filtration or flooding the foundation soil may be used (Chap. V) or artificially supplied heat may be used in the form of electricity, steam, etc.

With rapid thawing, for example with steam jets, the natural structure of frozen soil is disturbed and even mineral seams are destroyed. Under the influence of hydrodynamic forces of the steam, soil particles are moved and the space between them is filled with water. The soil swells and loosens and consequently is subject to considerable compaction when a load is applied. If the soil is thawed without dynamic action, for example by electric current or by solar energy, etc., the mineral seams retain their compact state and in the place of contact the mineral seams break up and are subject to plastic displacement occupying a relatively compact distribution under load. When the space in the soil left by melted ice inclusions is filled by mineral matter, the soil becomes stronger and acquires a higher bearing capacity.

When thawing is due to the heat contained in water (experience of Dal'stroi), if water jets are used, the change to which the foundation soil is subjected is similar to that occurring when steam jets are used. Thawing by drainage-filtering or by sprinkling has an intermediate influence on soil structure. It is therefore, desirable to select the method of thawing the frozen soil in correspondence with the structure and lithological composition, for example, clay loam and sand undergo different changes in structure after thawing. In some cases it is possible to thaw the foundation soil at the same time as the foundation is being constructed. This makes it possible to reduce the time spent on construction if it is impossible to allocate time for preparing the foundation soil. This is the way the above-mentioned work in improving the soil under the foundations of boilers of the second series were carried out at the Bayangol Power Station.

In choosing and carrying out methods of preconstruction thawing and compaction of the foundation soil the determining conditions are as follows:

(a) the geographical location of the area under development and the nature of the permafrost in this area;

(b) the time of the year with respect to the general plan of operation designated for thawing and compaction of the foundation soil;

(c) design and engineering features of the structure (rigidity of the structure, thermal and water regime expected when the structure is in operation, etc.);

(d) engineering possibility of carrying out the work and economic indices of the efficiency and applicability of the methods of thawing and compaction of the soil at the given building site.

Only after taking into account all of the above-mentioned conditions can one correctly choose the method of preconstruction thawing and compaction of the foundation soil.

Strengthening of the thawed soils can comprise: (1) reduction of the space between the mineral aggregates by decreasing the volume of the macropores (Fedosov, 1944, Bakulin, 1953 and Zhukov, 1957a); (2) by reducing the space between the mineral particles after the aggregates have been broken down; (3) fixing the mineral aggregates by injection of hardening solutions such as silicates, chemical mixtures, etc., into the macropores of the thawed soil. Apparently one can use electrochemical fixing arranged in conjunction with electric thawing of the soil.

A feature of thawed soil as compared with soil that has not been frozen is the presence of a large number of macropores as a property of the cryogenic* texture of the frozen soil. With a high percentage of silt particles and soil moisture corresponding approximately to the liquid limit there is a tendency for the soil to convert into a slurry when subject to dynamic action.

---

* Cryogenic - resulting from freezing.

It should, however, be pointed out that it is incorrect to think of the liquefaction of thawing of large volumes of soil; transition to a slurry is strictly a local effect which is observed near steam jets and in the layer immediately below the surface.

Below is a brief consideration of the applicability to thawed soils of some known methods of strengthening soil that has not been frozen:

1. Compaction due to the dead weight of the soil particles occurs in manmade formations such as fills, dumps, etc. The process of compaction occurs slowly and varies with depth since the weight of each layer is different from the above layer. The coefficient of porosity of thawed soil on compaction varies with its compressibility properties, principally corresponding to the filtration coefficient which initially is more rapid than the compaction of unfrozen soil but subsequently is analogous to the compaction of unfrozen soil.

2. Compaction by vibration is known in building practice for compacting freshly dumped sand in fills, as well as soils in their natural state. The action of a vibrator consists in disturbing the bonds between the soil particles resulting in a more dense packing.

Thawed soil is characterized by dense packing of mineral particles in aggregates and by loose packing of aggregates. The action of the vibrator can break up the aggregates into small parts and bring about more dense packing. However, with prolonged operation of the vibrator, because of hydrodynamic action the aggregates begin to break down into individual mineral particles and the distance between the particles increases resulting in an increase in the porosity coefficient, i.e. the structure of the soil becomes less dense than that of the previously existing conglomerates. In this case the desired compaction of the soil may not be attained regardless of the general increase in its average density and the more uniform distribution of mineral particles. This operation of the vibrator is permissible and can be useful only if on termination of the work of the vibrator the water is removed from the pores by some method such as reducing the level of groundwater, electro-osmosis, etc.

3. Electro-osmosis for compacting thawed soil is applicable to clay soils where the movement of groundwater is very slow. It is applicable also to removing water from sand and particularily silt soils which are most frequently encountered in the permafrost region. The removal of water from the soil is carried out to accelerate the process of compaction. Water is transferred to the negative electrode and, if this electrode is made in the form of a perforated pipe, water will accumulate in it and may be pumped out.

The consumption of electric energy for drying 1 m^3 of soil is 4 - 10 kw hours (Podvyazkin, 1948).

Electro-osmosis can serve as one of the methods of using electric power to improve the construction properties of thawed soil. Chemical changes occurring in the soil due to the action of direct current result in additional strengthening, particularly if some chemical additives are used (Rzhanitsyn, 1937).

It should be noted that the quantity of water removed by electro-osmosis is not a reliable indicator of the reduction in pore water since water may flow in from the sides. Therefore, at the same time as the water is removed the soil should be compacted and careful control should be exercised to ensure the required compaction of the soil.

4. Electrochemical fixing of thawed soil is not being developed to any extent at the present time. This method can be applied to thawed soils and it can be assumed that in the future it may become the most effective method.

By using the particular features of soil moisture at temperatures slightly below 0°C there is a possibility of developing methods of electrochemical fixing of frozen soils without thawing them, limiting the operation to raising the temperature towards 0°C.

5. Compaction by piles is based on the fact that on driving the piles the soil particles are brought closer together and thus the porosity of the foundation soil is decreased. However, silts and clay loams cannot be compacted by driving wood or reinforced concrete piles since in such soils the vibration accompanying the driving of the piles decreases the bond between the mineral particles and increases their mobility, thus preventing any possibility of compaction. To compact the soil some process must be provided for allowing the water to flow freely from the pores. Under ordinary conditions this problem is solved by the use of soil piles (Abelev, 1948), which comprises a fill pile made of uncemented coarse-grained soil such as sand, sand and gravel, etc. Moisture flows readily from the surrounding thawed soil to the soil pile and where necessary can be gradually removed from the pile.

The addition of unslaked lime to the sand-gravel mixture is helpful in that the lime absorbs a large quantity of water (1 kg of lime absorbs 1.8 - 2.7 litres of water) and on expanding it compacts the thawed soil. Moreover, the reaction between lime and water liberates a large quantity of heat which is of practical importance in thawing frozen soil.

Experience in constructing soil-fill piles in the permafrost region is as yet lacking.

6. Silicate and bitumen impregnation of thawed soil consists in the

following. After thawing, the soil retains its previous structure to some extent leaving large pores with communication between them. Mineral seams comprise compact bodies and compaction of the thawed soil is achieved in this case by filling the macropores by a soil-strengthening compound.

Water can move freely through the macropores left after thawing. This property can be used to strengthen the thawed soil by impregnation with silicates or bitumen. Mortar or bitumen readily penetrates into the soil along the macropores and fixes it. In this case there would be no need to compact the soil after thawing which, as is known, requires considerable effort and results in displacement of the surface of the building site, which is not always desirable particularly when building operations are going on at the same time as the soil is being strengthened. Experience in impregnating frozen soil with silicates and bitumen after thawing is also lacking. Perennially frozen soil after thawing becomes just as penetrable for fluids as unfrozen soil and therefore all methods of strengthening soils known in building practice should be applicable to thawed soil. Regarding the design of foundations, in general all types of foundations used in construction on unfrozen weak soil are applicable to construction with preconstruction thawing of the soil.

7. Measures to Prevent Heaving and Calculation of the Stability of Foundations on Heaving

The heaving of foundations on freezing of the thawed layer of soil is one of the most widespread causes of deformation of structures in the permafrost region. As mentioned in Chapter III, it is explained by the action of tangential forces which are developed along the lateral surfaces of the foundation frozen to the surrounding heaving soil.

In this section, only measures to counteract the influence of tangential heaving forces applied to the lateral surfaces of the foundation are considered (Fig. 58).

Measures directed towards ensuring stability of structures from the influence of normal forces of heaving acting on the foundation footing, where the foundation is placed in the permafrost, are not considered for the following reasons:

(a) the magnitude and nature of these forces is not sufficiently elucidated to undertake engineering calculations;

(b) to ensure stability of the structures normal heaving forces can be eliminated by placing the foundation below the permafrost table. Heaving of temporary wooden structures does not cause a great deal of harm and can frequently be permitted.

(c) foundations of major structures outside the permafrost region are laid within the layer of seasonal freezing only under special strictly limited conditions, when there can be no ice formation under the footing of the foundation that would cause heaving.

In planning measures to counteract the influence of tangential heaving forces two checks should be made: (a) the general stability of the foundation to heaving and (b) the tensile strength of the foundation.

Checking the general stability of the foundation. The foundation is not subject to heaving if the following condition is satisfied

$$T_h \cdot k < (N + T_s), \tag{6.36}$$

where k - the safety factor established by N and TU 118-54 equal to 1.1 - 1.2 depending on the sensitivity of the structure to differential settlement;

T_h - tangential heaving forces;

T_s - reaction force of the adfreezing bond between the foundation and the permafrost or resistance of the thawed soil to compression (Fig. 59);

N - load applied to the foundation along with its dead weight.

In determining the forces T_h the following formulae are used

$$T_h = \tau_1 u \tag{6.37}$$

or

$$T_h = m\tau_2\omega, \tag{6.38}$$

where u - the perimeter of the foundation;

ω - surface area of the adfreezing bond between the foundation and the soil subject to summer thawing (or on building sites where the permafrost table is deep or permafrost is absent, with the soil layer subject to winter freezing);

m - the portion of the total area of the adfreezing bond within which there is actually an increase in ice formation;

τ_1, τ_2 - the specific value of heaving forces (kg per linear cm of the foundation perimeter and kg per cm² of the surface of the adfreezing bond).

Formula (6.37) is used primarily in standard building practice because of its general simplicity, and formula (6.38) is used in research work since it provides a detailed consideration of the distribution of tangential heaving stresses along the lateral surface of the foundation.

The reaction force T_s or long-term shear strength of perennially frozen soil bonded to the lateral surface of the lower part of the foundation counteracting the lifting of the foundation is determined by the formula

$$T_s = \Omega \tau_s , \qquad (6.39)$$

where Ω - the area of the adfreezing surface;

τ_s - long-term shear strength of the frozen soil.

Without repeating the information regarding the typical nature and the numerical values of m, τ_1 , τ_2 and τ_s given in Chapter III we will move on to questions of designing and the provision of antiheaving measures used in the construction of foundations under permafrost conditions.

Antiheaving measures can be subdivided into three categories: (a) measures to eliminate or reduce heaving of the soil adjacent to the foundation; (b) measures to reduce tangential forces transmitted by the heaving soil to the foundation; (c) measures to increase the resistance of the foundation to the influence of tangential heaving forces.

General reduction in soil heaving at a given site can be achieved by heating and drying. Soil heaving is absent where, owing to local heating, the temperature throughout the winter remains above or near 0°C. Drying of the soil by draining surface water and lowering the groundwater table is also a proved method of counteracting heaving. Another measure is to create conditions which facilitate the migration of ground moisture from the surface of the foundation. Adding salt to the soil counteracts heaving, but the effect of this measure lasts only 2 - 3 years.

Tangential forces transmitted to the lateral surface of the foundation by frozen soil can be reduced by various methods: decreasing the area of the adfreezing bond between the sides of the foundation and the soil subject to thawing by substantially sloping ($\geqslant 15°$) the lateral walls of the foundation to the vertical, making the lateral surfaces as smooth as possible and covering the lateral surfaces of the foundation with a hydrophobic substance (as yet a temporary measure). Increasing the temperature of the soil and drying it greatly reduces the tangential heaving forces in addition to a general reduction in soil heaving.

Attempts to insulate the foundation from heaving of the surrounding soil by inserting slipping or easily deformable material between the foundation surface in the soil did not give the expected effect. Coating the foundation also did not give positive results.

The resistance of foundations to tangential heaving forces is increased by increasing the load on the foundation, strengthening the anchorage of the foundation in the permafrost by local widening of the foundation, lengthening bearing posts, strengthening transverse reinforcement belts, etc. and by increasing the surface area of the adfreezing bond between the foundation and the permafrost.

It should be noted that anchoring the foundation ensures stability against heaving only if it can withstand the tension forces.

The most effective type of foundation for counteracting heaving is a heavily loaded post type with or without anchorage in the permafrost with good drainage of the seasonally thawing layer. Satisfactory resistance to heaving can be obtained by piles which are driven deep into the permafrost and frozen in place.

The correct determination of the calculation temperature of the permafrost in which the foundation is anchored plays an important role in establishing the value of τ_s. During the period of intensive heaving, i.e. at the beginning of winter, this temperature is close to 0°C as a rule. Therefore τ_s is always much less than τ_2.

According to the data of the Igarka Permafrost Station (Vyalov, 1956), the value for silty sand loam and clay loam with full moisture saturation varies with temperature in the following manner:

at temperature of	0 - 0.1°C	$\tau_s = 0.2$ kg/cm²;
" " "	0.2 - 0.3°C	$\tau_s = 0.4 - 0.5$ kg/cm²;
" " "	0.4 - 0.8°C	$\tau_s = 0.7 - 0.9$ kg/cm².

In calculating the minimum depth of placing the foundation in frozen soil the value of τ_s used in practice today cannot have a value greater than 0.2 kg/cm².

A reduction in the area of the frozen bond between the foundation and the soil layer subject to seasonal thawing ω is achieved by reducing the perimeter of the upper part of the foundation. The trend to reducing the perimeter of the foundation was one of the first reasons for rejecting strip type foundations and replacing them with individual posts. The load per bearing column is increased by increasing the space between the columns, which is one of the main advantages of this type of foundation, since in other types of foundations the load on the support cannot be increased to the same extent.

An increase in the holding force of the adfreezing bond T is achieved by increasing the depth at which the foundation is placed in the permafrost or by constructing anchoring devices.

The depth of sinking post foundations h with respect to resistance to heaving is determined from equation (6.40)

$$h_\Phi \geqslant \frac{T_s - N}{u_s \tau_s} + h_s, \qquad (6.40)$$

where u_s - perimeter of that portion of the foundation frozen into the permafrost;

h_s - greatest depth of summer thawing measured from the top of the foundation.

In calculating the resistance to heaving of foundations with bearing slabs it is assumed that it is due to the resistance to spalling of the frozen soil along the vertical surfaces passing through the perimeter of the bearing slab; the unit resistance of frozen soil to spalling differs little numerically from the adfreezing strength of the soil on the sides of the foundation. Because of this, formula (6.40) extends also to the case of a foundation with a slab if the perimeter u_s is extended to the outline of the slab in plan view. To increase the anchoring properties of post supports or reinforced concrete piles a change can be made in the cross-section, for example, making it in the form of a double-T (Egerev, 1957).

Deep permafrost table. When the permafrost table is deep and also when a structure is designed with adaptation to a thawing foundation soil, the heaving stability can be checked in the following way (Porkhaev, 1957).

Let us assume that in correspondence with Fig. 59 the thawed layer 1, between the lower surface of the frozen layer ab and upper surface of the anchor cd, is compressed by the load of the structure transmitted by forces of the adfreezing bond to the layer of frozen soil 2, and resistance of the anchor to being pulled out.

Using the solution of M.I. Gorbunov-Posadov (1946) with respect to normal stresses at the boundary between the compressed and non-compressed layers, one can express the resistance of the anchor to being pulled out by heaving forces greater than the weight of the structure by the equation (6.41) where

$$T_h = \left(\frac{\gamma_{fr} h_{fr}}{k} + \gamma_T h_T\right) (\omega_a - \omega_c), \tag{6.41}$$

where γ_{fr} - unit weight of frozen soil;

γ_T - unit weight of thawed soil;

h_{fr} - thickness of frozen layer;

h_T - thickness of thawed layer;

ω_a - area of anchor footing;

ω_c - cross-sectional area of the column;

k - coefficient derived by M.I. Gorbunov-Posadov to determine the stress at the boundary between the compressed and non-compressed soils used with respect to Table XLIV.

Since with an increase in the depth of freezing h_{fr} the thickness of the thawed layer h_T decreases, the value of T_{fr} according to formula (6.41) should be considered to be at the lowest value, i.e. calculation with a safety factor.

Checking the strength of foundations during heaving. From the total value of the load of the soil N let us derive the weight of the foundation N_f. If it turns out that during the period of heaving on fulfilling the condition (6.36) or (6.41) the foundation will function in tension under the influence of a vertical force equal to $T_h - N + N_f$, for which the column and anchor part of the foundation should be checked including all details and linkages of the anchor part.

Two checks of the foundation subject to heaving are not required in the following cases:

(a) if the seasonal thawing layer consists of coarse-grained well-drained relatively dry soil starting from coarse sand and ending with boulders with a very limited addition of fine-grained soil;

(b) if the soil contains little moisture, the ground water table is low and during the freezing period there is no possibility of water flowing to the lower boundary of the seasonally thawing layer.

8. Foundation Work Carried Out in the Permafrost Region

At the present time the problem of carrying out foundation work in the permafrost region has received little practical or theoretical development. Common experience in this type of work in most cases is of negative value, i.e. it indicates an incorrect approach of builders in organizing foundation work and the absence of any concern to improve it. Meanwhile the technique of carrying out foundation work in permafrost conditions is of particular importance, the more so because the frozen state of the ground and the severe climatic conditions do not always hinder operations but frequently facilitate and accelerate foundation work; it is sufficient to recall the construction of foundations in saturated soils by the method of freezing using natural or artificial cooling.

The basic feature of foundation work in permafrost conditions is the necessity of taking time into account. Time has a fundamental influence on changing the properties of the soil; in several days, or sometimes hours, frozen soil may thaw and thawed soil may freeze. The walls of an excavation may collapse and fill with water, etc. On the other hand, the organization and technique employed are in close relation to the construction methods adopted, i.e. with the thermal interaction between the structure and the soil. Climatic conditions and time of year during which the work is to be carried out influence the choice of method of operation and in labour productivity as shown in Chapter VII.

It can be stated that the direct influence of climatic conditions are determined by a short summer and a long cold severe winter, which in most regions of the high arctic is accompanied by strong winds and blizzards. However, this general characterization of the influence of climate on construction work is incomplete and insufficient. Methods of construction which are suitable in regions where winter anticyclones prevail (Yakutiya, DVK, Chita Oblast), where the climate in summer is dry and suitable for any type of operation and where the winter is without snow and without wind, may be completely unsuitable in the high arctic where the summers are wet and frequent blizzards occur in winter. The winters of the high polar regions where there are frequent blizzards require the builder to consider carefully problems of protecting the building site and transport route from snow drifting which may paralyze construction. In organizing construction work in the high arctic one must take into account the continuous sunlight in the summer where a three-month working day must be considered normal, and the polar night where for several months artificial illumination is required for all types of indoor and outdoor operations.

We give some examples of the influence of the above factors on carrying out construction work.

1. If excavation and building of the foundations is carried out in the second half of the summer or in the fall it is necessary to speed up operations to erect the superstructure so that by the time frost begins the foundation will be sufficiently loaded to counteract heaving. If it is impossible to accelerate work to this end, temporary measures to counteract heaving must be taken: heating of the foundations and the surrounding soil, covering the foundations to avoid freezing and heaving, etc.

2. In construction where the permafrost is to be preserved foundations of prefabricated reinforced concrete can be erected in any region throughout the year. In this case, foundations of monolithic concrete can be laid directly on the ground only in the Far North. In the southern portion of the permafrost region, concrete foundations can be poured in situ during the warm period of the year only when the foundation soil is supposed to thaw.

Preparation of the building site. One of the important organizational features of carrying out foundation work on perennially frozen ground is the necessity of preparative operations which must be planned rather fully.

The preparation of conditions which ensure that the operations will be carried out include the following: delivery of building materials at the proper time and distribution of the materials in locations convenient for supplying them to the excavation, adequate water pumping devices for work in the summer time, protection of the site from snow during winter, etc.

In some cases there may be a need for special equilization of permafrost conditions in the building site, i.e. partial freezing or thawing of the ground in some areas which may differ from adjacent frozen ground; a great deal of attention should be paid to grading operations and controlling surface water.

Levelling of the building site. The building site should be levelled before the excavations are dug since the levelling may change the position of the permafrost table and it is desirable that these changes occur before the structure is built. Levelling is required particularly to eliminate the possibility of surface water flowing into the excavations. If this requirement cannot be fulfilled completely, for example when the beginning of construction and bringing in of materials to the building site occurs in winter, the ground should be levelled to the planned grade in a strip about 8 - 10 m wide around the planned structure and drainage channels should be dug around the location of the future excavation.

Earth works and foundation construction. In digging excavations for foundations in permafrost conditions, digging has to be done usually in thawed as well as in frozen soil. Earth works are carried out in thawed soil by the methods used in digging excavations in moist and saturated soils. Excavations in frozen soil can be made by preconstruction thawing of the soil or by the use of explosives, pneumatic drills, etc. The thawing method is used usually when the work is performed during the warm season of the year and particularly when thawing is one of the measures to be undertaken to ensure stability of the structure. In other cases it is more efficient to use mechanical methods.

The specific difficulties encountered in digging excavations in frozen soil are connected with the time of the year when the work is carried out. In the summer, measures must be taken to counteract collapse of excavation walls and the flow of water into the excavation from the seasonally thawed layer. For this purpose the following measures are required: (a) the excavations should be made as quickly as possible, (b) protection of the excavation wall from direct solar radiation, (c) provision of barriers around the walls of the excavation and filling (in some cases) the space between the wall of the excavation and the barrier with sawdust or other insulating materials, (d) careful drainage of surface water.

Before the foundation is placed, the bottom of the excavation should be levelled with a layer of sand and the grillage installed, if this is provided for in the plan.

Excavation work in the winter eliminates some difficulties and the problem is reduced to the most effective way of breaking up the frozen ground and in northern regions to combatting snow drifting. In a number of cases where there is considerable flow of surface and groundwater, special measures are undertaken to freeze the excavation.

Foundations on permafrost can be either of prefabricated or monolithic concrete. In construction with the intention of preserving the frozen state of the soil foundations of prefabricated reinforced concrete were first used in 1934 in Yakutsk and at the present time this type of foundation can be considered fully mastered and the most effective way of construction during summer and winter. They can be used in southern regions providing that rapid backfilling is possible. Prefabricated foundations are fully suitable also to construction with preconstruction thawing of the soil. The question of the possibility and conditions of erecting prefabricated foundations when the construction method provides for thawing of the soil after the structure is in operation has not yet been solved, and requires further development particularly when considerable and differential settlement is expected.

The most complex method from the construction point of view must be considered monolithic foundations erected where the soil is to be preserved in its frozen sate. In this case the basic requirements are as follows:

(a) preservation of the soil under the footing of the foundation in the frozen state from the time the excavation is dug until backfilling is completed;

(b) provision of normal conditions for setting of the concrete if it is poured in the excavation or of the mortar if the foundation consists of prefabricated sections;

(c) compact backfilling of the foundation with soil and the construction of a protective decking on top of the soil;

(d) provision for resistance of the foundation to heaving during the first winter after construction.

It will be readily noticed that some of the above requirements are contradictory, such as, for example, the requirement of preserving the soil in the frozen state and the requirement of ensuring normal conditions for the setting of concrete poured in situ.

Special investigations (Chernigov, 1957) have established the possibility of using known methods of providing the required temperature for the hardening of concrete: the thermos method, electric heating and the electrothermos method. The heating of concrete foundations by steam in undesirable and can be permitted only where reliable measures can be taken to counteract the influence of steam and condensed water on the frozen foundation soil. As an experimental measure one can use "cold concrete". The concrete should be poured on wood insulation the thickness of which is determined by thermal calculations with the consideration that the soil temperature directly under the wood insulation would be equal to 0°C. In preparing the foundation soil one should be governed by the following data gained by experience:

(a) in the case of isothermic heating of the concrete laid directly on frozen soil having a moisture content by weight of the order of 25%, the frozen soil may thaw to a depth of 60-70 cm and more before the concrete reaches 90 - 100% of its design strength. With thermos cooling the hardening conditions of concrete in contact with the soil cannot be considered normal;

(b) if the concrete is layed directly on perennially frozen stony or gravel soil with a moisture content of 14 - 15%, then with isothermic heating of the concrete the conditions of hardening in contact with such a foundation soil are completely satisfactory and thawing will not cause settlement;

(c) if in computing the amount of insulation it turns out that the thawing of the soil will reach 15 - 20 cm, it is unnecessary and some times ineffective to provide for a greater thickness of insulation. In such cases it is better to provide a gravel-sand base than flood it with water and let it freeze;

(d) it may be expedient to cool the frozen foundation soil in the excavation if the concrete is to be heated for a short time at a high temperature. The low temperature of the soil achieved by preliminary cooling protects the foundation soil from thawing for 5 - 7 days during which the concrete is heated. If there is a short warm spell before the concrete is poured the effect of preliminary cooling is lost after 2 - 3 days.

Resistance of support foundation posts to heaving. Resistance of the supports to heaving during the time of construction is ensured if

$$N + T_s > T_h, \tag{6.43}$$

where N - weight of foundation plus weight of that portion of the building finished by the time the ground begins to freeze;

T_h - tangential forces acting on the adfreezing surface between the foundation and the seasonally freezing layer of soil;

T_s - tangential forces that have arisen at the adfreezing surface between the foundation and the permafrost.

In designating the time of the year when the work is to be carried out, condition (6.43) must be satisfied. If this schedule is not maintained more radical temporary measures lasting for one or two winters may be taken to reduce the forces T_h. This is achieved by coating the supports with a mastic* or other frost-resistant substances, or by artificial local heating of the supports providing for a temporary weakening of the adfreezing bond between the supports and the soil. On completion of the building and bringing the load on the supports up to the design value, the need for these temporary measures disappears.

In checking the stability of supports with the use of equation (6.43), it should also be kept in mind that the freezing of supports into the permafrost, ensuring the calculated value of the tangential holding forces T, is due partly to heat exchange with the permafrost and partly due to winter freezing. A useful effect of the heat exchange can be expected if the temperature of the permafrost is of the order of -3°C or lower. Judging from experience in construction in Yakutsk under such conditions one can expect that the force T_s will reach the calculation value in 1 - 3 months after backfilling of the supports. Under other conditions the calculation value of T_s can be reached only after one or two winters (for example in Igarka).

Regardless of the method, the lateral surface of the foundations in the seasonally thawing layer should be smoothed carefully to reduce the heaving forces.

Final operations in building foundations and levelling the surface around structures. Finishing operations in constructing foundations consist in (a) backfilling the excavation, (b) general grading of the surface around the structure and (c) the construction of protective decking around the outside perimeter of the building. Neglecting these operations leads to a decrease in strength and stability of the structure and is frequently the reason for serious deformations.

Backfilling. The purpose of backfilling after completion of the foundation work is to reestablish, in correspondence with design requirements, the ground surface around the structure that was disturbed during the building operation. A basic engineering requirement is careful compaction of the soil to eliminate settlement after backfilling.

Backfilling excavations in the permafrost region have the following features determined by permafrost climatic conditions:

(a) the unavoidability in some cases of backfilling with frozen soil;

(b) the unsuitability of some soils, particularly saturated silt soils, for backfilling excavations and the necessity of bringing substitutes;

(c) the necessity of carrying out antiheaving measures.

---

* B.I. Dalmatov suggests a bitumenous mastic consisting of four parts ash (waste product of thermal electric stations), three parts bitumen mark III and one part solar oil.

In carrying out foundation work during the summer, thawed dried soil for backfilling should be prepared in good time.

If the backfill is to consist of the soil removed from the excavation, it should be kept in piles as small as possible to accelerate drying and thawing. Piles of soil intended for backfill should be protected by ditches or earth walls to drain off water.

In winter the backfill should be either thawed or the frozen soil should be broken into lumps 2 - 5 cm in diameter and the frozen lumps should be mixed with sand. Each layer of frozen soil not more than 10 - 15 cm thick should be covered with a layer of dry sand which, by tamping, fills the cavities between the lumps of frozen soil as much as possible. Corresponding to the volume of the cavities in the frozen soil the volume of sand should be not less than 30 - 50% of that of the frozen soil.

Final grading. Final grading of the surface around the structure, after the construction debris and remains of building material have been removed to provide surface drainage, is of prime importance and does not differ from the usual practice.

Decking. Under permafrost conditions, decking serves not only to drain off surface water but in some cases to reduce the ground temperature; the insulating decking shown in Fig. 37 serves to reduce the depth of summer thawing and bars the movement of ground water above the permafrost table.

For the insulating decking to best serve the above purpose, it is desirable to install it at the beginning of the warm season, otherwise its full effect will be obtained only during the following year.

The organization of foundation work on perennially frozen soil. In carrying out construction on permafrost, an accelerated rate of operation can give advantages which are at the present time not fully appreciated by builders. Hence follows the necessity and efficiency of using mechanized methods and modern procedures enabling uninterrupted operations. Very important advantages are gained by using prefabricated reinforced concrete foundations prepared at a suitable factory rather than in the excavation.

With mechanized drilling devices one can reduce earthwork to drilling holes in the permafrost for wooden, reinforced concrete or metal piles, and reduce the time taken up for foundation work to a minimum regardless of the time of year; with such operations there is no disturbance in the natural regime of the foundation soil.

High-speed methods of construction consisting in careful preparation of phases of the work reducing the time consumed to a minimum are applicable to foundation work in the permafrost region. The efficiency of mechanized and high-speed methods of operation is particularly effective in erecting large structures.

Shift work in most cases is efficient.

In addition to the advantages of fast operations by mechanized methods considerable advance can be attained by the intelligent use of natural forces such as solar radiation on the one hand and the low winter temperatures on the other, depending on the method of construction adopted (Chapters IV and V).

The organization of measures to increase the productivity of labour as applied to foundation work in the permafrost region should include a continuous improvement in the technical knowledge of all personnel involved from the supervisors to the labourers and familiarizing them at least with the elementary information concerning permafrost engineering.

Literature

Abelev, Yu.M. Basic principles of planning and construction on macroporous ground. Stroivoenmorizdat, 1948.

Bakulin, F.G. The ice content and ground subsidence on thawing of permafrost in the Vorkuta Raion. Manuscript. Fondy in-ta merzlotov. AN SSSR, 1953. Published in Problems of Road Construction in Permafrost Regions. UNIIS Mintransstroya, 1958.

Baturin, V.V. and Kucheruk, V.V. Aeration of industrial building. ONTI, 1937.

Chernigov, V.A. Instructions and recommendations for use of concrete in foundations on permafrost (project). Tr. in-ta merzlotov. AN SSSR, Vol. 14, 1958.

Egerev, K.E. Electric method of determining the tangential reactions distributed along the lateral surface of a pile subjected to a load and frozen into ground. Tr. in-ta merzlotov. AN SSSR, Vol. 14, 1958.

Fedosov, A.E. Analysis of settlement of structures on thawing of foundation soils. Tr. in-ta merzlotov. AN SSSR, Vol. 4, 1944.

Gorbunov-Posadov, M.I. Subsidence of foundations on a soil layer resting on rock. Gos. izdat. stroit. literatury, 1946.

Lapkin, G.I. Calculation of settlement of structures on thawing soils by the method of contact pressures. Izv. AN SSSR, OTN, No. 3-4, 1943.

Maksimov, G.N. Construction and operation of a field station for the study of permafrost. Paper read at the interdepartmental discussion on permafrost studies, 1956.

Pchelintsev, A.M. Tangential stresses on lateral surfaces of foundations resulting from thawing of foundation soils. Materialy k osnovam ucheniya o merzlykh zonakh zemnoi kory, No. III, Izd-vo AN SSSR, Moscow, 1956.

Podvyazkin, K.A. Electrochemical strengthening and electrical drainage of soils. Stroitel'stvo dorog, No. 5, 1948.

Porkhaev, G.V. Design of ventilating basements in buildings erected by the method involving the retention of permafrost. Tr. in-ta merzlotov. AN SSSR, Vol. 11, Izd-vo AN SSSR, 1952.

Porkhaev, G.V. and Bondarev, P.D. Changes of upper surface of permafrost under embankments. Materialy k osnovam ucheniya o merzlykh zonakh zemnoi kory, No. 4, 1957. Izd-vo AN SSSR.

Porkhaev, G.V. and Sarkisyan, R.M. Effect of convectional heat exchange in the layer saturated with water on the thawing of underlying permafrost. Manuscript. Fondy in-ta merzlotov. AN SSSR, 1957.

Rzhanitsyn, B.A. Methods of strengthening soils and rocks. Transzheldorizdat, 1937.

Saltykov, N.I. Settlement of a building erected on coarse foundation soil. Tr. in-ta merzlotov. AN SSSR, Vol. 11, 1952-b.

Saltykov, N.I. Brief instructions for designing foundations on thawing soil by considering their settlement and reactions between foundation soils and the base of foundation. Izd-vo AN SSSR, 1953.

Saltykov, N.I. and Saltykova, N.N. Thermo-technical design of ventilated basements. In: Issledovanie vechnoi merzloty v Yakutskoi respublike, No. 2, Izd-vo AN SSSR, 1950.

Vyalov, S.S. and Tsytovich, N.A. Determination of bearing capacity of cohesive soils by impressing a spherical puncher. DAN SSSR, 3 (6), 1956.

Yakovlev, V.I. Design of wooden foundations of residential and administrative buildings in permafrost regions based on experience gained in Igarka. Izd-vo AN SSSR, 1952.

Table XXIX

Maximum dimensions of heated buildings which do not require cooling arrangements to retain the foundation soil in a frozen state, in metres

θ_{NG}	θ_0	R_{f1} \ θ_1	5	10	15	20	30
−50	−10	1	15×10	10×20	5×10	—	—
		2	30×60	20×40	10×20	5×10	—
		3		30×60	20×40	10×20	5×10
−25	− 5	1	5×10	—	—	—	—
		2	20×40	10×20	5×10	—	—
		3	30×60	20×40	10×20	5×10	—
−10	− 2	1	3×6	—	—	—	—
		2	7×14	5×10	—	—	—
		3	10×20	7×14	5×10	—	—

Table XXX

Mean monthly temperature, relative humidity and air density, ground surface temperature, wind velocity in the vicinity of the building site

MONTH	AIR TEMP. °C	TEMP. OF DEW POINT OF OUTSIDE AIR	REL. HUM. OF AIR	AIR DENSITY kg/m³	GROUND SURF. TEMP. °C	WIND VELOCITY m/SEC.
I	−43,2	—	80,4	1,41	−42,0	1,1
II	−35,6	—	82,0	1,40	−36,0	1,1
III	−22,4	—	77,4	1,38	−23,0	1,1
IV	− 8,1	—	66,6	1,33	− 7,3	2,1
V	5,6	− 1,0	59,6	1,27	—	2,5
VI	15,5	7,2	60,8	1,23	—	2,2
VII	18,9	12,0	64,6	1,22	—	2,1
VIII	14,7	10,0	73,4	1,22	—	2,0
IX	6,1	3,0	74,6	1,27	—	2,0
X	− 7,9	—	81,6	1,33	− 6,9	1,9
XI	−27,5	—	83,1	1,37	−26,4	1,5
XII	−39,5	—	81,0	1,41	−38,0	1,1

The temperature in the crawl space is regulated monthly.

Table XXXI

Module of ventilation of crawl space
for different widths of the building, thermal
resistance of the floor and floor
surface temperature in the first storey
(calculated without taking wind pressure into account)

WIDTH OF BLDG. M	THERMAL RESIST. OF FLOOR M² HR·DEG./K/CAL	TEMP. OF FLOOR OF FIRST STOREY, DEG.	VENTILATION MODULE OF THE CRAWL SPACE FOR THE ZONE:		
			SUBARCTIC	TEMPERATE	SOUTHERN
15	1	15	0,0015	0,005	0,025
		30	0,005	0,015	0,03
	2	15	0,001	0,003	0,012
		30	0,002	0,007	0,020
	3	15	0,0005	0,002	0,007
		30	0,001	0,004	
30	1	15	0,0025	0,008	OPEN
		30	0,0075	0,025	»
	2	15	0,0015	0,006	0,015
		30	0,0035	0,010	0,03
	3	15	0,0007	0,003	0,007
		30	0,0022	0,005	0,010
50	1	15	0,003	0,010	OPEN
		30	0,01	0,03	»
	2	15	0,002	0,007	0,020
		30	0,005	0,015	OPEN
	3	15	0,001	0,004	0,10
		30	0,003	0,007	OPEN

Table XXXII

Calculation of the thaw basin under a heated structure

x	y	$f(\xi_1)$	$f(\xi_2)$	A_1	A_2	Y	X_1	X_2	B_1	B_2	x	y	Y	X_1	X_2	B_1	B_2	F_2	y
						First Room					Second Room								
9	18	0,35	0,35	−2,35	−1,40	10,0	+0,55	+ 5,0	+0,195	+1,65	9	18	+7,5	−3,75	+0,416	−2,8	0,32	+0,82	
	20	0,36	0,36	−2,4	−1,45	11,1	+0,55	+ 5,0	+0,175	+1,5		20	8,34	−3,75	+0,416	−2,5	0,30	+0,27	22
	22	0,37	0,37	−2,5	−1,50	12,2	+0,55	+ 5,0	+0,16	+1,35		22	9,1	−3,75	+0,416	−2,1	0,29	−0,42	
18,5	15	0,23	0,511	−1,55	−2,0	8,34	+5,83	+10,3	+2,25	+3,1	18,5	15	6,25	+0,208	+4,4	+0,19	+3,6	+0,70	
	16	0,235	0,510	−1,60	−2,0	8,9	+5,83	+10,3	+2,1	+3,0		16	6,67	+0,208	+4,4	+0,17	+3,4	+0,50	18,5
	18	0,25	0,509	−1,70	−2,0	10,0	+5,83	+10,3	+1,9	+2,6		18	7,5	+0,208	+4,4	+0,15	+3,0	−0,15	
−1	10	0,533	0,17	−3,55	−0,7	5,55	−5,0	−0,55	−2,6	−0,31	−1	10	4,16	−7,9	−3,75	−6,5	−4,1	+0,34	
	12	0,528	0,19	−3,5	−0,8	6,67	−5,0	−0,55	−2,3	−0,25		12	5,00	−7,9	−3,75	−6,0	−3,7	−0,5	12,4
	2	0,65	0,035	−4,7	−0,14	1,11	−5,0	−0,55	−4,5	−1,25		2	0,83	−7,9	−3,75	−8,7	−7,0	+0,20	
	1	0,75	0,017	−5,2	−0,07	0,55	−5,0	−0,55	−5,0	−2,00		1	0,415	−7,9	−3,75	−9,0	−7,7	+0,03	1,0

Note

1. The line Y = 6.67 (Fig. 52) should be extrapolated to the intersection with X = 0.208. Calculation with respect to the nomogram is carried out in the usual manner.
2. When x = 0 and l_n - x = 0 the logarithmic nomograms are inapplicable.

Table XXXIII

Depth of thaw basin h
(corresponding to Fig. 54)

No. of area	x, cm	y, cm	h, cm	No. of area	x, cm	y, cm	h, cm
1	—380	+200	275	7	+ 20	0	300
2	—180	+200	260	8	+220	0	250
3	+ 20	+200	250	9	—380	—200	475
4	220	+200	220	10	—180	—200	475
5	—380	+ 0	375	11	+ 20	—200	400
6	—180	0	350	12	+220	—200	275

Table XXXIV

Auxiliary expressions
for calculating the values of L, M, N

No. of area	uh, kg	uh^2, kg·cm	uhx, kg·cm	uh^2x, thous. kg·cm²	uhy, kg·cm	uh^2y, thous. kg·cm²
1	259,46	71352	— 98595	— 27114	+51892	+14270
2	255,32	66383	— 45958	— 11949	+51064	+13277
3	252,63	63158	+ 5053	+ 1263	+50526	+12632
4	244,90	53878	+ 53878	+ 11853	+48980	+10776
5	290,91	109091	—110546	— 41455	0	0
6	282,35	98822	— 50823	— 17788	0	0
7	266,67	80001	+ 5333	+ 1600	0	0
8	252,63	63158	+ 55579	+ 13895	0	0
9	331,03	157239	—125791	— 59751	—66206	—31448
10	331,03	157239	— 59585	— 28303	—66206	—31448
11	300,00	120000	+ 6000	+ 2400	—60000	—24000
12	257,46	71352	+ 57081	+ 15697	—51892	—14270
Σ	3326,39	1111673	—308374	—139652	—41842	—50211

Because the values of A and a are constant it is possible to simplify the calculation: instead of determining Σ Auhx and Σ auhx it is sufficient to calculate Σ auhx and multiply it by A and a, respectively. All values in the table contain

$$u = \frac{2h_s \, \Delta\omega}{ah\left(2h_s - \frac{h}{2}\right)}, \text{ in which } \Delta\omega = 4 \cdot 10^4 \, m^2$$

is a constant. This makes it possible to carry the value $\Delta\omega$ outside the summation sign

Table XXXV

Coefficients of s_o, tg α_x, tg α_y in equation (6.33)

No. OF AREA	a, kg/cm	ax, kg	ay, kg	ax^2, THOUS. kg·cm	ay^2, THOUS. kg·cm	axy, thous. kg·cm
1	0,9435	−358,53	+188,70	136,24	34,74	−71,71
2	0,9820	−176,76	+196,40	31,82	39,28	−35,35
3	1,0105	+ 20,20	+202,10	0,404	40,42	+ 4,04
4	1,1132	+244,90	+222,64	53,876	44,53	+48,98
5	0,7757	−294,77	0	112,01	0	0
6	0,8067	−145,21	0	26,14	0	0
7	0,8889	+ 17,78	0	0,356	0	0
8	1,0105	+222,31	0	48,91	0	0
9	0,6969	−264,82	−139,38	100,63	27,88	+52,96
10	0,6969	−125,44	−139,38	22,58	27,88	+25,09
11	0,7500	+ 15,0	−150,0	0,3	30,0	− 3,0
12	0,9435	+207,57	−188,70	45,67	37,74	−41,51
Σ	10,618	−637,76	+192,32	578,936	285,47	−20,5

Table XXXVI

Calculation of reaction of the foundation soil per unit area of foundation footing Q = gxy · 4 · 10^4

No. OF AREA	Ah, cm	$h\left(h_f + \frac{h}{2}\right)$, cm²	$a\gamma\left(h_f + \frac{h}{2}\right)h$, cm	$x \operatorname{tg} \alpha_x = -0{,}00102x$, cm	$y \operatorname{tg} \alpha_y = -0{,}0165y$, cm	[ ]*, kg/cm²	Q = [ ] a·4·10^4, kg
1	4,125	106562,5	0,959	−0,3876	+3,3	0,944	35620
2	3,90	99800	0,889	−0,1836	+3,3	1,03	40458
3	3 75	93750	0,844	+0,0204	+3,3	1,03	41633
4	3,30	79200	0,713	+0,2244	+3,3	1,40	62339
5	5,625	164062	1,477	−0,3876	0	2,23	69192
6	5,25	148750	1,339	−0,1836	0	2,53	81638
7	4,50	120000	1,080	+0,0204	0	3,34	118757
8	3,75	93750	0,844	+0 2244	0	4,12	166530
9	7,125	231562,5	2,084	−0,3876	−3,3	3,42	95336
10	7,125	231562,5	2,084	−0,1836	−3,3	3,22	89789
11	6,00	180000	1,62	+0,0204	−3,3	4,60	138000
12	4,125	106562,5	0,959	+0,2244	−3,3	6,93	261538
							Σ Q=1200830

*THE SYMBOL [ ] HERE AND BELOW IN THE TABLES DENOTES THE EXPRESSION

$$\left[s_0 - Ah - a\gamma\left(h_f + \frac{h}{2}\right)h - x \operatorname{tg} \alpha_x - y \operatorname{tg} \alpha_y\right]$$

Table XXXVII

Data of depth of thaw h

No. OF AREA	x, cm	y, cm	h, cm	No. OF AREA	x, cm	y, cm	h, cm
1	−225	+150	190	8	+175	−150	300
2	−125	+150	200	9	+ 75	−150	380
3	− 25	+150	210	10	− 25	−150	500
4	+ 75	+150	250	11	−125	−150	475
5	+175	+150	310	12	−225	−150	400
6	+175	+ 50	300	13	−225	− 50	325
7	+175	− 50	290	14	−225	+ 50	245

Table XXXVIII

Reaction of the
foundation soil per unit area

No. OF AREA	$\Delta h = 0{,}02h$, cm	$h_f + \frac{h}{2}$, cm	$\left(h_f + \frac{h}{2}\right)h$, cm²	$\alpha T\left(h_f + \frac{h}{2}\right)h$, cm	$x \operatorname{tg} \alpha_x = -0{,}00156x$, cm	$y \operatorname{tg} \alpha_y = -0{,}0158y$, cm	ц, cm	$q = u$ц, kg/cm²	$Q = u$ц$\cdot 10^4$, kg
1	3,80	345	65550	0,5899	+0,351	+2,366	1,479	1,80	18012
2	4,00	350	70000	0,630	+0,195	+2,366	1,395	1,63	16273
3	4,20	355	74550	0,671	+0,039	+2,366	1,310	1,47	14678
4	5,00	375	93750	0,844	−0,117	+2,366	0,493	0,48	4802
5	6,20	405	125550	1,130	−0,273	+2,366	−0,837	−0,69	− 6939
6	6,00	400	120000	1,080	−0,273	+0,779	1,000	0,85	8483
7	5,80	395	114550	1,031	−0,273	−0,779	2,807	2,44	24410
8	6,00	400	120000	1,080	−0,273	−2,366	4,145	3,52	35162
9	7,60	440	167200	1,505	−0,117	−2,366	1,964	1,42	14186
10	10,00	500	250000	2,250	+0,039	−2,366	−1,337	−0,83	− 8319
11	9,50	487	231325	2,081	+0,195	−2,366	−0,824	−0,52	− 5250
12	8,00	450	180000	1,620	+0,351	−2,366	0,981	0,69	6867
13	6,50	412,5	134062	1,207	+0,351	−0,779	1,307	1,05	10472
14	4,90	372,5	912624	0,821	+0,351	+0,779	1,735	1,72	17164
									$\Sigma Q = 150004$

Table XXXIX

Reaction of the
foundation soil per unit area

No. OF AREA	[ ], cm	$Q-[\ ]\times 10^4$, kg	$Q_{CORRECTED}$ kg	q, kg/cm²
1	1,44	17530	17740	1,774
2	1,33	15515	15700	1,57
3	1,22	13670	13830	1,383
4	0,37	3600	3645	0,364
5	NEGATIVE	0	0	0
6	0,75	6360	6435	0,644
7	2,48	21566	21820	2,182
8	3,71	31470	31840	3,184
9	1,52	9580	9695	0,97
10	NEGATIVE	0	0	0
11	NEGATIVE	0	0	0
12	0,65	4550	4605	0,46
13	1,08	8650	8755	0,876
14	1,59	15750	15935	1,594
		148231	150000	

Table XL

Depth of thaw and coefficient A and a

No. OF AREA	x, cm	y, cm	h, cm	a, cm²/kg	A
1	−225	+150	190	0,0030	0,016
2	−125	+150	200	0,0037	0,017
3	− 25	+150	210	0,0045	0,018
4	+ 75	+150	250	0,0052	0,019
5	+175	+150	310	0,0060	0,020
6	+175	+ 50	300	0,0057	0,0207
7	+175	− 50	290	0,0053	0,0213
8	+175	−150	300	0,0050	0,022
9	+ 75	−150	380	0,0051	0,021
10	− 25	−150	500	0,0053	0,021
11	−125	−150	475	0,0054	0,020
12	−225	−150	400	0,0055	0,019
13	−225	− 50	325	0,0046	0,018
14	−225	+ 50	245	0,0038	0,017

Table XLI

Reactions of the foundation soil

No. OF AREA	Ah, cm	$a\tau\left(h_{f}+\frac{h}{2}\right)h$, cm	$x\ \mathrm{tg}\ \alpha_x = -0.0055x$, cm	$y\ \mathrm{tg}\ \alpha_y = -0.0202y$, cm	[ ], cm	$Q=\mu[\ ]10^4$, kg
1	3,04	0,354	+1,23	+3,03	0,846	17165
2	3,4	0,472	+0,69	+3,03	0,908	14128
3	3,78	0,604	+0,14	+3,03	0,946	11768
4	4,75	0,886	−0,41	+3,03	0,244	2262
5	6,2	1,356	−0,96	+3,03	−1,123	−7749
6	6,21	1,225	−0,96	+1,01	1,015	7592
7	6,18	1,099	−0,96	−1,01	3,191	26038
8	6,6	1,080	−0,96	−3,03	4,81	40788
9	7,98	1,544	−0,41	−3,03	2,416	17000
10	10,5	2,367	+0,14	−3,03	−1,476	−8723
11	9,5	2,247	+0,69	−3,03	0,907	−5360
12	7,6	1,782	+1,23	−3,03	0,918	5838
13	5,85	1,11	+1,23	−1,01	1,32	11510
14	4,17	0,624	+1,23	+1,01	1,466	19087
						$\Sigma Q=151344$

Table XLII

Reaction of foundation soil per unit area

No. OF AREA	[ ], cm	$Q=[\ ]\mu\cdot10^4$, kg	$Q_{CORRECTED}$ kg
1	0,796	16151	16115
2	0,838	13040	13013
3	0,846	10525	10503
4	0,114	1057	1055
5	NEGATIVE	0	0
6	0,735	5500	5488
7	2,831	23100	23050
8	4,350	36888	36812
9	1,986	13980	13950
10	NEGATIVE	0	0
11	NEGATIVE	0	0
12	0,568	3612	3605
13	1,070	9330	9311
14	1,316	17135	17098
		150318	150000

Table XLIII

Calculation of the depth of thaw with time, settlement, rate of settlement

Soil	t Time from beg. of thaw, day	Duration of interval: Δt, day	Duration of interval: Δt, hour	Δh, m	h, m	$1+k\sqrt{t}$	h_0, m	$A+pa$	Δs, mm	$v = \frac{\Delta s}{\Delta t}$, mm/day
Clay loam	0									
	100	100	2400	0,60	0,60	1,00	0,60	0,021	12,6	0,126
	200	100	2400	0,47	1,07	1,03	1,04	0,021	10,0	0,10
	400	200	4800	0,76	1,83	1,067	1,52	0,021	16,0	0,08
	600	200	4800	0,67	2,50	1,084	2,31	0,021	14,0	0,07
	800	200	4800	0,56	3,06	1,098	2,80	0,021	11,7	0,058
	916	116	2780	0,30	3,36	1,103	3,00	0,021	6,3	0,054
Sand	0									
	100	100	2400	0,75	0,75	1,0	0,75	0,0145	10,5	0,105
	200	100	2400	0,70	1,45	1,0	1,45	0,0145	10,0	0,100
	400	200	4800	1,1	2,55	1,13	2,3	0,0145	15,0	0,075
	600	200	4800	0,95	3,50	1,14	3,07	0,0145	12,6	0,063
	750	150	3600	0,65	4,15	1,16	3,5	0,0145	9,0	0,060
Clay loam	0									
	100	100	2400	0,2	0,2	1,0	0,2	0,021	4,2	0,042
	200	100	2400	0,15	0,35	1,0	0,35	0,021	3,1	0,031

Note

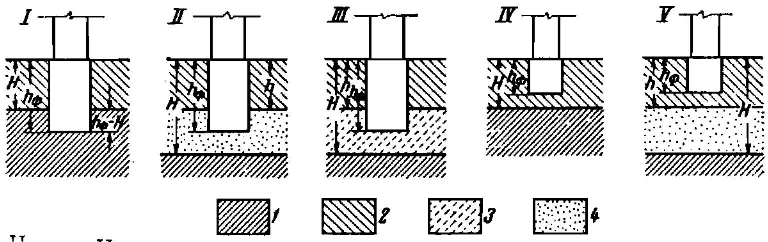
In determining the correction coefficient $1 + k\sqrt{t}$ the time in hours is counted from the beginning of the thawing of each layer (taken from column 1, Table XLIII)

Table XLIV

Values of the
coefficient k in formula (6.41)

$\frac{h_T}{b^*}$	CIRCLE (RADIUS)	Rectangles with the ratio of sides α			
		1	2	3	≥10
0,0	1,000	1,000	1,000	1,000	1,000
0,25	1,009	1,009	1,009	1,009	1,009
0,50	1,064	1,053	1,033	1,033	1,033
0,75	1,072	1,082	1,059	1,059	1,059
1,0	0,965	1,027	1,039	1,026	1,025
1,5	0,684	0,762	0,912	0,911	0,902
2,0	0,473	0,541	0,717	0,769	0,761
2,5	0,335	0,395	0,593	0,651	0,636
3,0	0,249	0,298	0,474	0,549	0,560
4,0	0,148	0,186	0,314	0,392	0,439
5,0	0,098	0,125	0,222	0,287	0,359

* b—half width of the foundation



$H_{\phi} = H_f$

Fig. 33

Standard schemes of
locating foundations with respect to frozen soil:

1 - soil that remains in a frozen state throughout the entire period that the structures are in operation; 2 - soil subject to seasonal freezing and thawing; 3 - soil that thaws when the structure is in operation; 4 - soil that remains unfrozen throughout the entire period the structure is in operation

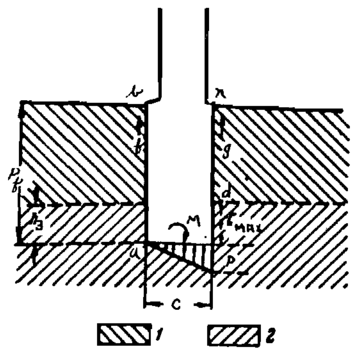


Fig. 34

A diagram of the distribution of forces when a tilting moment is acting on the foundation:

1 - soil subject to seasonal freezing and thawing; 2 - soil that remains in the frozen state throughout the entire period the structure is in operation

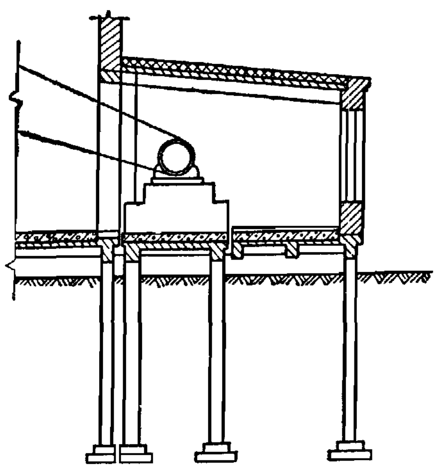


Fig. 35

An example of construction with a ventilated crawl space under heavy equipment. The equipment is supported on posts passed through the floor of the first storey

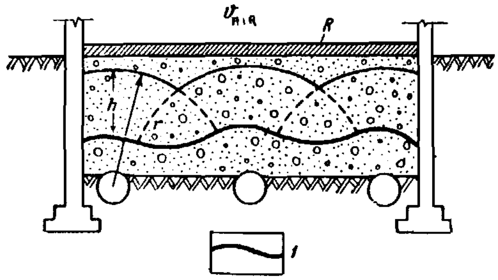


Fig. 36

Diagram of underground cooling of the foundation soil by ducts:

1 - permafrost table

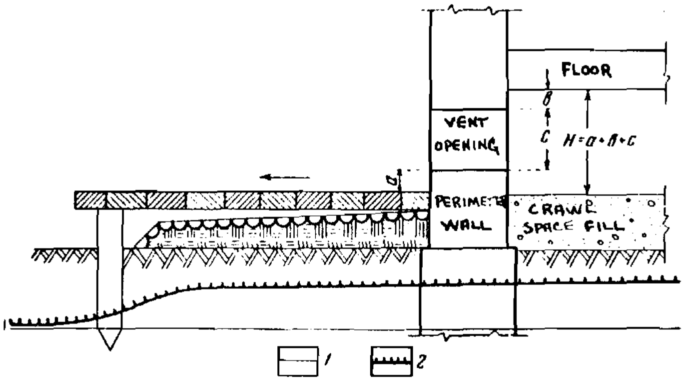


Fig. 37

Cooled crawl space, ventilation openings in the perimeter wall and insulating decking:

1 - permafrost table before construction
2 - the same after construction

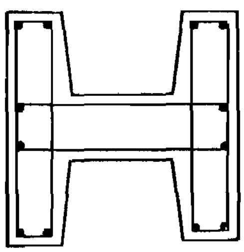


Fig. 38

Reinforced concrete pile with a double-T cross-section

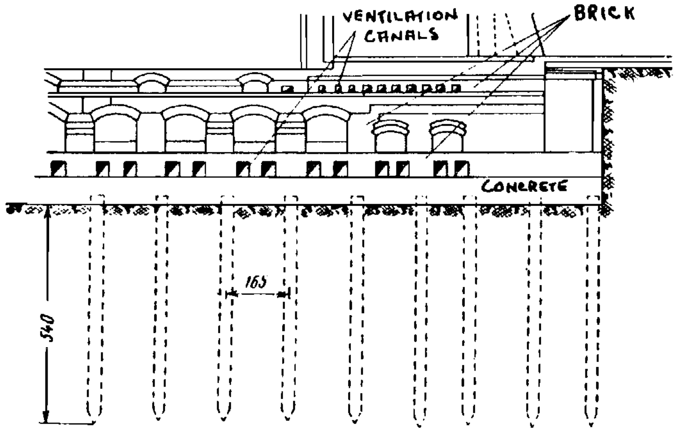


Fig. 39

Foundation of a Hofman furnace
at the Pokrovskii brick factory in
the Yakut USSR

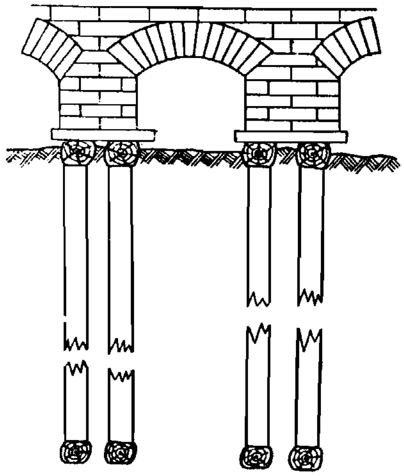


Fig. 40

Foundation of the floor level
furnace at the
Pokroskii brick factory

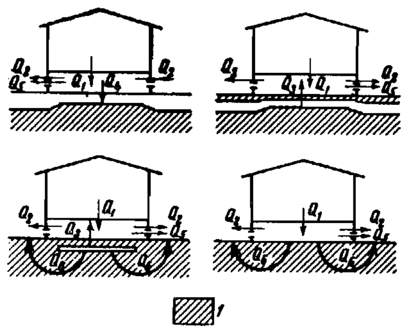


Fig. 41

Diagram of heat flow in a
ventilated crawl space
at various times of the year:

1 - frozen soil (arrows indicate
the direction of heat flow)

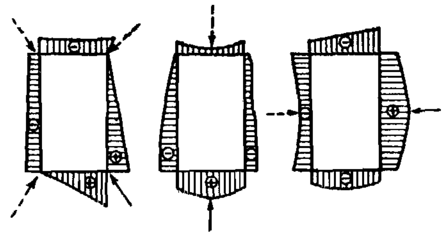


Fig. 42

Standard diagrams
of wind pressures against the
walls of a building

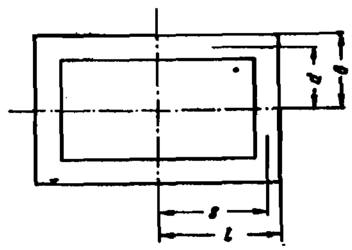


Fig.. 43

Diagram of ventilated floor
space in plan view (for calcula-
tion example)

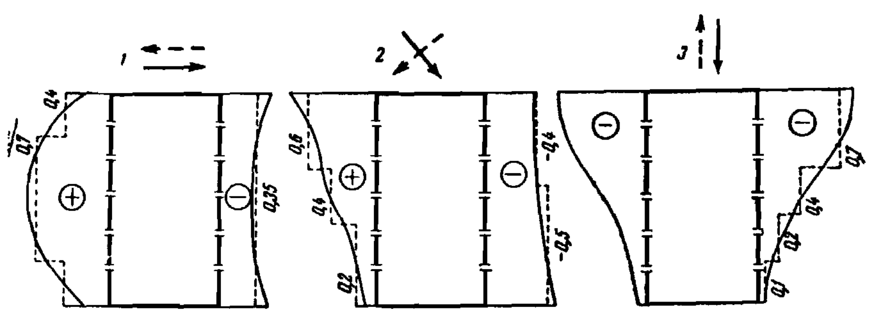


Fig. 44

Diagrams of wind pressures against the perimeter wall of the ventil-
ated crawl space for three wind directions
(example calculation). (After Baturin and Kucheruk)

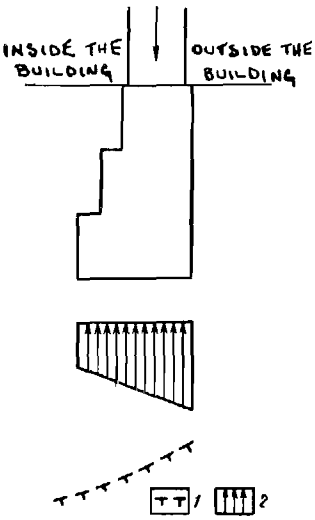


Fig. 45

Extension of the foundation in the direction of the greatest thawing of the foundation soil:

1 - permafrost boundary; 2 - reaction of foundation soil at the footing

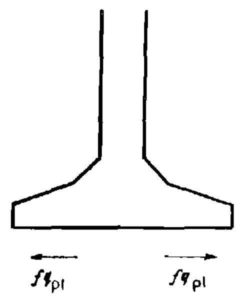


Fig. 46

Friction forces at location of plastic deformation of the soil

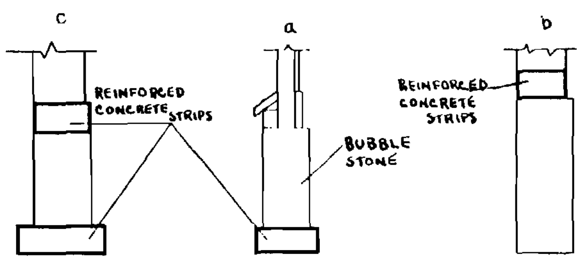


Fig. 47

Examples of concrete and rubble-stone concrete wall-type foundations strengthened by reinforced concrete strips

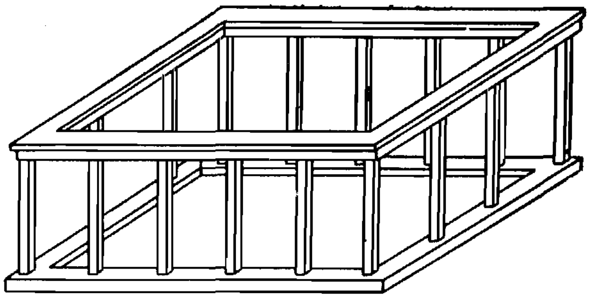


Fig. 48
Diagram of reinforced concrete strip foundation in the form of a frame

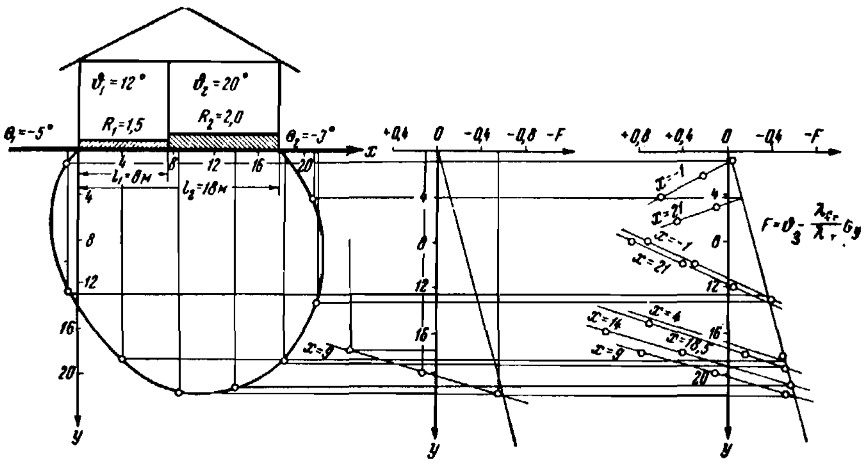


Fig. 49
Graphic construction of thaw basin under a heated building by the G.V. Porkhaev method (example calculation)

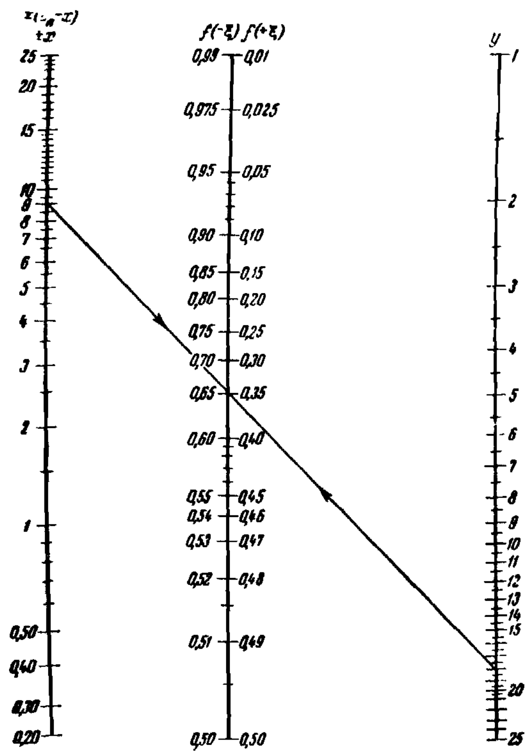


Fig. 50

Nomogram of
the function $f(\pm \xi) = \frac{1}{2} - \operatorname{arctg} \xi$.

Note

For positive values of x and l_n - x the reading is taken from the scale $f(+\xi)$, for negative values from the scale $f(-\xi)$.

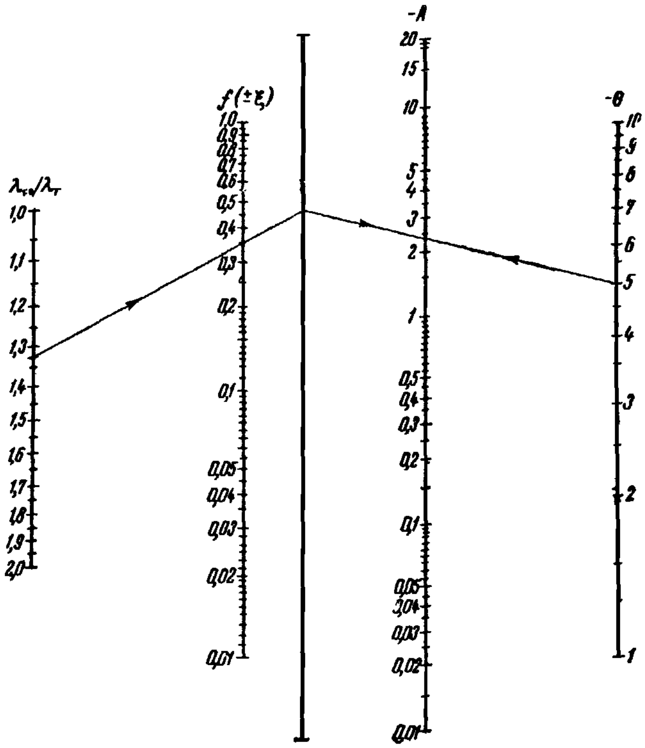


Fig. 51
Nomogram of function A

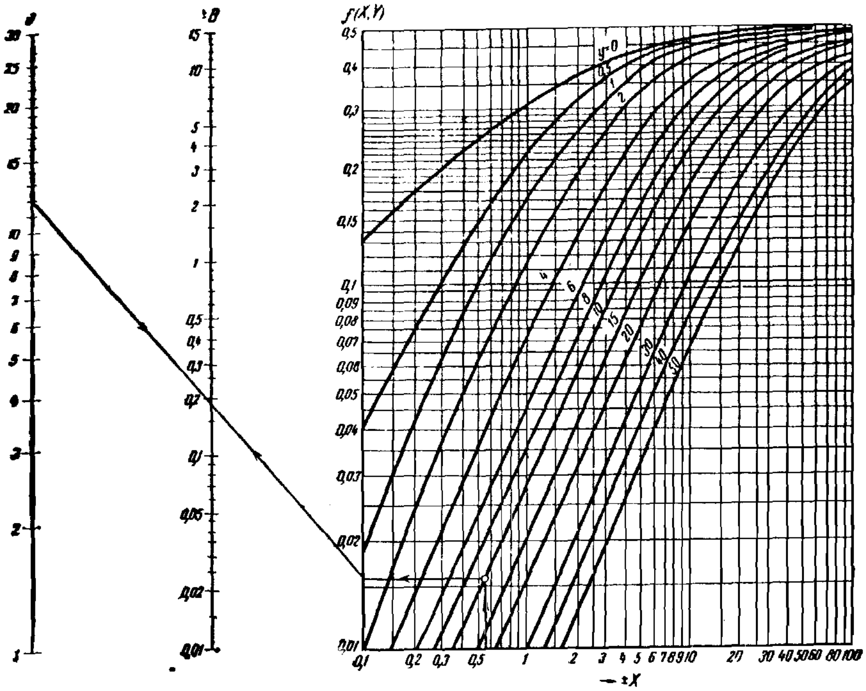


Fig. 52
Nomogram of function B

Note
Positive values of X correspond to positive values of B and negative to negative

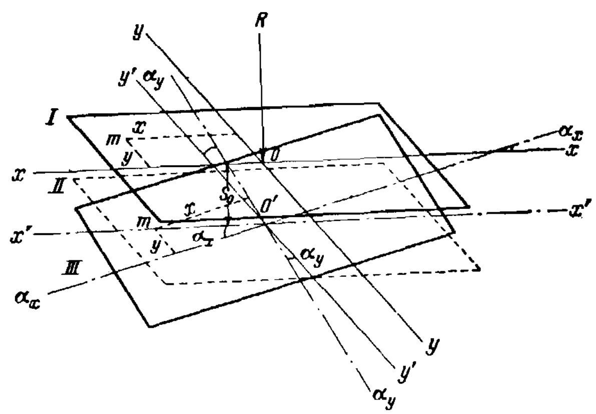


Fig. 53a
Diagram for calculating formula (6.25)

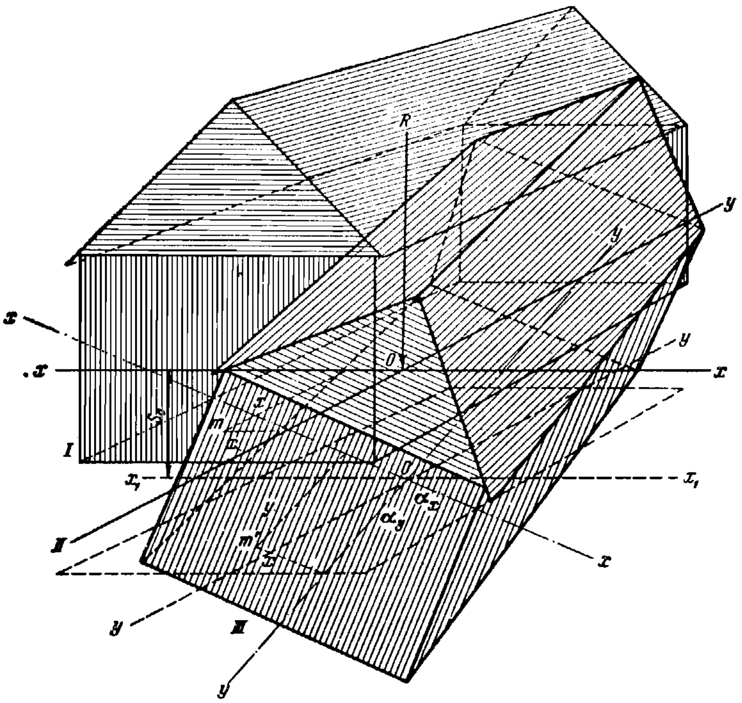


Fig. 53b
Diagram for calculating formula (6.25). Building is shown in axonometry

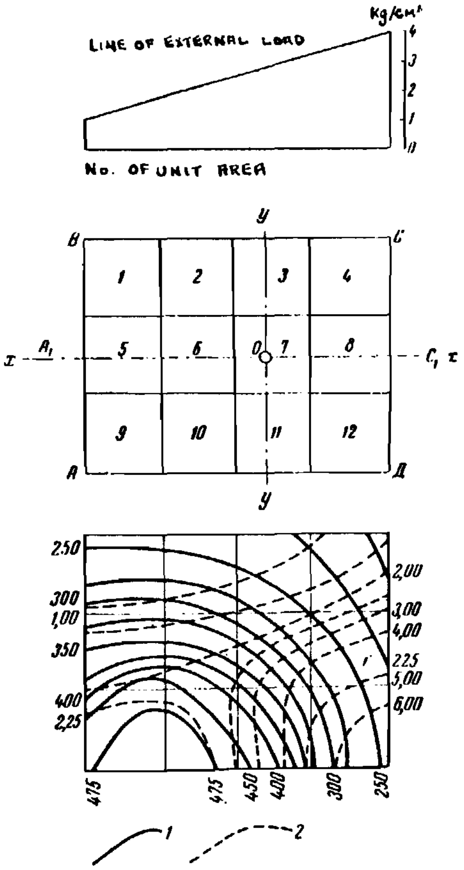


Fig. 54

Contours of thawing, distribution of load and reaction of the foundation soil:

1 - Contours of thawing surface in cm; 2 - lines of reaction of the foundation soil, kg/cm^2

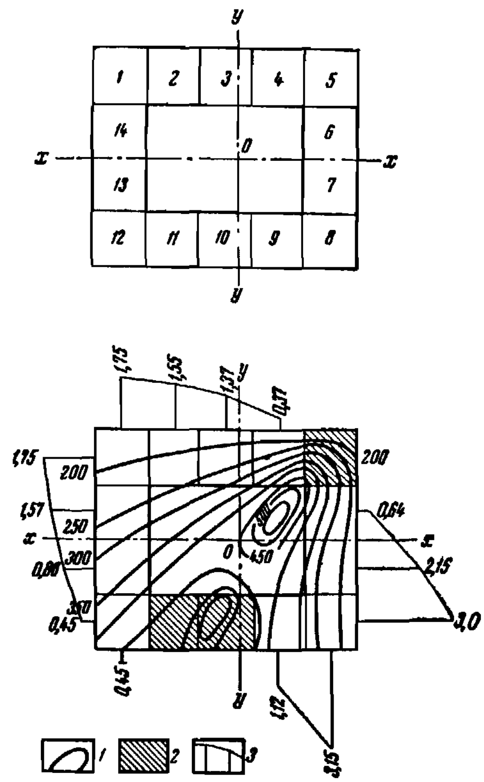


Fig. 55

For example 2

1 - Contours of thaw basin under a strip foundation; 2 - locations of zero reactions; 3 - diagrams of reactions of the foundation soil along the strip

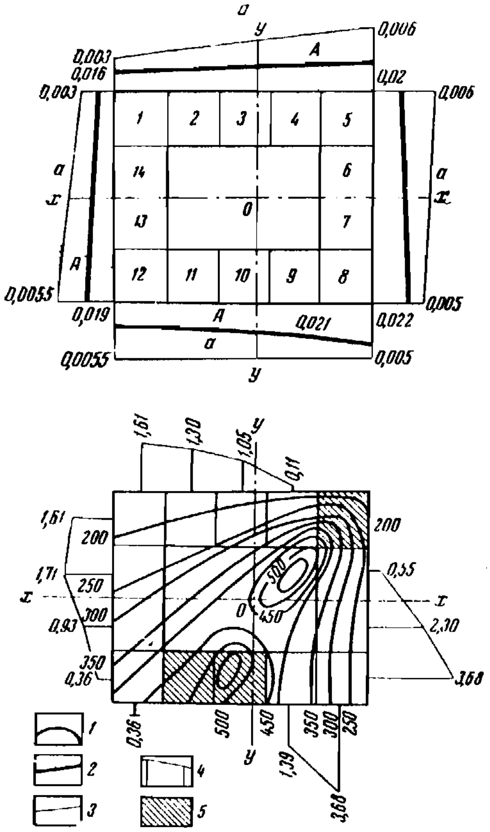


Fig. 56

For example 3

1 - isolines of thaw basin;
2 - value of thawing coefficient A; 3 - value of compaction coefficient a; 4 - diagrams of reactions of the foundation soil;
5 - regions of zero reactions

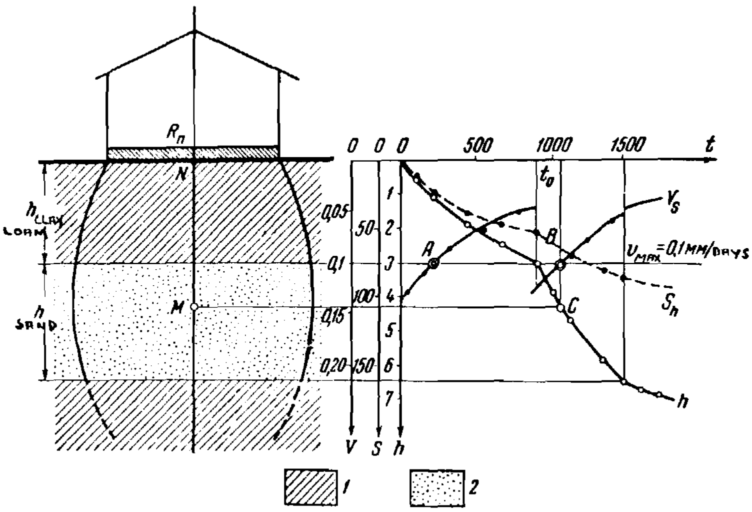


Fig. 57

For the example calculation of the depth of preconstruction thawing

1 - clay loam; 2 - sand

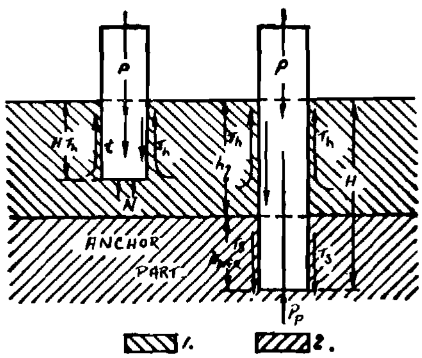


Fig. 58

Diagram of the action of heaving forces and forces counteracting heaving when the foundation is anchored in permafrost:

1 - seasonally freezing soil; 2 - permafrost

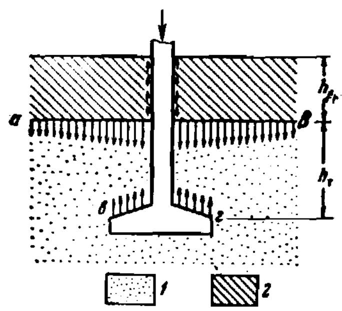


Fig. 59

Diagram of forces heaving and holding down the foundation when it is laid in thawed soil:

1 - unfrozen soil; 2 - seasonally freezing soil (after G.V. Porkhaev)