INVESTIGATION OF AIRFIELD DRAINAGE

ARCTIC AND SUBARCTIC REGIONS

PART II

TRANSLATION OF SELECTED TOPICS

by

St. Anthony Falls Hydraulic Laboratory
University of Minnesota

Project Report No. 17

Submitted by
Lorenz G. Straub
Director

Prepared by
Meir Pilch

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PREFACE TO ENGLISH TRANSLATIONS

Under contract W-21-018-eng-430, dated July 1947, between the University of Minnesota and the St. Paul District, Corps of Engineers, U. S. Army, the St. Anthony Falls Hydraulic Laboratory was to conduct a research project specified as "Research Investigations of Drainage of Airfields in Arctic and Subarctic Regions." A series of conferences held in September 1948 between the parties involved clarified the fact that the project was to include library research and translations of selected articles from Russian sources.

This report is a collection of translations of selected topics from three Russian sources and is composed of three parts. The first part, WATERPROOFING AND DRAINAGE OF DEFENSE AND NONDEFENSE STRUCTURES, is an exact translation; it includes six chapters, corresponding respectively to Chapters I, II, III, IV, VIII, and XIV of the original text. The second part, ABSTRACTS OF SCIENTIFIC RESEARCH WORK FOR 1945, Obruchev Institute of Frost Science, is an exact translation; it includes only that section of the original manuscript which contains the reports of the Institute of Frost Science. The third part, ICINGS AND COUNTERMEASURES, is a detailed abstract of the original text.

The three original Russian publications are as follows:

1. GIDROIZOLYATSIYA Y DRENAZH OBORONITELNIKH Y NEOBORONITELNIKH SO-ORUJENI by P. A. Bookreiev. Gos. Izdat. Stroy. Lit., Moscow-Leningrad, 1943, 124 pages, 83 figures, 29 tables.

2. REFERATI NAUCHNO-ISSLEDOVATELNIKH RABOT V 1945 G. Otdel Geol.-Geogr. Nauk, Akad. Nauk SSSR, Moscow, 1947.

3. NALEDI Y BORBA S NIMI by A. M. Chekotillo. Dor. Izdat. NKVD SSSR, Moscow, 1940, 133 pages, 70 figures, 8 tables.

The original texts were obtained on loan from the Stefansson Library by the St. Paul District, Corps of Engineers, and were reproduced photographically or on microfilm. The line drawings and photographic figures included in this report are subsequent reproductions modified dimensionally and in other respects to satisfy space and legibility considerations. The original arrangement of chapters and paragraphs as well as the numbering system for equations and formulas have been modified, but the original mathematical expressions and nomenclature have been retained.

The translations were made by Meir Pilch, Research Fellow, with the advisory assistance of Loyal A. Johnson. Alexander P. Rodionov assisted in the translation of part three, and Polly Canfield, Thomas Timar, Leona Schultz, and E. Roy Tinney assisted in preparation of the manuscript.

The work was administered by Dr. Lorenz G. Straub, Director of the St. Anthony Falls Hydraulic Laboratory, and generally supervised by Mr. Johnson, Research Associate and Supervisor of this project.

WATERPROOFING AND DRAINAGE OF DEFENSE AND NONDEFENSE STRUCTURES

by

P. A. Bookreiev

Government Publications of Construction Literature
Moscow - 1943 - Leningrad

LIST OF ILLUSTRATIONS

INTRODUCTION

Many structures erected in zones of aggressive ground water are periodically flooded by the ground water penetrating through the floors and the walls; this occurrence creates complications and occasionally makes it impossible to use the structure. Frequently structures are erected in regions having a high ground water table. To assure normal and uninterrupted use of buildings existing under these conditions, it is necessary to give particular consideration to problems pertaining to special waterproofing measures. It should be noted that the quality of the waterproofing depends not only on the kind and quality of the materials used but also, to a great extent, upon the quality of the workmanship.

This book contains directions for waterproofing of structures erected by both open and closed methods, and for lowering and diverting the ground water from the structures.

The author obtained the material for this book from a number of his previously published articles, from data of his construction experience, and from information furnished by various organizations that had recently completed waterproofing jobs. Since the Russian literature lacks any major works on problems of waterproofing, it may be presumed that the information contained in this book will help the engineers to cope with this rather complicated problem.

P. A. Bookreiev

CHAPTER I

CHARACTERISTICS AND ORIGIN OF GROUND WATER

A. Characteristics of Ground Water

The following types of ground water are classified in accordance with their origin and character of motion: perched, vein, artesian, and karst.

Perched water generally occurs in the upper layers of the ground, and often even in a portion of the cultured layer. This water has definite boundaries and occurs in the form of individual lenses. Formation of perched water is largely facilitated by unevenness of the relief of the locale and the surface of the impermeable layer; this unevenness impedes the flow and percolation of the surficial water into the lower layers. In most cases the perched water has no contact with the ground flow located below it, but it occasionally has a general hydraulic connection with this flow (Fig. 1).

Vein water, which is formed because of the complexity of the individual layers, occurs in clays and loams having veins of sand with percolation properties which differ from those of the surrounding impermeable ground. The vein water, which collects in separate basins and underground lakes, penetrates into the ground through the permeable veins (Fig. 2). Considering the relatively small cross-sectional area of the water-bearing layers, the vein water can be readily cut off, thus preventing inflow into the excavation.

Ground water is formed by atmospheric precipitations filtering through the soil, by waters from other water-bearing layers, by seepage of river and lake waters, and by condensation waters. Sloping of the impermeable bed causes the motion of the ground water which occasionally appears at the surface in the form of springs. An obstacle located in the path of the ground flow may create a pressure raising the level of the ground water. The water filling a basin in the contour of the impermeable bed does not have any definite motion, but the water occurring in shallow zones may find an outlet to the free surface along the capillaries.

Artesian water is ground water under pressure (Fig. 3). The magnitude of the pressure of this water depends mainly on the elevation of the water surface in the region feeding this water. In constructing deep tunnels, shafts, and similar structures, it is necessary occasionally to take into consideration the presence of artesian water.

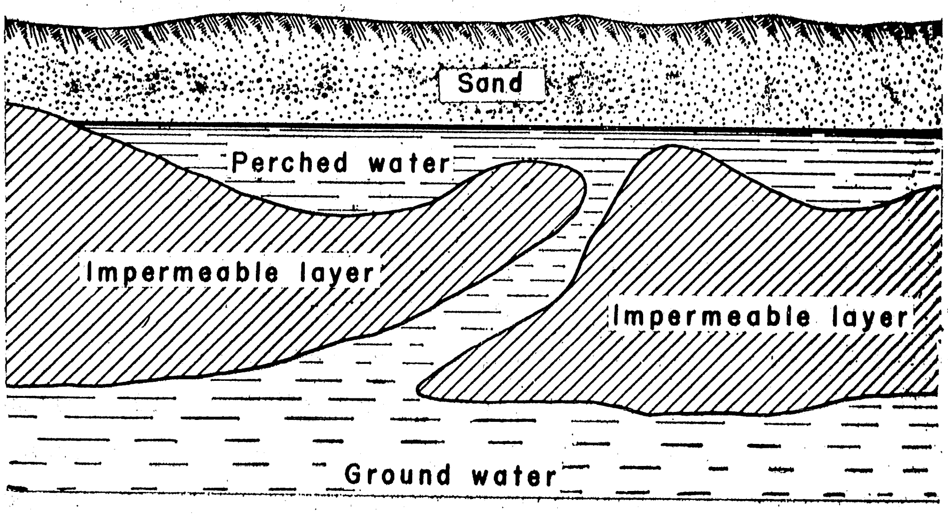


Fig. 1—Perched Water in Contact with Water Table

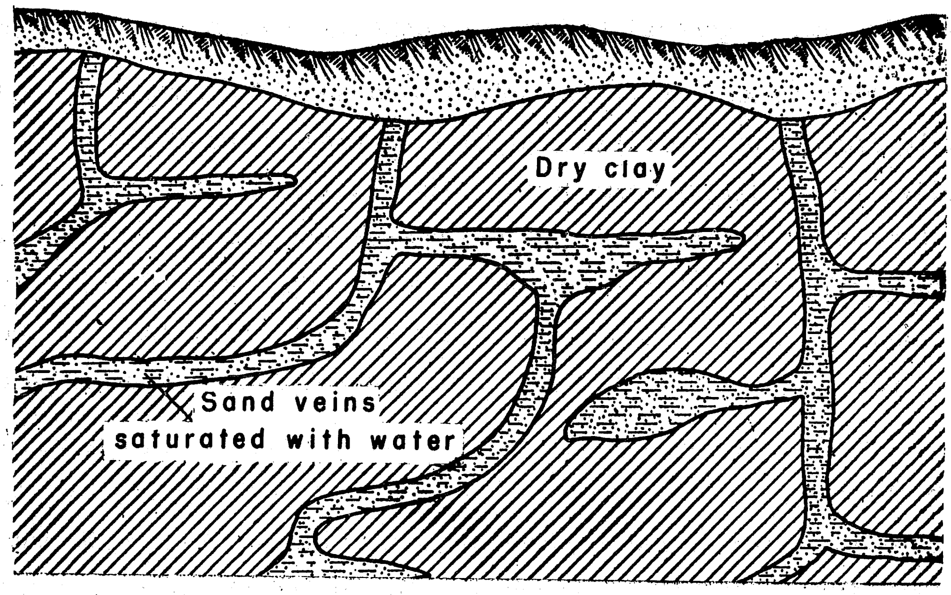


Fig. 2—Vein Water

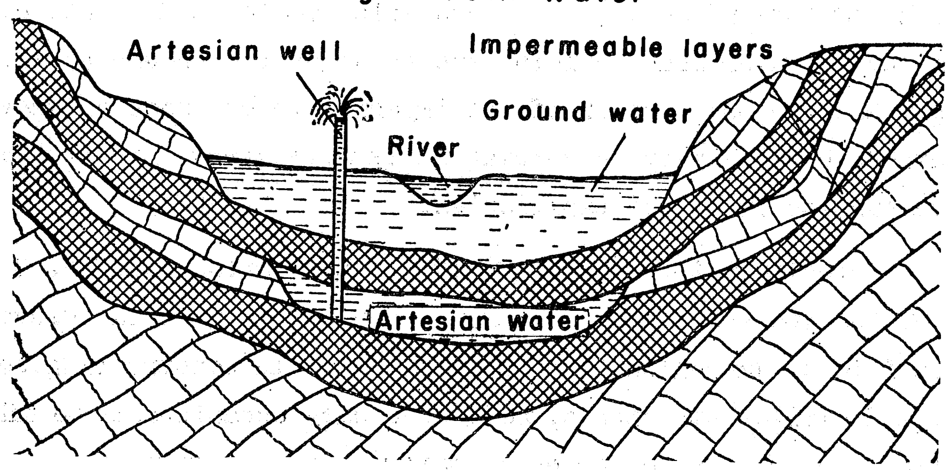


Fig. 3—Artesian Water

Karst water, occurring in karst formations having large fissures, is fed exclusively by atmospheric precipitations seeping through fissures which attained considerable width because of scour. In European USSR karst water occurs in the Crimea and near the Ural Mountains, and is seldom a factor in construction.

B. Percolation of Water in the Ground

The permeability of the ground, that is, the rate of percolation of water in the ground, is an important factor in construction engineering. The flow velocity of ground water is generally expressed by the formula

$$v = Ki^m \tag{1}$$

in which v is the velocity, i is the slope of the ground-stream surface, and K is the coefficient of percolation or permeability; K is constant for any given physical structure and condition of the ground.

The magnitude m depends on the flow conditions. If the water flows along large fissures and vacuums and the motion is turbulent in a manner similar to the motion of water in rivers and channels, then the flow is subject to Chezy's law and $m = 0.50$, while the magnitude K is subject to the laws of hydraulics and is determined from the relationship $K = c\sqrt{R}$.

If the ground flow proceeds along capillary spaces, as in sandy soil, the motion is characterized by uniform acceleration and is subject to Darcy's law, whereby $m = 1$ and

$$v = Ki \tag{2}$$

When $i = 1$, $v = K$; accordingly, the percolation coefficient is defined as the velocity of the ground flow at a slope $i = 1$ and has the same dimension as this velocity. In view of the fact that the velocity of ground flow is very small, it is usually expressed in terms of the distance traversed by the flow in one day; for example, $K = 0.50$ m per day.

C. Determination of the Percolation Coefficient

If the percolation coefficient is known, it is possible to solve such practical problems as the determination of loss of water in reservoirs due to seepage, discharge from wells and fissures, and magnitude of inflow into a basin. The value of the percolation coefficient can be determined in several ways: by mechanical analysis of the soil, by laboratory investigations, by test pumping from the fissure or bore in the field, and by direct measurements of the flow velocity of the ground water.

Preliminary and approximate evaluations can be made from the data of the mechanical analysis by applying Krüger's widely used formula (particularly applicable to sandy loam and to sands of small and medium grain size)

$$K = 1.44 \times 10^6 \frac{p}{\theta^2} \tag{3}$$

in which

K is the percolation coefficient in meters per day at $t = 18^\circ$ C,

p is the porosity of the ground, expressed as a fraction ($p < 1$), and

θ is the total surface area in square centimeters of the soil particles contained in 1 cu cm of the ground.

The objective of the laboratory mechanical analysis is to determine the granulometric composition of the ground, that is, the size distribution. In order to determine the relation between the magnitude θ of Krüger's formula and the data of the mechanical analysis, the following derivations are used. The total surface area of some portion of grains, assumed to be spherical in shape, is

$$\theta_1 = \pi d_1^2 n_1 \tag{4}$$

where d_1 is the mean diameter of the grains and n_1 is the number of particles in the given portion. The magnitude n_1 can be expressed by the formula

$$n_1 = \frac{v_1}{\pi \frac{d_1^3}{6}} \tag{5}$$

where v_1 is the volume of all the particles of the given portion and is expressed by the relationship

$$v_1 = g_1 (1 - p) \tag{6}$$

in which g_1 is the fraction of the given portion in the mechanical composition of the soil.

Making the proper substitution in formula (4) gives

$$\theta_1 = \frac{6g_1 (1 - p)}{d_1} \tag{7}$$

The corresponding result for any other sample is

$$\theta_2 = \frac{6g_2 (1 - p)}{d_2}$$

Since the total surface area of the soil particles is equal to the sum of the surface areas of the particles of all the portions, that is,

$$\theta = \theta_1 + \theta_2 + \ldots + \theta_n \tag{8}$$

the value of θ is

$$\theta = \frac{6g_1 (1 - p)}{d_1} + \frac{6g_2 (1 - p)}{d_2} + \ldots + \frac{6g_n (1 - p)}{d_n} \tag{9}$$

Introducing Eq. (9) into Krüger's formula and making the appropriate simplifications, the relationship in question assumes the form

$$K = 40{,}000 \frac{p}{(1 - p)^2} \left[\frac{1}{\frac{g_1}{d_1} + \frac{g_2}{d_2} \ldots + \frac{g_n}{d_n}} \right]^2 \tag{10}$$

The term enclosed in brackets in Eq. (10) denotes "the effective diameter of Krüger" (d_q). If d_q is expressed in millimeters, Krüger's formula takes the following form:

$$K = 400 \frac{p}{(1 - p)^2} d_q^{\,2} \tag{11}$$

Plotting the weights of the portions as abscissae and the g/d values of these portions as ordinates gives a family of straight lines for the various portions. These lines begin at the origin of coordinates (Fig. 4, the right side of the nomogram for Krüger's formula). In the case of the given nomogram,

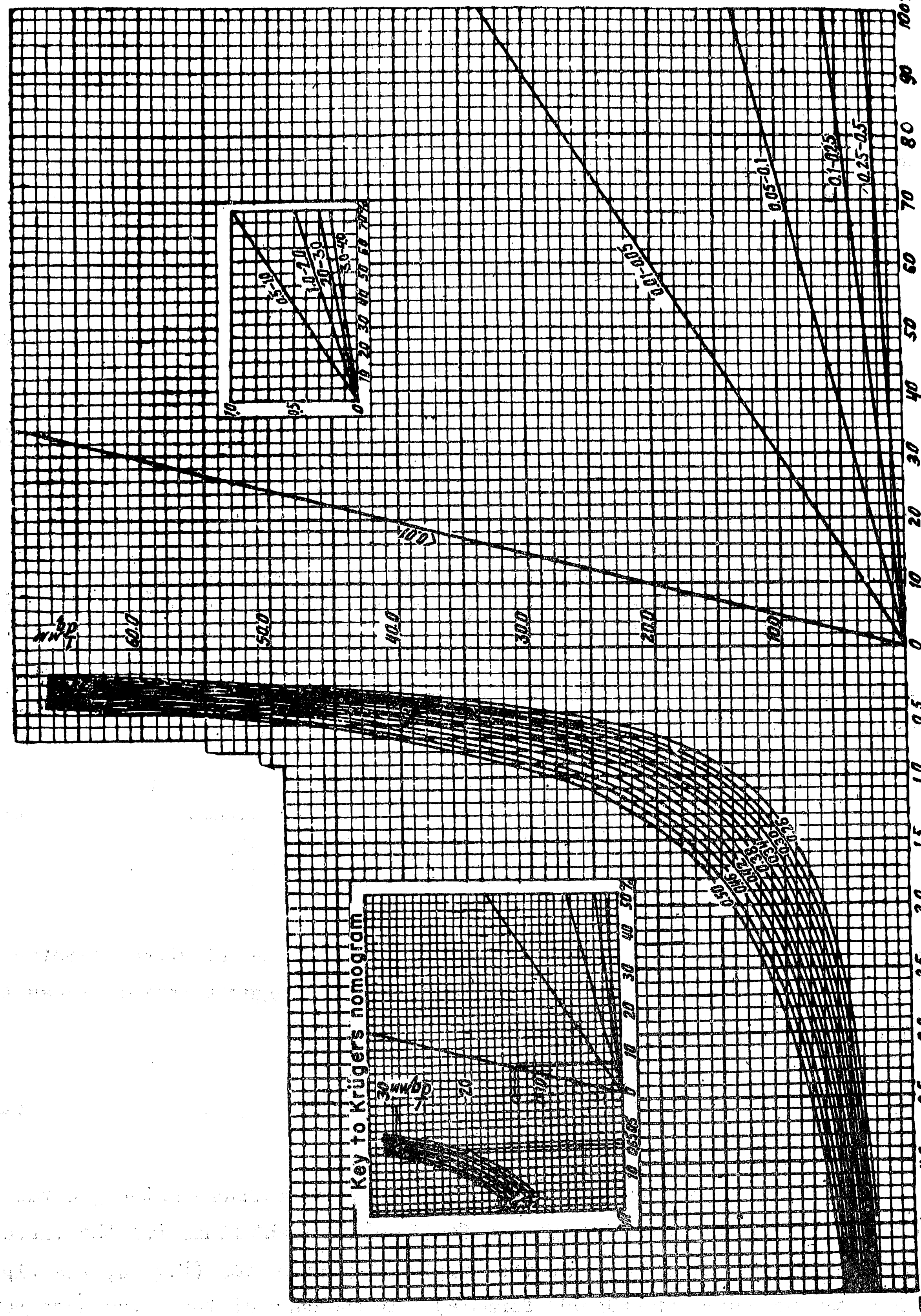


Fig. 4 – Nomogram for Determining K_ϕ from Krüger's Formula

use was made of the values ordinarily determined from mechanical analysis (Table 1) made by Salanin's method (also Salanin and Orlov) and by sieve analysis. This nomogram facilitates the determination of the magnitude $1/d_q$. Use of the nomogram is illustrated in Fig. 4.

TABLE 1

MECHANICAL ANALYSIS OF THE SOIL

Grain Size mm	Weight Per Cent
0 - 0.01	7
0.01 - 0.05	30
0.05 - 0.10	12
0.10 - 0.25	45
0.25 - 0.50	6
	100%

To determine the values of g/d for the individual portions, perpendiculars are erected from the points situated on the x-axis and showing the percentage weight of the portions to the intersection with the straight lines for the corresponding portions. The length of each perpendicular, computed according to the scale of the y-axis, gives the respective value of the magnitude g/d. The construction is shown in Fig. 4 by dotted lines (as seen in the key to the nomogram.)

TABLE 2

VALUES OBTAINED FROM THE NOMOGRAM

Portion	Measured Distance	Length
0 - 0.01	0-a	14
0.01 - 0.05	0-b	10
0.05 - 0.10	0-c	1.6
0.10 - 0.25	0-d	2.6
0.25 - 0.50	0-e	0.2
	Total	28.4

There is no need to determine the numerical value of the magnitudes g/d for each portion separately. The summation should be carried out graphically by using a piece of paper and marking on it successively the ordinate lengths (the sum of all portions); thus, the magnitude $1/d_q$ is represented by the distance O-A. Such a nomogram makes it possible to determine the percolation coefficient for sandy loam and for sand of small and medium grain size, provided the coefficient does not exceed 15 to 20 m per day; ordinarily this coefficient is within 1 to 5 m per day. Addition of portions larger than 0.25 to 0.50 mm to the fine-grained soil has practically no effect on the permeability of the soil, since the larger grains increase the total surface area θ.

The left side of the nomogram is used for determining the percolation coefficient in relation to the porosity of the ground (p). If this coefficient is taken as 0.36 for the ground under consideration, then a straight horizontal line is drawn from the point on the y-axis corresponding to the reciprocal of the effective diameter to the point of intersection with the curve corresponding to the porosity of the ground (the curve 0.36 in the given example). The length of this horizontal straight line represents the magnitude $\sqrt{K}$ computed according to the scale of the y-axis (in the given example, $\sqrt{K} = 0.65$; hence, $K = 0.43$ m per day).

The percolation coefficient can be determined also from Kozeny's formula modified by A. A. Zamarin

$$K = 4000 \frac{p^3}{(1 - p)^2} d_k^2 \tag{12}$$

in which K is the speed of percolation in meters per day at a temperature of 0° C, and d_k is the effective diameter of Kozeny in millimeters. The nomogram shown in Fig. 5 and drawn analogously to the nomogram for Krüger's formula is used for determining the percolation coefficient from Kozeny's formula. The magnitudes $1/d_k$ and $\sqrt{K}$ are determined respectively from the left and right parts of the nomogram. In the given example, the percolation coefficient according to Kozeny's formula is 0.18 m per day (at $t = 0^\circ$ C). To determine the percolation coefficient corresponding to any other temperature, the value of K obtained from the nomogram is multiplied by a correction coefficient r given by Poiseuille's relationship

$$r = 1 + 0.0337t + 0.000221t^2 \tag{13}$$

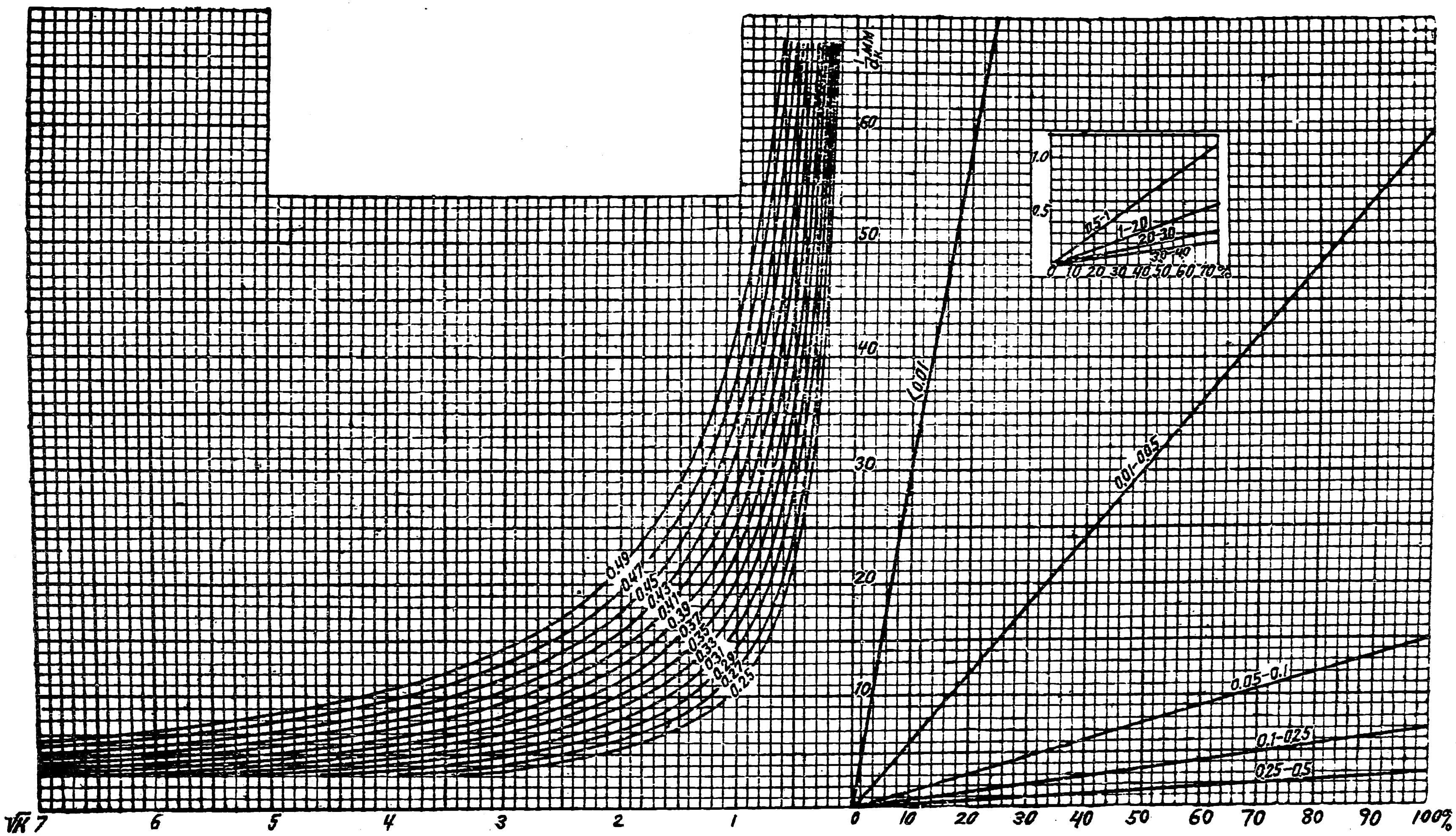


Fig. 5 – Nomogram for Determining K_ϕ from Kozeny's Formula

The accuracy of the determination of the percolation coefficient from the nomogram is sufficient for practical purposes.

The percolation coefficient can also be determined from Allen Hazen's and Darcy's formulas. Allen Hazen's formula has the form

$$K = Cd^2I\ (0.70 + 0.03t)\ \text{m per day} \tag{14}$$

in which t is the temperature of the water in degrees Centigrade; C is a numerical coefficient varying within the limits of 400 to 1200 (for sands this coefficient varies between 700 and 1000; the lower value is valid for sands mixed with particles of clay and silt); d is the effective diameter of the particles of homogeneous soil yielding the same magnitude of the percolation coefficient as the soil under consideration having heterogeneous particles; $I = H/\ell$ is the hydraulic slope.

Darcy's formula reads

$$Q = KF\frac{H}{\ell} = KFI \tag{15}$$

in which Q is the discharge and F is the total cross-sectional area of the ground, including the voids and the sectional area of the soil particles. The specific discharge per unit area is

$$V = \frac{Q}{F} = KI \tag{16}$$

Darcy's formula is valid only for soils having particles not greater than 3 mm, at small slopes and velocities. With increasing diameter of the particles, the rate of percolation approaches that given by Chezy's formula

$$V = KI^{\frac{1}{2}}$$

Darcy's formula is valid only for low heads and velocities.

Test pumping yeilds values of the percolation coefficient that are more accurate than those computed from laboratory data. However, this procedure requires a more complicated installation and experienced technicians not readily available in the field.

If the granulometric composition of the soil is known, an approximate determination of the percolation coefficient can be obtained by comparison with another soil for which the value of K is known already. The following average values can be used in this procedure.

a. For fine and very fine sand containing clay, at negligible permeability, K = 0.00002 to 0.00005 m per day.

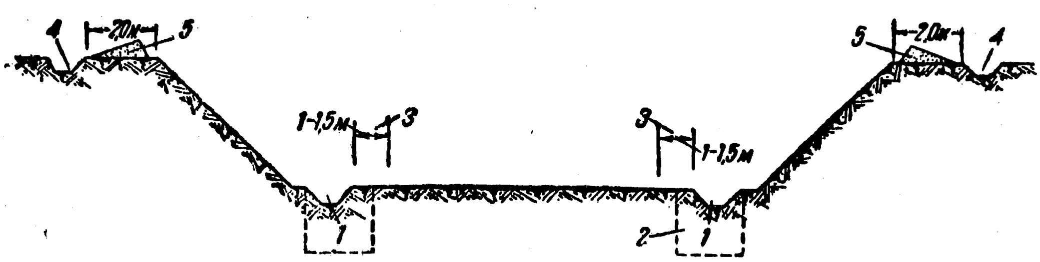
b. For sand of medium grain size, containing little clay, and for clean fine sand, at medium permeability, K = 0.0001 to 0.0005 m per day.

c. For coarse sand containing fine gravel, at high permeability, K = 0.001 to 0.005 m per day.

For direct determination of the velocity and direction of flow of ground water, a solution of table salt or special dyes (fluorescein, uranin) is introduced into the test hole. The salt solution or the dye moves together with the ground stream. A number of holes are drilled at some distance around the first test hole; the instant at which the salt solution or the dye introduced into the first hole appears in these holes is determined by analyzing the chemical content or color of the water from samples drawn from these holes at short intervals.

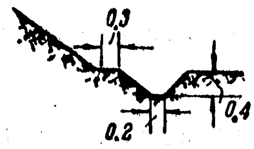
The electrical method (Slichter's) consists of introducing an electrolitic solution (ammonia, for instance) into one of the holes and electrodes into the other holes. The instant the solution reaches one of the neighboring holes, an increase in the electrical conductivity of the water is recorded on a galvanometer.

The method of determining the velocity by studying the wave motion of the ground water is based on the determination of the shape of the wave-like surface of the flow. Assuming that the wave motion occurs with the speed of motion of the surface, it is possible to determine the direction and speed of the wave displacement, and hence, the percolation coefficient, from two perpendicular profiles of the ground water.

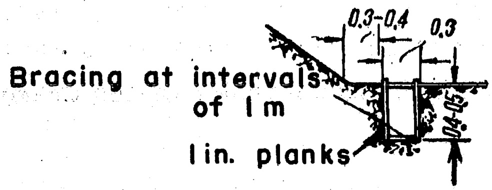


1. Drainage ditches
2. Collecting well
3. Boundary of structure
4. Outside ditches
5. Embankments

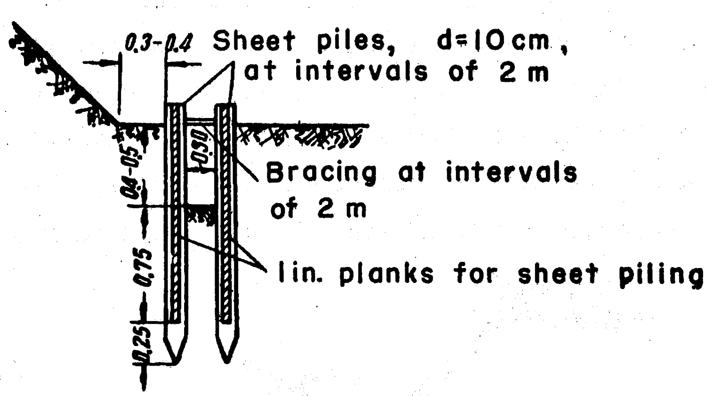
Fig. 6 – Cross Section of Excavation for an Underground Structure



(a) In firm soils



(b) In weak soils



(c) In unstable soils

Dimensions are in meters.

Fig. 7 — Diagrams of Drainage Ditches

CHAPTER II

DRAINAGE OPERATIONS

A. Protecting the Structures from Inflow of Water

The protection of underground structures, or of structures that are partly in the ground, from atmospheric water and ground water (which constitutes the subject of this work) can be accomplished by various means which can be divided into two principally different categories. The first category includes those preventive measures, the purpose of which is to divert or prevent, fully or partially, the flow of both atmospheric and ground water to the structure. The second category includes the measures for direct waterproofing of the structure.

The first category includes the following measures: diverting the surface water by means of open ditches, removing the water collecting on the surface and in the ground layers by pumps and other water-removing methods, organizing the collection of the ground water in a system of collecting wells and tunnels, capping of springs, and draining the ground layers by means of closed drainage installations.

The category of measures for direct waterproofing of the structures includes cement waterproofing, forcing the waterproofing solution behind the barriers, application of gunite to the surface, salinification of the protecting layer of earth, and several other measures.

The use of open ditches is not limited to diverting the atmospheric water which precipitates and collects on the surface. These ditches also help to dry the ground to a certain depth, depending on the depth of the ditch and the filtering property of the ground.

The preventive measures for protecting the structures from the action of the surrounding wet medium are most effective when used together with drainage operations employing open and closed canals.

Drainage operations which are used primarily during construction are frequently used also for exploitation purposes if other methods of protection from inflow and from the action of atmospheric and ground water on the structure prove insufficient.

B. Drainage by Open Ditches

The plans for drainage operations, which include the design of the plan, profile, cross sections, and surface reinforcing of the drainage ditches and of the regulating structures, depend on the functional purpose and the construction of the structure. The principles of preparing these plans are presented in special hydraulic publications, in textbooks of railroads, highways and bridges, and in other special technical literature.

This chapter presents some general directions for carrying out drainage operations in accordance with a given plan, independently of the type of structure.

C. Diverting the Surface Water by Open Ditches

Open ditches and troughs catch and remove the water of the surface flow from excavations and also collect and remove the surface water and the seepage of ground water from the excavations. In the presence of natural streams crossing the excavations, it is necessary to make provisions for diverting the streams by means of raised ditches or troughs having adequate section and passing through the excavation.

The diversion ditches should be located along the edges of the excavations outside the area of the base of the structures. Figure 6 shows a possible layout scheme for diversion troughs and collecting wells in the excavation. If the excavation is located on a slope, it is sufficient to surround it by ditches situated only on the uphill side.

If the ditches arranged along the contour of the excavation are insufficient for draining the entire area of the excavation, an additional net of collecting ditches is placed within the area of the excavation normal to the ground flow. After completion of the construction, these ditches should be filled in order that they do not form any paths for the water under the structure. The collecting ditches are connected to the diversion ditches.

It is necessary to evaluate the section of the ditches in accordance with the area which they are designed to drain. The minimum width of the ditch at the bottom is taken as 0.30 m, and the depth of the collecting ditches should be not more than 1.0 m. The collecting ditches should have slopes of 0.0005 to 0.001, and the slope of the diversion ditches and of those situated on the uphill side should be not less than 0.001 and, if possible, not greater than 0.005, depending on the contour of the site. In weak soils the walls of the ditches in the excavation should be shored by horizontal planks with cross bracing, while in unstable soils the walls should be shored by sheet piling (Fig. 7). Holes not larger than 15 mm in diameter are drilled in the piling to let the water through.

If the excavation consists of separate areas situated at different depths, the water from the ditches at higher levels is transferred to the adjacent lower levels by wooden spillways or chutes, or each area is provided with a separate collecting well. Local conditions permitting, the water from the excavations should be removed by gravity flow into gullies, ravines, streams, rivers, and other low places. The drainage ditches should have adequate capacity, so that no overflow will occur in the excavation. If the water cannot be removed by gravity flow, pumps must be used. Care should be taken that the water entering into the collecting ditches is not turbid and does not contain any suspended soil particles; the presence of suspended particles indicates that soil is being removed from the base and that the collecting ditches do not function properly. In this event, it is necessary to reduce the slope or to increase the dimensions of the ditches within the area where scour occurs.

At places where the ditches cross ramps or roadways used for removing the excavated material, the ditches are reinforced like culverts. In these areas the ditches are covered with planks to prevent the dirt from filling the ditches. While digging the ditches situated on the uphill side of the slope and surrounding the pit, the excavated soil is placed on the downhill side and arranged in the form of a parapet having a transverse slope in the direction opposite to that of the hill. The ditches situated on the uphill side should have a slope not less than 0.002; the greatest slope is determined from considerations of scour, but should be not steeper than 0.005 to 0.008, if feasible. If the specified slope of the ditches cannot be achieved because of the greater slope of the region, then the ditches are divided into stages with ledges properly strengthened, depending on the height of the drop.

To each type of ground there corresponds a limiting maximal flow velocity causing scour in drainage ditches designed to function both during construction operations and afterwards. In view of this fact, the flow velocity should not exceed the limiting values given in Table 3. However, if these velocities are exceeded because of topographical conditions, then the bottom and sides of the ditches must be reinforced by sodding, using a single or double layer, in accordance with the data of Table 3.

TABLE 3

FLOW VELOCITY IN RELATION TO BED SCOUR

No.	Type of Soil and Manner of Strengthening the Surface	Allowable Velocity Along the Bed m per sec	Mean Calculated Velocity, m per sec
1	Silty soil	0.10	0.15
2	Fine river sand	0.49	0.64
3	Ordinary or loamy sand	0.81	1.02
4	Dense clay with gravel	0.92	1.15
5	Large rubble soil	1.51	1.67
6	Overall sodding	0.60	0.77
7	Sides sodded	1.50	1.87
8	Single layer on moss or rubble	2.10	4.29
9	Double layer on moss or rubble	3.05	3.47

D. Designing the Cross Section of the Ditches

The cross section dimensions of the ditches are determined from the maximal possible value of the discharge Q_{max} evaluated according to the formula

$$Q_{max} = CTKF^{3/4} \tag{17}$$

in which

Q is the quantity of inflow in cubic meters per second,

C is the coefficient of topography, the value of which is taken as 10 for plane and slightly rolling areas, 15 for strongly undulating areas, 20 for slightly mountainous areas, and 25 for strongly mountainous areas,

T is the permeability coefficient of the soil, the value of which is taken as 1 for average absorption, 1.5 for low absorption, and 0.5 for high absorption,

K is the coefficient of climate, the value of which is taken as 1.0 for the central region of European USSR, and

F is the area in square meters of the watershed from which the water is collected. This area is determined from a map or with the aid of a goniometer.

In designing ditches to divert water from temporary structures built partly in the ground, the maximal value of the discharge (obtained from formula (17) and corresponding to the most intensive rainfall observed for a period of many years) may be reduced as follows: $Q_{des} = \frac{1}{2}Q_{max}$, for ditches the overflow of which may endanger the structure, and $Q_{des} = \frac{1}{3}Q_{max}$, for ditches the overflow of which does not endanger the structure.

The effective section of the ditch is calculated by the method of successive approximation. Assuming the depth of flow in the ditch, the slope of the sides is taken as 1:1 to 1:1½ and the following hydraulic elements are computed:

1. Area of effective section $\underline{W}$ (abcd, Fig. 8)
2. Wetted perimeter $P = ac + cd + db$
3. Hydraulic radius $R = \frac{W}{P}$
4. Mean velocity

$$V = C \sqrt{Ri} \tag{18}$$

where i = maximum slope of ditch, allowable for the given ground, and $C = \frac{87}{1 + \gamma\frac{1}{\sqrt{R}}}$ = coefficient, the values of which are given in Table 4; γ = roughness coefficient of the walls and bed of the channel, the values of which are given in Table 4.

From the computed magnitudes of the effective section and the mean velocity, the discharge of the ditch is found in accordance with the formula $Q = Wv$, and this discharge is compared with the magnitude Q_{des}. These two magnitudes should be approximately equal. If $Q > Q_{des}$, or $Q < Q_{des}$, the dimensions of the ditch have to be reduced or increased, respectively, and the computation is then repeated until Q and Q_{des} become nearly equal.

Example: Assume the area of the basin F = 0.95 sq km. The region is a plain, and the soil (dense clay with gravel) has medium absorption. Then the discharge in the basin of the ditch is

$$Q_{max} = CTKF^{3/4} = 10 \times 1 \times 1 \times 0.95^{3/4} = 10 \times 0.96 = 9.60 \text{ m}^3 \text{ per sec}$$

TABLE 4

VALUES OF THE COEFFICIENT $C = \dfrac{87}{1 + \gamma \dfrac{1}{\sqrt{R}}}$

Hydraulic Radius R, m	Roughness Coefficient γ					
	0.06	0.16	0.46	0.85	1.30	1.75
0.05	68.5	50.7	28.4	18.1	12.8	9.9
0.06	69.8	52.6	30.2	19.4	13.8	10.7
0.07	70.9	54.2	31.7	20.6	14.7	11.4
0.08	71.8	55.6	33.1	21.7	15.5	12.1
0.09	72.5	56.7	34.4	22.7	16.3	12.7
0.10	73.1	57.7	35.5	23.6	17.0	13.3
0.11	73.6	58.7	36.5	24.4	17.7	13.9
0.12	74.1	59.5	37.4	25.2	18.3	14.4
0.13	74.6	60.2	38.2	25.9	18.9	14.9
0.14	75.0	60.9	39.0	26.7	19.4	15.3
0.15	75.3	61.5	39.7	27.2	19.9	15.8
0.16	75.6	62.1	40.5	27.8	20.4	16.2
0.17	75.9	62.7	41.2	28.4	20.9	16.6
0.18	76.2	63.2	41.8	29.0	21.4	17.0
0.19	76.5	63.6	42.4	29.5	21.8	17.3
0.20	76.7	64.1	42.9	30.0	22.3	17.7
0.21	76.9	64.5	43.5	30.5	22.7	18.1
0.22	77.1	64.9	44.0	30.9	23.1	18.4
0.23	77.3	65.2	44.4	31.4	23.4	18.7
0.24	77.5	65.5	44.8	31.8	23.8	19.0
0.25	77.6	65.9	45.3	32.2	24.2	19.3
0.26	77.8	66.2	45.7	32.6	24.5	19.6
0.27	78.0	66.5	46.1	33.0	24.8	19.9
0.28	78.1	66.8	46.5	33.4	25.2	20.2
0.29	78.3	67.0	46.9	33.7	25.5	20.5
0.30	78.4	67.3	47.3	34.1	25.8	20.7
0.31	78.5	67.6	47.6	34.3	26.1	21.0
0.32	78.6	67.8	47.9	34.7	26.4	21.2
0.33	78.8	68.0	48.2	35.1	26.7	21.5
0.34	78.9	68.2	48.5	35.4	26.9	21.7
0.35	79.0	68.4	48.8	35.7	27.2	22.0
0.36	79.1	68.6	49.2	36.0	27.5	22.2
0.37	79.2	68.8	49.5	36.3	27.7	22.4
0.38	79.2	69.0	49.8	36.6	28.0	22.7
0.39	79.3	69.2	50.1	36.8	28.2	22.9
0.40	79.4	69.4	50.4	37.1	28.5	23.1
0.41	79.5	69.6	50.6	37.4	28.7	23.3
0.42	79.6	69.7	50.9	37.6	28.9	23.5
0.43	79.7	69.9	51.1	37.9	29.2	23.7
0.45	79.8	70.2	51.6	38.4	29.6	24.1
0.46	79.9	70.4	51.8	38.6	29.8	24.3
0.47	80.0	70.5	52.0	38.8	30.0	24.5
0.48	80.0	70.6	52.3	39.1	30.2	24.7
0.49	80.1	70.8	52.5	39.3	30.4	24.8

TABLE 4 (Continued)

Hydraulic Radius R, m	Roughness Coefficient γ					
	0.06	0.16	0.46	0.85	1.30	1.75
0.50	80.2	70.9	52.7	39.5	30.6	25.0
0.55	80.4	71.5	53.7	40.5	31.6	25.9
0.60	80.7	72.1	54.6	41.4	32.5	26.7
0.65	80.9	72.6	55.4	42.3	33.3	27.4
0.70	81.1	73.0	56.1	43.1	34.1	28.1
0.75	81.3	73.4	56.8	43.9	34.8	28.8
0.80	81.5	73.8	57.4	44.6	35.5	29.4
0.85	81.7	74.1	58.0	45.2	36.1	30.0
0.90	81.8	74.4	58.6	45.9	36.7	30.6
0.95	81.9	74.7	59.1	46.5	37.3	31.1
1.00	82.0	75.0	59.6	47.0	37.8	31.6
1.10	82.2	75.4	60.5	48.0	38.8	32.6
1.20	82.4	75.9	61.3	48.9	39.7	33.5
1.30	82.6	76.3	62.0	49.8	40.6	34.3
1.40	82.8	76.6	62.6	50.6	41.4	35.1
1.50	82.9	76.9	63.2	51.3	42.2	35.8
1.60	83.0	77.2	63.8	52.0	42.9	36.5
1.70	83.1	77.5	64.3	52.6	43.6	37.1
1.80	83.2	77.7	64.8	53.2	44.2	37.7
1.90	83.3	77.9	65.2	53.8	44.8	38.3
2.00	83.4	78.1	65.6	54.3	45.3	38.9
2.20	83.6	78.5	66.4	55.3	46.4	39.9
2.40	83.7	78.8	67.1	56.2	47.3	40.8
2.60	83.8	79.1	67.7	57.0	48.1	41.7
2.80	83.9	79.4	68.2	57.7	48.9	42.5
3.00	84.0	79.6	68.7	58.3	49.7	43.3
3.20	84.1	79.8	69.2	58.9	50.4	44.0
3.40	84.2	80.0	69.6	59.5	51.0	44.6
3.60	84.3	80.2	70.0	60.1	51.6	45.8
4.00	84.4	80.5	70.7	61.0	52.7	46.4
4.50	84.6	80.9	71.5	62.1	53.9	47.6
5.00	84.7	81.2	72.1	63.0	55.0	48.8
5.50	84.8	81.4	72.7	63.8	56.0	49.8
6.00	84.9	81.6	73.2	64.6	56.8	50.7

Since overflowing of the ditch cannot endanger the structure, the following is valid:

$$Q_{des} = \frac{Q_{max}}{3} = \frac{9.60}{30} = 3.20 \text{ m}^3 \text{ per sec}$$

Considering the ditch the dimensions of which are shown in Fig. 8, the values assumed are as follows: slope of ditch i = 0.003, roughness coefficient $\gamma = 1.75$, and depth of flow = 1.30. The area of the effective section abcd is $w = 3.25\ m^2$; $P = 4.96$ m; $R = \frac{3.25}{4.96} = 0.65$ m; $C = 27.4$ (from Table 4) and $V = C\sqrt{Ri} = 27.4\sqrt{0.65 \times 0.003} = 1.20$ m per sec. Hence, the discharge of the ditch is: $Q = 3.0 \times 1.20 = 3.60\ m^3$ per sec.

The discharge computed from the dimensions of the ditch differs little from Q_{des} computed from the area of the basin, since $Q_{des} = 3.20\ m^3$ per sec; hence, the assumed dimensions of the ditch may be regarded as final dimensions.

Comparison of the computed velocity V = 1.20 m per sec with the mean design velocity given in Table 3, v = 1.15 m per sec, indicates that it is permissible not to use any artificial strengthening of the walls and bottom of the channel.

TABLE 5

ROUGHNESS COEFFICIENT γ

No.	Type of Wall	Roughness Coefficient γ
1	Very smooth walls (planed planks, smooth cement plaster, etc.)	0.06
2	Smooth walls (unplaned planks, thin planks or brick masonry, concrete and cast iron pipes, well-applied concrete, and others)	0.16
3	Rough walls (good rubble, moderate concrete workmanship)	0.46
4	Intermediate category (coarse rubble, very rough application of concrete on rock, paving with rubble, well-kept walls of dense earth, walls cleanly hewn in rock)	0.85
5	Earth walls in natural condition (also paved walls covered with some vegetation, etc.)	1.30
6	Earth beds offering large resistance (when poorly maintained, covered with considerable algae, or the bottom of which is rocky with detritus or consists of coarse stones)	1.75

CHAPTER III

PUMPING OPERATIONS

A. Collecting Wells

When the topography of the area does not permit gravity flow of the water collected by the drainage ditches and in order to lower the ground water table of the adjacent region, collecting basins are built and the water is removed from them by pumps.

Collecting wells are used as an independent means of draining when the soil does not readily yield the water (clays, loams, soils of fine silt), when open shallow ditches are sparsely distributed and cannot lower the ground water table sufficiently, and when the percolation coefficient is small, equivalent to an inflow into the excavation not greater than 0.01 liter per sec from 1 m^2.

If the wells are used only for the purpose of collecting the water from the ditches and removing it by pumping, then the density distribution of the wells depends on the quantity of water entering from the collecting ditches, on the distribution of the ditches in plan, and on the contour of the bottom of the excavation. The density distribution of the wells designed for collecting and pumping the water entering directly from the ground depends on the water yield of the ground. The collecting wells are located outside the area of the base of the structures and have a rectangular section of 1.50 by 1.50 m to 2 by 4 m. The cross section is determined in accordance with local conditions, depending on the drainage area and the design capacity. The minimum distance between the edge of the well and the outer edge of the structure is tentatively taken as equal to the depth of the well.

In designating the locations of the collecting wells it is essential that the wells intercept the water before it reaches the excavation and that they are located outside the excavation. If the major streams of water flowing into the excavation are intercepted by the wells, it is not difficult to dry the bed of the excavation by light pumping.

In unstable grounds the walls of the wells are shored with sheet piling. Soil particles transported by the underground flow may enter the wells with the water, causing a condition which might result in the formation of cave-ins in the surrounding regions. Since this situation might endanger the erected structure, filters are installed in the collecting wells to prevent scour of the soil. The bottom of the well should be located 1.0 to 1.5 m below the bottom of the approach ditches; it should be lined with plank flooring (Fig. 9) in order to prevent turbulence or heaving of the ground. Openings are made in the walls of the wells at points where the ditches approach and at points of inflow from the ground. The part of the approach ditch adjacent to the well is lined with planks for a distance not less than 1.50 m.

B. Pumps

During pumping operation the intake valve (strainer) of the suction pipe of the pump is lowered into the well. To protect the valve from dirt, it is necessary to insert the strainer in a box installed in the well, or to install a filter in the well. The pump is mounted on a special frame attached to the well or near it, or on the berm of the excavation sidewall. The suction head should be not greater than 5 to 6 m. The pump capacity required is determined on the basis of hydrogeological data for the percolation coefficient and on the basis of data obtained from test pumping from bores previously drilled to the bottom of the planned excavation. The capacity of the pumps and the power of the motors for various diameters of the pump pipes are given in Tables 6 and 7.

TABLE 6

CHARACTERISTICS OF SINGLE-STAGE CENTRIFUGAL PUMPS

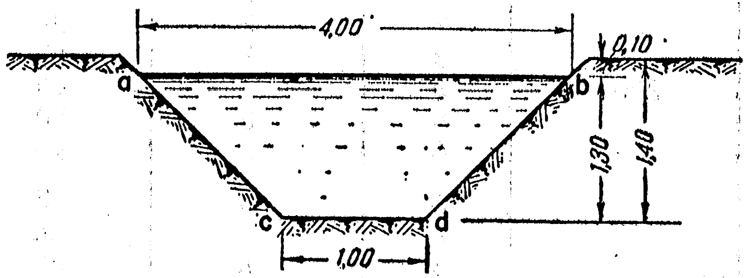
Diameter of Suction and Discharge Pipes, mm	40	50	60	75		100	125	150	200	250
No. of rpm	2900	2900	2880	1450	2850	1450	1470	1470	980	980
Liters per sec	2	4	6	8	11	18	25	41	82	118
m^3 per hr	7.2	14.4	21.6	28.8	39.6	64.8	90.0	147.6	295.2	432.0
Shaft hp	1.1	2.6	4.8	2.0	11.0	6.0	12.0	25.0	42.0	77.0
Motor hp	1.5	3.25	6.0	2.5	12.1	6.6	13.2	27.5	56.8	104.0
Total head in m (suction, pressure, and loss in pipes)	16	22	30	10	41	15	22	30	26	34

TABLE 7

CHARACTERISTICS OF MULTI-STAGE CENTRIFUGAL PUMPS

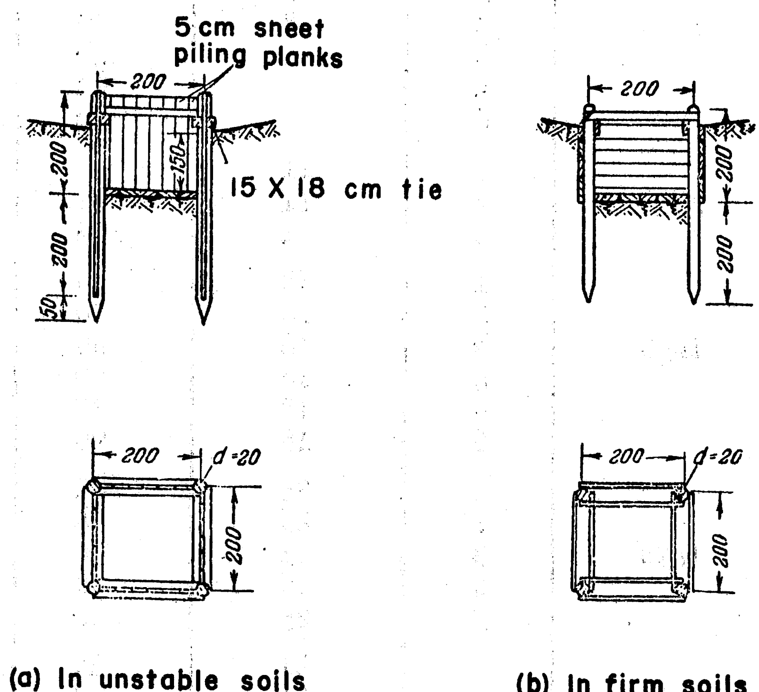
Diameter of Suction and Pressure Pipes, mm	Revolutions Per Min.	Number of Stages	Capacity, Liters Per Sec	Head H, m	Shaft Horsepower
40	2900	1	2	16	1.0
		2	2	32	2.1
		3	2	48	3.1
		4	2	64	4.0
		5	2	80	5.5
		6	2	96	6.5
50	2900	1	4	22	2.6
		2	4	44	5.0
		3	4	66	7.5
		4	4	88	9.5
		5	4	100	12.0
60	2880	1	6	30	4.8
		2	6	60	9.5
		3	6	90	14.0
		4	6	120	18.0
75	1450	1	8	10	2.0
		2	8	20	3.8
		3	8	30	5.8
		4	8	40	7.5
		5	8	50	9.0
		6	8	60	10.5
75	2850	1	11	41	11.0
		2	11	82	21.0
		3	11	123	30.0
100	1450	1	18	15	6.0
		2	18	30	12.0
		3	18	45	17.0
		4	18	60	23.5
		5	18	75	29.0
		6	18	90	35.0
125	1470	1	25	22	12.0
		2	25	44	23.0
		3	25	66	35.0
		4	25	88	45.0
		5	25	110	56.0
150	1470	1	41	30	25.0
		2	41	60	50.0
		3	41	90	74.0
		4	41	120	97.0

The installations for collecting and removing the water should be correctly utilized and reconditioned at the proper time by organized repair crews. All the auxiliary structures used for the mechanical drainage operations, such as booths for pumping equipment and fuel storehouses, should be heated in winter.



Dimensions are in meters.

Fig. 8—Cross Section of a Drainage Ditch



Dimensions are in centimeters.

Fig. 9—Diagrams of Collecting Wells

CHAPTER IV

SPECIAL MEASURES FOR PREVENTING THE INFLOW OF WATER

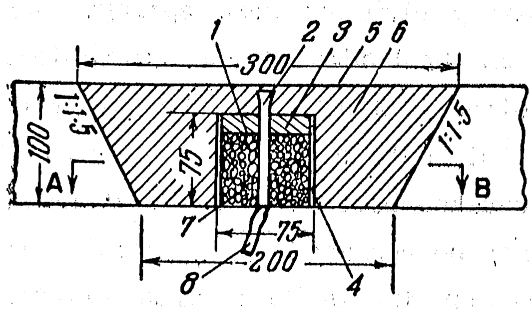
A. Capping of Springs

If relatively high pressure springs issuing from under the impermeable layer occur on the bottom of the excavation, these springs are capped at the source as follows. A pit approximately 2 by 2 m in cross section and 1 m in depth (Fig. 10) is dug around the spring, and a bottomless barrel or a solidly built and caulked bottomless box 0.75 by 0.75 by 0.75 m is placed in the pit. The box may be replaced by a concrete ring of the same dimensions. The pit around the box or ring is filled with well-tamped 0.10- to 0.15-m layers of clay or, preferably, with a mixture of clay and cement. An iron pipe 100 to 150 mm in diameter is placed over the opening of the spring. The pipe and box, or concrete ring, are compacted with gravel to three-quarters height. A hand-pump hose is inserted in the end of the pipe, and the water is continuously pumped out. The upper part of the box or ring is filled to the edges with a 1:2 cement mixture. The water is pumped until the cement mixture is hardened. After that the pumping is ceased, the iron pipe is plugged with a wooden plug, and the filling of the pit is completed.

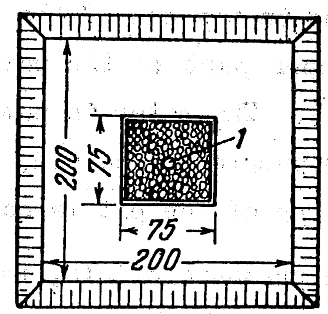
If considerable pressure exists, the danger arises that the spring may burst to the surface outside the capped area or expel the wooden plug. In this event, a more permanent plug is required. For this purpose the central pipe is gradually extended above the concrete structure until any further inflow of water from the spring ceases under the influence of the head formed in the pipe. The pipe is then plugged with concrete under which is placed a gravel filter to prevent the removal of small particles by the water and the scouring of the concrete. If the capping of the spring is unsuccessful and the spring recurs at another place, the entire operation is repeated at the place where the spring recurred.

B. Draining the Surface Water from Trenches

In order to drain the surface water from trenches, the bottom of the trench should have a 0.02 to 0.03 transverse slope towards the rear wall. A ditch 12 to 15 cm deep and 10 to 12 cm wide along the bottom, dug at the foot of the rear wall, collects and drains the water (Fig. 11). Topographic conditions permitting, the longitudinal slope of the bottom of the trench



Cross section



Section A–B

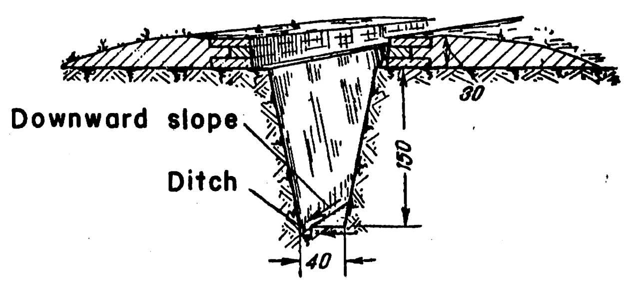
1. 100–150 mm steel pipe
2. Wooden plug
3. 1:2 cement mixture
4. 25–38 mm planks
5. Bottom of excavation
6. Impermeable ground
7. Fine gravel
8. Mouth of spring

Dimensions are in centimeters.

Fig. 10 – Diagram for Capping of a Spring on the Bottom of an Excavation

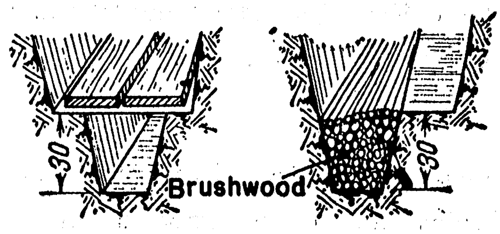
and ditch should be such that the water could drain from the trench by gravity flow into ravines, gullies, streams, rivers, and similar places of lower elevation.

If the trenches are dug in clay, loam, or other solid soils of poor permeability, the ditches should be made deeper and covered with planks or logs, or the ditch should be filled with fascines (Fig. 12). If the water cannot be drained from the trench by gravity flow, collecting wells are built and the water is periodically bailed out with buckets (Fig. 13). If the trench is situated on a slope, a ditch is dug on the uphill side at a distance of 3 to 5 m from the trench to catch all the water flowing down the slope. The longigudinal slope of this ditch should be not less than 0.002; the greatest slope is determined on the basis of the scour of the soil in which the ditch is made, and if possible should not be greater than 0.005 to 0.008. When ditches are dug in sandy soil, it is not necessary to make provisions for draining the surface water, since the water filters through the sand without collecting on the bottom of the trench.



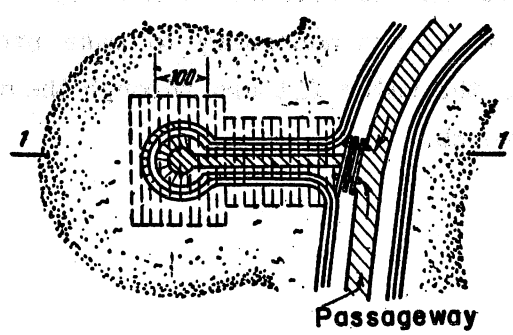
Dimensions are in centimeters.

Fig. 11 — Draining a Trench or a Connecting Passageway by Means of a Ditch Covered with a Plank

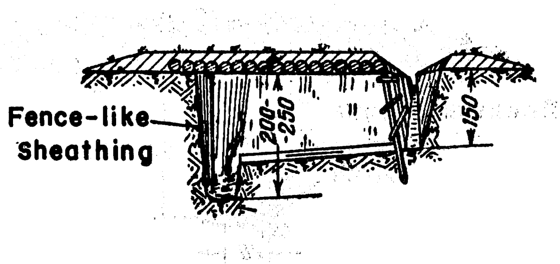


Dimensions are in centimeters.

Fig. 12– Arrangement of Drainage Ditches in Soils of Poor Permeability



(a) Plan



(b) Section I–I

Dimensions are in centimeters.

Fig. 13– Collecting Well in a Trench

C. Draining the Sites of Structures by Closed Drains

Drainage denotes an installation for removing water from the ground and carrying it to a waste basin. In erecting individual structures or a system of shelters and dugouts under conditions which make it possible to remove the ground water into lakes, ravines, or similar low places, it is essential to drain the soil so that waterproofing may become unnecessary.

Drainage is effective only in sand, gravel, conglomerate, and similar soils of sufficient permeability. In sandy loams and in light loam, drainage is not always sufficiently effective; in heavy loam and in clay, drainage is usually not effective at all. Closed drains are used when it is either unsuitable or impossible to effect drying by open drainage ditches, as in the following cases: in narrow, deep excavations where the ground water table occurs at a depth greater than 1.0 m; when it becomes necessary to effect a general lowering of the ground water table in an area where open ditches are not feasible (for instance, when the ground water issues from nests of springs); and, finally, when it is necessary to tap the springs and divert them to definite locations.

The drains collecting the water consist of a narrow and deep ditch 0.30 m wide at the bottom and 1 to 2 m deep. This ditch is filled to a height of 0.30 to 0.40 m with porous materials, such as logs, fascines, stones or gravel 5 to 6 cm in size, surrounded on all sides by a simple filter of perhaps a 10-cm layer of coarse sand. In the case of large discharges, such as removal of water from tapped springs, wooden or vitreous pipes protected by the filter are laid along the bottom of the ditch (Fig. 14), and the upper part of the ditch is filled with filtering soil.

Vitreous drains are laid at a slope of 0.005 to 0.03, while fascine and other drains are laid at a slope of 0.001 to 0.02, depending on the topography of the region, the hydrogeological conditions, and the flow velocity. The allowable flow velocity is within 2.50 m per sec for a system of wooden pipes, and 3 to 4 m per sec for vitreous pipes. The danger of scour does not arise in the case of drains formed from brushwood, fascines, and logs, regardless of the steepness of the slope.

Figures 15 and 16 show a schematic diagram of the drainage and removal of surface water from dugouts.

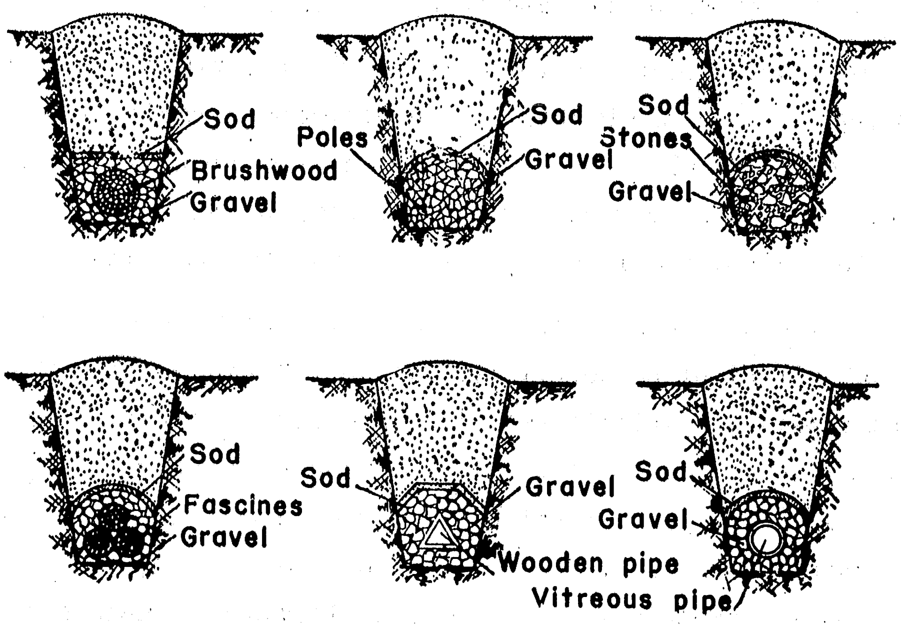
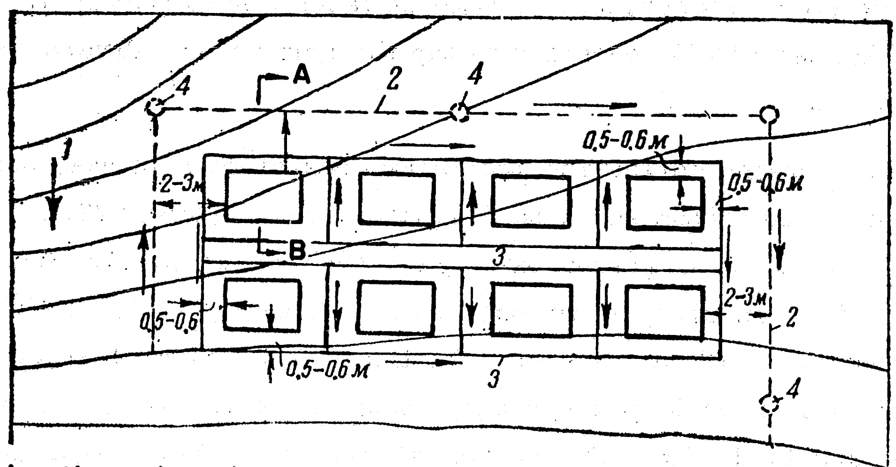


Fig. 14 — Typical Construction of Drains



1. Direction of surface slope and of drop of water-bearing layers
2. Drainage ditches
3. Open ditches
4. Manholes

Arrows indicate direction of slope of drainage and open ditches.

Fig. 15 — Layout Plan of Drainage System for Dugouts

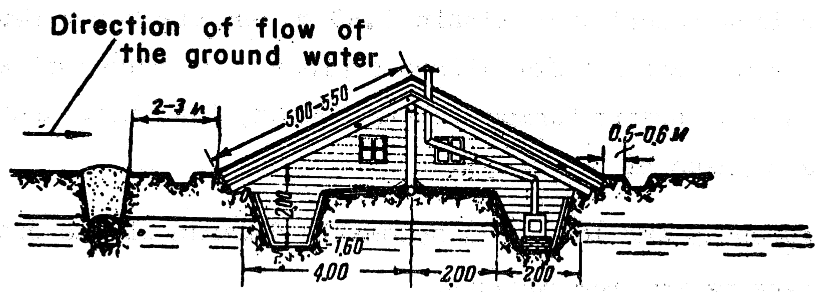


Fig. 16 — Cross Section of a Dugout (Section A—B)

CHAPTER V

MEASURES OF WATERPROOFING AGAINST GROUND WATER

A. Removing the Ground Water

If a high ground water table exists, it is necessary to lower it to a level below the base of the excavation prior to pouring the concrete which is to be waterproofed. The lowering of the water table is accomplished by drainage ditches or by pumping the water from wells, sumps, and similar installations.

To achieve effective pumping operation, it is necessary to leave small wells in the concrete base. These wells may be of circular or square section, 0.75 by 0.75 m, which occasionally are deepened to a level below the bed of the excavation. A 4-to 5-in. steel pipe is introduced into the well, and the space around the pipe in the well is filled with fine gravel to three-quarters height. These wells, which are called sumps, are located in the lowest places; for this purpose the bed of the excavation is designed with a slight slope (about 2 per cent) in the direction of the sump locations under consideration. As a result of such an arrangement, the ground water flows into the sumps and is removed by pumps with hoses connected to the steel pipe mentioned above; this pipe is plugged with a wooden plug during the time between pumping operations.

The sump can be formed from a bottomless box made of tightly connected and caulked planks 0.75 m high, or from a concrete ring of dimensions 0.75 by 0.75 m. Occasionally sumps are formed from empty cement barrels.

The pumping must proceed continuously during the waterproofing operations. After the membrane has been attached, the pumping must cease for a short time in order that the quality of the waterproofing may be checked. If the membrane then bulges uniformly under the influence of the water pressure, it is indicative that the quality of the work is satisfactory. However, if water appears on the inner surface of the structure, it is indicative of loose spots in the waterproofing membrane or at places where the membrane adjoins the elements of the construction to which the insulation is applied.

The lowering of the ground water table is completely stopped after the waterproofing material is in place and after the reinforced concrete, which reaches to 60 cm above the ground water table, attains the desired strength.

If ground water appears at the base or at the walls during the operation of attaching the membrane, the leaks should be sealed with mortar to which is added liquid glass or calcium chloride in the amount of 10 to 20 per cent by weight of the dry substance with respect to the weight of the cement. The use of alumina cement is recommended for such cases. If the seepage is considerable, a drain tube is inserted at the point of seepage, and the area around the tube is sealed with a cement mixture containing liquid glass or calcium chloride; the tube is sealed after the mixture has hardened.

Prior to applying the waterproofing material, the surfaces of the slab and walls should be dried out; air blowers, electric heaters, and increased ventilation may be used for this purpose. If the percolation coefficient is so large that the lowering of the ground water table to the level indicated becomes quite costly, then an additional layer of rubberoid is placed over the surfaces and the seams are sealed. A binder made of a cement mixture containing liquid glass or calcium chloride is applied over the rubber, and the waterproofing membrane is laid on the hardened and dried surface.

B. Waterproofing of Basement Shelters

Elimination of seepage in basements utilized as shelters can be achieved (as described above) by applying a waterproofing membrane over the surface of floors and walls which have been dried and plastered with a cement mixture. This is true only if the level of the inflow of ground water does not rise more than 10 cm above the floor level. A small inflow at high ground water table constitutes a difficult seepage problem. This problem can be solved in two ways: by a design based on the magnitude of the hydrostatic pressure and the percolation properties of the ground, and by a design based on the construction of the foundation, the distance between lateral walls, the height of the chamber, and similar aspects of the structure that is being insulated. Figures 17 and 18 show examples of designs for waterproofing of existing basements.

To prevent flotation of the reinforced concrete slab subjected to pressure, the slab is fastened to the existing walls by extending the reinforcing bars into grooves made in the walls. If little seepage occurs at individual places in the basement, it is sufficient to plug the leaks (Fig. 19) and it is not necessary to carry out any waterproofing operations.

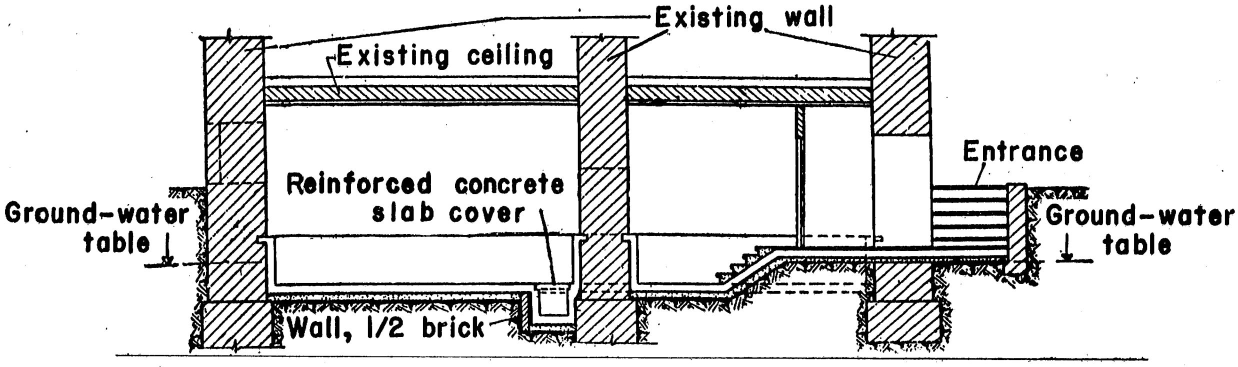


Fig. 17 – General View

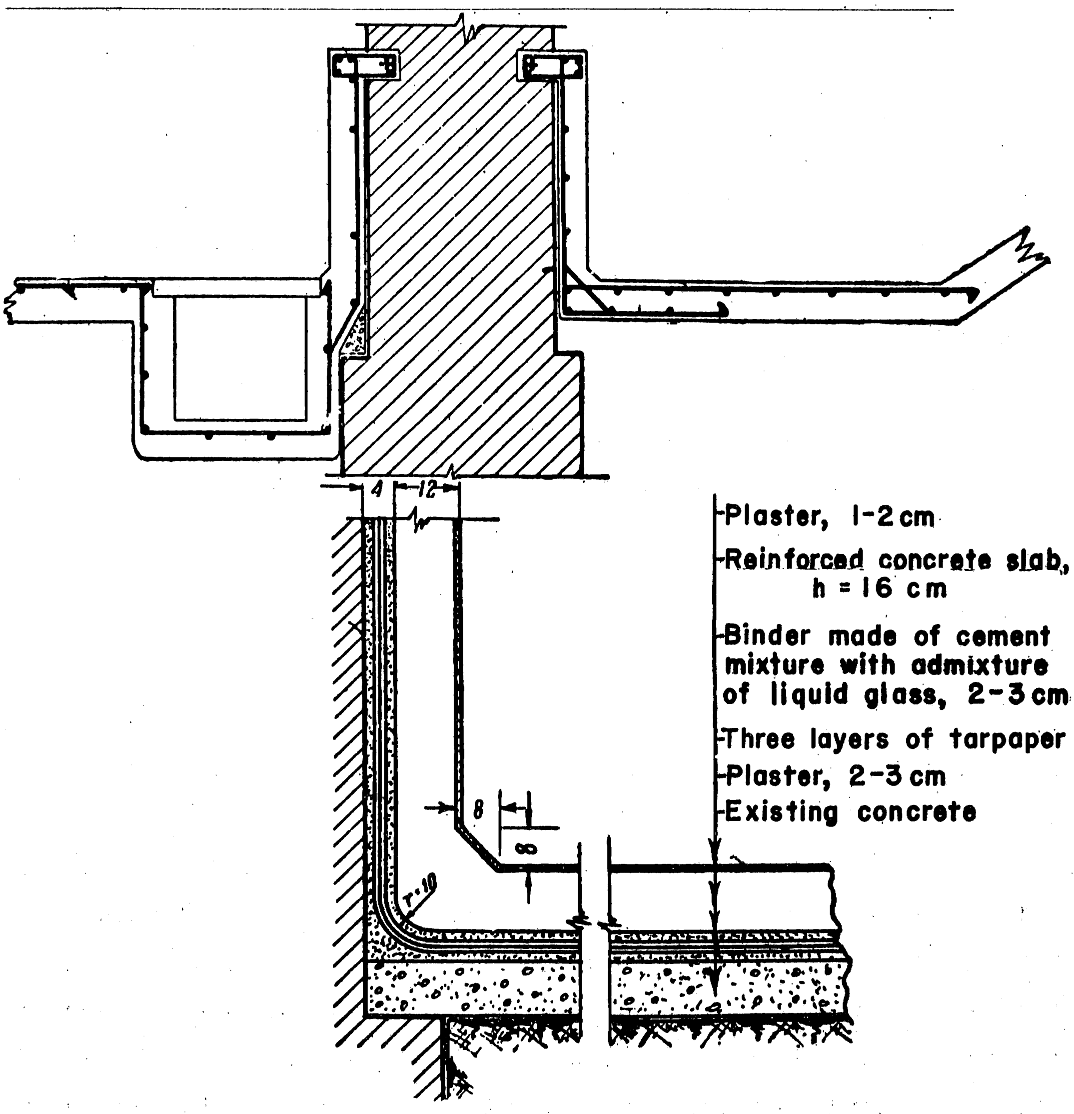


Fig. 18 – Details

Fig. 17 and 18 – Waterproofing of a Shelter Arranged in an Existing Building under Conditions of Considerable Inflow

C. Waterproofing the Sump

Sumps are a means of contributing to the effectiveness of pumping operations. According to this definition, therefore, after completion of the waterproofing operations the sumps should be sealed and waterproofed, so that they cannot serve as conduits through which the surrounding ground water would penetrate into the chamber. However, the sumps are occasionally utilized when the basement is in use; they are used for periodic removal of the ground water collecting under the floor when the ground water table rises, as it frequently does during the spring. In the latter case, the steel pipe of the sump is raised above the floor level and is usually plugged with a wooden plug, while its location is concealed by benches, chairs, and similar items.

Waterproofing of the sump occurs after the operation for waterproofing the structure is completed. For this purpose, the box or ring is filled to the upper pipe ring with concrete having an admixture of liquid glass or calcium chloride and deposited in layers not greater than 20 cm; simultaneously, the water is continually pumped out until the concrete has set, at which time the pumping is stopped, the pipe is plugged, and the sump is filled with concrete to the level of the top of the concrete floor (Fig. 20). After the concrete has hardened and the sump surface has dried, waterproofing material is laid in the usual way, so that the tar paper forms a lap joint with the layers laid previously. The last layer of tar paper is covered with hot cement over which is laid a cement binder 2 to 3 cm thick.

D. Waterproofing of Pipes at Junction with Concrete

The junction of the tar paper material with the main should be particularly accurate at the intersection of the steel pipes and the concrete walls of the basement chambers. A flange is carefully welded to the pipe at the level of the membrane (Fig. 21); this flange is 10 cm wide and has bolts welded to it with their threads facing the insulation. The flange surface facing the membrane is cleaned until it shines, painted with bituminous varnish and then dried. The flange is fastened to the wall on the side of the varnished surface. A collar of the same size as the flange is then prepared with its holes opposite the flange bolts facing the membrane. The surface of the collar is thoroughly cleaned, and it is covered with bituminous varnish which is then allowed to dry. The tar paper material is placed so that it completely covers the flange, while the holes for the bolts are carefully cut out. Each layer is placed only after both the surface which is to be insulated and the tar paper are thoroughly covered with cement; then the layer is pressed down with a spatula. After all the waterproofing layers have been placed around the flange, an additional layer covered with cement is applied; then the collar is mounted and firmly attached to the flange by tightening the bolts. A binder is placed over the surface of the membrane and collar; concrete is poured for external insulation, or a protective cover is added for internal insulation.

If the concrete mass is intersected by a cast iron main, the insulating operations proceed in a manner analogous to that given above, except that a sleeve with a flange is mounted on the pipe, instead of welding the flange to the pipe (Fig. 22). Insulating tar paper packing covered with cement is placed between the sleeve and the pipe. The inner surface of the sleeve and the outer surface of the pipe, which face each other, are cleaned and covered with bituminous varnish which is then allowed to dry. The subsequent operations are carried out in the same way as in the case of intersection with steel pipes.

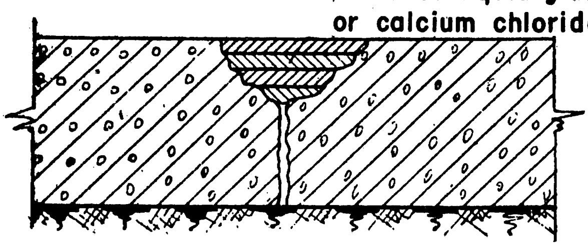
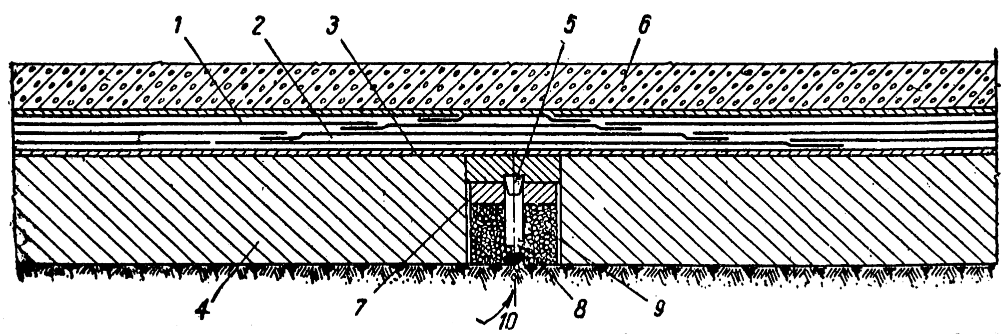
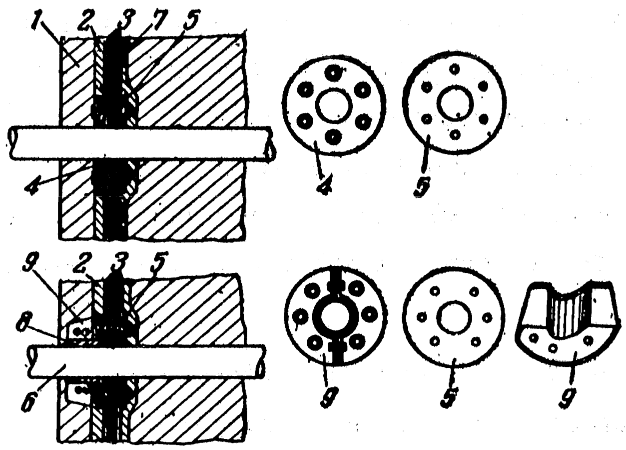


Fig. 19—Sealing the Concrete Slab of a Shelter Floor



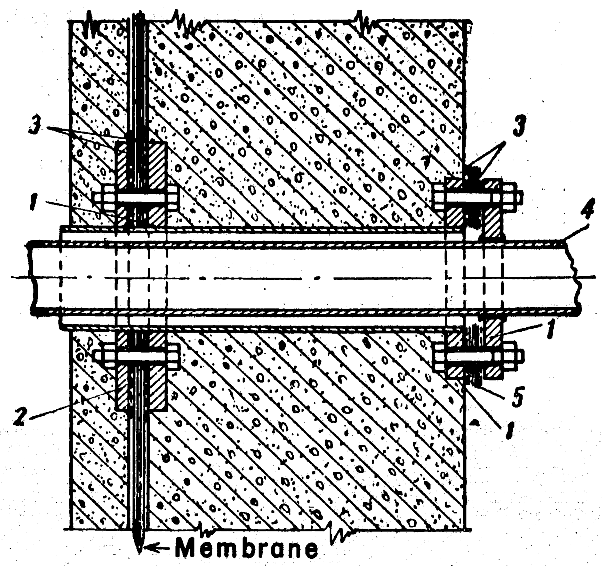
1. Four layers of tarpaper 2. 2 cm concrete binder 3. Plaster 4. Concrete 5. Wooden plug 6. Reinforced concrete 7. 1:2 cement mixture with liquid glass admixture 8. 100—150 mm steel pipe 9. Fine gravel 10. Ground water outlet

Fig. 20—Diagram Illustrating Sealing of a Sump



1. Protective layer 2. Plaster 3. Membrane
4. Flange 5. Collar 6. Main 7. Binder
8. Insulating layer 9. Clamps

Fig. 21 – Junction of Waterproofing Membrane with a Steel Main



1. Flange (welded) 2. Collar
3. Wire screen covered with bitumen
4. Main 5. Waterproofing seal

Fig. 22 – Junction of Waterproofing Membrane with a Cast Iron Main

CHAPTER VI

WATERPROOFING BARRIERS FOR UNSTABLE AND WATER-BEARING SOILS

During construction of tunnels, mine shafts, dugouts, and similar structures in unstable and water-bearing soils, impermeable barriers are frequently erected by means of the cementation method* as well as by claying, chemical hardening, bituminizing, and freezing of the soil.

A. Claying

This method consists of applying a clay mixture under pressure, which results in complete impermeability of the cracked ground. Calcium chloride and lime are added to the mixture in order to expedite the precipitation of the clay particles in the cracks and the removal of excess water, while sealing is achieved by increasing the amount of pressure applied. The order of operations and the equipment (pneumatic method or reciprocating pumps) are the same as those used in regular cementation process. Actual results confirm the fact that the claying method produces considerable durability and resistance to the action of aggressive water.

The mixture used for claying should have the following properties:

1. Sufficient viscosity, but not to the extent that the pumping of the mixture becomes difficult.

2. Highest possible specific weight.

3. Colloidal qualities permitting the formation of a firm gel.

4. Minimum sand content.

The viscosity can be regulated with precision sufficient for practical purposes by means of Funel's viscosimeter. This viscosimeter has the shape of a funnel and is used for determining the viscosity of clay mixtures during drilling of oil bores (Fig. 23). The viscosimeter is operated as follows: The lower opening of the funnel is closed with a finger and 0.50 liter of the solution is poured into the funnel; then the finger is removed and the duration of the discharge of the entire quantity is recorded by a stop watch.

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*This method is described in detail in the original text but is not given in this translation.

At high viscosity the discharge of the mixture does not stop suddenly; therefore, the termination of flow is assumed to correspond to the instant at which the jet begins to break up and the mixture flows in separate drops. Pure water should discharge from Funel's viscosimeter in 18.5 seconds. After a mixture has been selected, suitable for the given pumping system, the viscosity of the mixture is determined by means of Funel's viscosimeter. The viscosity of additional mixtures is compared with that of the previous mixture.

The colloidality of a mixture is characterized by the quantity of sediment deposited in 24 hours. For example, if 8 cm^3 are precipitated in 24 hours from 100 cm^3 of the mixture, then the colloidality of the mixture is 92 per cent. To assure greater precision, the precipitation is allowed to proceed for 48 hours. The objective of the sedimentation test is twofold: (1) to determine the quantity of sand settling in a few minutes from the mixture diluted with water, and (2) to determine the colloidality of the remaining clay mixture.

The clay used for claying purposes should have an Atterberg plasticity coefficient not less than six or seven. The specific weight of the clay is determined by weighing or by an areometer.

The claying method was used in constructing the main shaft no. 1 in Kizil and shaft no. 1 of the Karnalit mine. The Kizil shaft was excavated from an elevation of 107 m to an elevation of 162.5 m in three stages by the method of A. I. Gertner, which consisted of introducing a mixture of lean clay into the crack. Calcium chloride was used to precipitate the clay particles, and after completion of the claying, Portland cement was applied to the cracks at a pressure of 60 to 80 atmospheres. During the second stage (124.7 to 141.7 m) the claying was done with a pure clay mixture containing calcium chloride and was completed with clay containing 10 to 15 per cent cement (instead of pure cement), which expedited the precipitation of the clay from the mixture. The cementation was done at a pressure of 50 to 60 atmospheres. The use of claying in combination with cementation made it possible to excavate the shaft through a karst limestone layer 55.5 m thick and discharging about 200 m^3 per hr, resulting in a saving of 20,000 barrels of cement, as compared with the cementation method.

Shaft no. 1 of the Karnalit mine was also excavated in three stages from an elevation of 11.7 m to an elevation of 49.7 m. According to the data of engineer I. A. Androsov, the following quantity of materials was used in excavating the Kizil and Karnalit shafts (Table 8).

TABLE 8

MATERIALS USED FOR STRENGTHENING THE GROUND

Material	Unit	Kizil Shaft No. 1			Karnalit Shaft No. 3		
		Stages					
		1	2	3	1	2	3
Clay	barrels	18,884	2863	18,499			
Clay mixture	m^3				500.4	1888.4	2981.8
Portland cement	barrels	8407	578	1496			
Water glass	barrels	224	6	18			
Calcium chloride	barrels	81	68	214	64		
Magnesium cement	m^3				3.90		
Table salt	kg	2399					
Meters per stage		17.70	17.00	20.80	8.15	15.00	12.00

In addition to economizing on cement, the use of claying yields the following advantages:

1. Interruptions in pumping are possible during claying, but not during cementation.

2. Concrete structures are protected from damage because the chemical reagents contained in the circulating water do not affect the clay.

3. The excavating operations are less difficult in claying than in cementation; hence, they are carried out faster.

4. The claying method can be used when digging through water-bearing strata containing various dissolved salts and acids which interfere with the setting of the cement or damage the cement.

5. The danger of cementing pipes, pumps, and hose is excluded.

6. The claying method is economical in case the cracks are re-opened for the purpose of a second packing.

B. Bituminizing

During erection of structures in soils having considerable flow velocity of ground water as well as large voids, the cement mixture applied under pressure is washed out; therefore cementation does not bring any positive results. In such a case, bituminizing is a reliable means of stabilizing the soil; this method was first used in 1926 by the engineer Christians during construction of the Hales Bar Dam on the Tennessee River.

In order to bituminize the foundation of the dam, 68 holes were drilled for a total length of 1876 m. The bitumen was forced through $1\frac{1}{2}$-in. pipes which had openings in the lower part through which the bitumen penetrated into the rock. The temperature of the bitumen in the pipes lowered into the holes was maintained constant by an electric current through a steel wire located in the middle of the pipe and supported on insulators. Use of the bituminizing method in the Hales Bar Dam has reduced the water seepage to a minimum.

Bituminizing was used on construction of the Palace of Soviets in Moscow as a waterproofing measure for the foundations built in an open pit, excavated through thick layers of greatly cracked and water-bearing limestones. The use of bitumen in the given case was advantageous because the reinforced concrete foundations were protected by the bitumen covering which is not affected by the action of aggressive water.

When the bitumen, which is applied under pressure, enters into the cracks of the rock and comes in contact with the cold water flowing in the crack, it hardens and forms an impermeable barrier. Petroleum bitumen having average coefficients of penetration and expansion is used for bituminizing. It is expedient to begin the bituminizing operations with bitumen of liquid consistency which fills the small cracks more reliably. In the construction of the Palace of Soviets in Moscow, bitumen no. 3 was used first and then bitumen no. 5.

The installation for bituminizing consists of the following elements: a bitumen heater, a pump for pumping the molten bitumen, and pipes connecting the feeder system to the crack (Fig. 24). The heaters used for heating the bitumen usually have a capacity of about 700 to 800 liters and are mounted on wheels to facilitate maneuverability and to reduce the length of the feeder piping. The heater has two sections: in the first section the bitumen is loaded and melted; in the second section the bitumen, freed somewhat from the precipitated admixtures, is kept at a temperature of 180° to 190° C corresponding to the temperature required for use. The bitumen leaves the heater through the upper pipe.

The heater is fired with wood, which assures uniform distribution of heat in the heating chamber. Observations show that if coal is used as fuel, an excessively high temperature is developed in the heating chamber, causing the bitumen to burn and the heater to become encrusted. Hence, use of coal is not suitable. If the effective capacity of the heater is 700 to 800 liters, then the firing chamber occupies 0.12 to 0.13 m^3. The bitumen should not be heated to a temperature higher than 200° to 220° C. The heater is equipped with a thermometer so that the temperature can be observed.

Reciprocating pumps having a capacity of 150 to 200 liters per hr are generally used to force the hot bitumen into the cracks; the pumps are driven by any prime mover rated at about 1 hp. The pumping is done through 38-to 50-mm gas pipes capable of withstanding pressures of 30 atmospheres. The pipe feeding the bitumen to the crack is equipped with tees and stopcocks. The pressure in the pipe is measured by manometers.

The holes may be drilled with any type of drill, provided the holes are strictly vertical, straight, and of the required diameter (75 to 100 mm). At larger diameters, the removal of the bituminizing pipes becomes difficult, in which case the pipes are lowered into the holes without wedging. The precision of hole size and distance between holes depends in each individual case on the water-bearing property and the thickness of the layers undergoing bituminization.

After the hole has been drilled, the pipes used in the drilling process remain in the hole until the arrangement of the bituminizing pipes and the plugging with cement have been completed. Then these pipes are removed from the hole during the first few hours before the cement in the plugs sets. At the completion of drilling, the hole is cleaned by blowing air or is washed out with water, and the system of bituminizing pipes is lowered into it.

The hot bitumen is pumped through extra-strong steel gas pipes or smooth steel pipes of 44.50-mm outside diameter and 38-mm inside diameter, manufactured under a pressure of 20 to 25 atmospheres. The pipes are cut into lengths not greater than 2 m because the insulators are to be spaced a maximum of 2 m apart in order to prevent the possibility of a short circuit. The ends of the pipe lengths are threaded. The part of the pipe in contact with the rock requiring bituminization is equipped with 18-mm perforations arranged in staggered rows and spaced 100 to 140 mm apart (Fig. 25).

Electric heating is used to prevent cooling of the bitumen during its passage through the cold pressure pipes; this heating is used also for reheating the cold bitumen in the pressure pipes during interruptions in the bituminizing process. The electric heating is done with low-voltage current (11 to 55 volts) by means of a 6-to 8-mm steel wire; the lower end of the wire is fastened to the lower end of the pressure pipe, and the upper end of the wire is fastened to a special supporting arrangement consisting of a system of springs. The leads from the power source are connected to the upper end of the steel wire. The steel wire is held in the center of the pipe by porcelain insulators located where the pipes are joined by couplings.

The holes are heated with electric current before the bituminizing is started. The first pumping of bitumen is stopped only when constant pressure is achieved. A constant discharge of bitumen, continued for several hours at a constant pressure, indicates that the cracks are filled. Pumping is discontinued for a period of 20 to 40 hours, and then a second pumping is begun and lasts for 20 to 30 minutes. The hot bitumen reaching the cracks melts the surfaces of the previously injected and cooled bitumen, so that these surfaces are firmly pressed to the cracks of the rock, completely plugging the cavities and shutting off the inflow of water. After the bituminizing has been completed, the installation in the upper parts of the hole is removed, while the pressure pipe with the steel wire (electric wire) remains in the hole until all operations in the capping zone are completed.

The filtration coefficient of the neighboring rocks is determined from test pumping. This coefficient indicates the degree to which the cracks have been filled with bitumen. If the filtration coefficient approaches that assumed in the design, then the bituminization of the hole is considered completed; otherwise, additional bitumen is pumped through the defective hole.

In addition to the operation given above, the following checking operations are carried out:

1. The pit is opened and the bituminized ground is uncovered.

2. Several holes are drilled, one of them being located centrally.

3. The water is pumped out from the central hole, and the curve of the drop in ground water is determined from observations on the other holes.

Additional bituminizing is administered if these tests give unsatisfactory results.

It is essential to keep a record of all the operations performed during the bituminizing process, including the following data: depth, diameter, and construction of the bore; depth of plug; stroke of pump piston; pressure in the supply pipe; magnitude of the heating current; duration and number of heatings applied to the bore; duration of pumping and quantity of bitumen used; number and duration of intervals between pumpings. According to the construction data of the Palace of Soviets, the cost of bituminizing operations amounted to 84.76 rubles per linear meter of bore.

Figures 26 and 27 are diagrams of bituminizing processes performed respectively from the surface and from a stope.

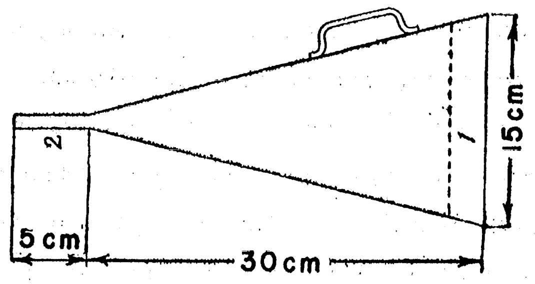
C. Freezing

The method of freezing of water-bearing strata has been known for a century. Natural freezing was used in Siberia to excavate shafts through sands saturated with water, when the location of a gold vein necessitated a shaft in the middle of a river. Construction of the shaft started in the winter after the ground had been subjected to natural freezing for three or four days. A fire was built at the proposed location of the shaft, the ground underneath the fire thawed out and was readily excavated, while the shaft walls remained frozen and retained their stability. These operations of successive freezing and thawing were repeated until the entire water-bearing layer had been penetrated. In this way it was possible to excavate shafts 20 m deep. Of course, this method of excavation was extremely slow and could be used only in the winter. Subsequently there occurred the idea of artificial freezing for purposes of excavation.

The freezing process is as follows: Special refrigerating pipes, in which brine cooled below 0° C is circulating, are lowered into bores drilled in the ground; as a result, the ground freezes and is converted into a stable mass in which excavating operations can be carried out readily and safely. The frozen ground is impermeable and has great mechanical strength, depending on its granulometric composition. The temporal compressive strength of frozen sand containing 16.50 per cent water (by weight), expressed in kilograms per square centimeter, can be determined from Albi's formula

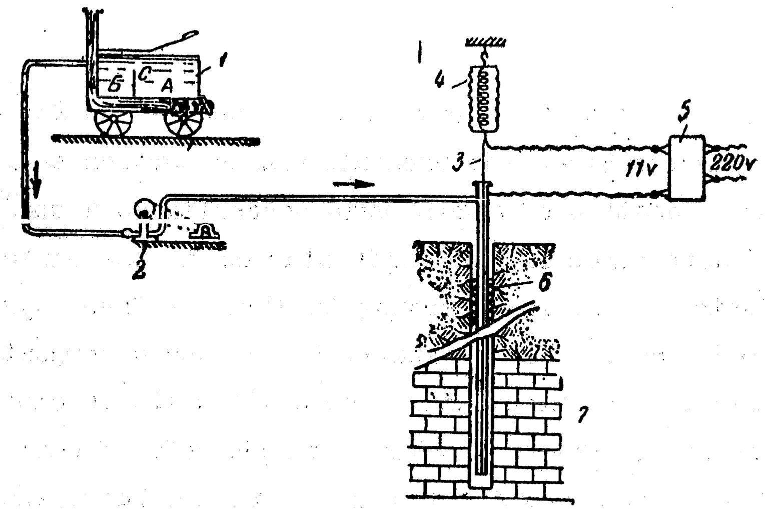
$$R = -0.153t^2 + 11t + 20 \qquad (19)$$

where $\underline{t}$ is the refrigeration temperature in degrees C.



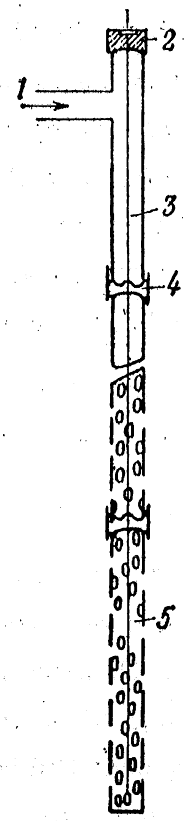
I. Sieve with 3mm mesh
2. Copper tube with 4.5mm outlet

Fig. 23 – Funel's Viscosimeter



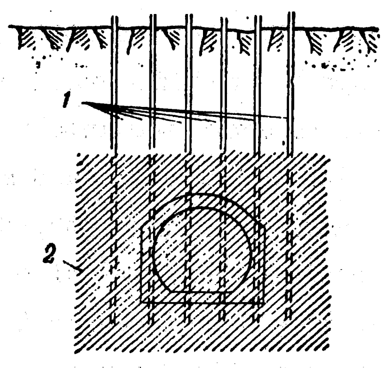
I. Heater 2. Pump 3. Heating wire 4. Spring
5. Transformer 6. Seal 7. Cracked stratum

Fig. 24 – Arrangement for Bituminizing



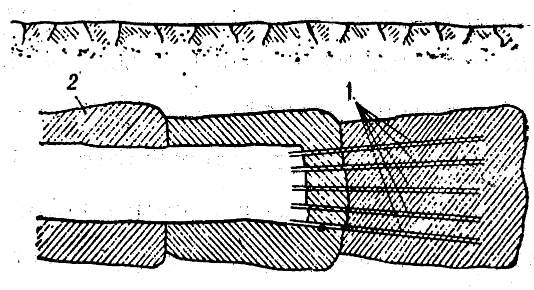
I. Bitumen insulator 2. Stuffing–box
3. Heating wire
4. Insulator 5. Perforated part

Fig. 25–Pressure Pipe



I. Pipes for injecting the bitumen
2. Cracked rock filled with bitumen

Fig. 26 – Bituminizing from the Surface



I. Pipes for injecting the bitumen
2. Cracked rock filled with bitumen

Fig. 27 – Bituminizing from a Stope

The results of a series of tests on mixtures of various materials, evaluated in accordance with Albi's formula, yield the data compiled in Table 9.

TABLE 9

TEMPORAL COMPRESSIVE STRENGTH OF VARIOUS REFRIGERATED MATERIALS IN KILOGRAMS PER SQUARE CENTIMETER

No.	Material	Component Parts			Refrigeration Temperature Degrees C			
		Water	Sand	Clay	-14	-15	-17	-25
1	Sand saturated with water	165	1000		144			200
2	Pure clay	1000		1000	78	65-80		
3	Sandy clay	500	1000	500	74			
4		200	1000	200		84-100		
5		125	1000	150		93-104		
6		1000	1000	100		104		
7		50	1000	50		84-100		
8		2000	1000	1000			94-104	
9		333	1000	200			90-113	
10		200	1000	125			18-122	
11		125	1000	125			17-127	
12		2000	1000	2000			94	

The required thickness d of the protective layer of frozen ground can be determined from the data of Table 9 in accordance with Lamé's formula

$$d = r\sqrt{\frac{R}{R - 2\ell k}} \qquad (20)$$

where

r is the radius of the shaft (roughly),

ℓ is the total lateral pressure of ground and water,

k is the safety factor, taken as 2 to 3, and

R is the temporal compressive strength of the frozen ground.

The magnitude d can be computed also from Kögel's formula

$$d = \frac{K\ell D}{2(R - \ell K)} \qquad (21)$$

where D is the shaft diameter (roughly) and the other notations are the same as in Lamé's formula.

If it is necessary to freeze the ground in order to install a horizontal underground structure (Fig. 28), then the required thickness of the walls and covering of refrigerated ground can be approximately computed from the following formulas:

Wall thickness

$$b = \frac{(H + f)\gamma ak}{R - (H + f)\gamma k} \tag{22}$$

where

H is the distance from the surface of the ground to the top of the structure,

f is the rise of the exterior contour of the arched part of the structure,

γ is the specific weight of the ground,

a is half the span of the tunnel (roughly),

R is the temporal compressive strength of the refrigerated ground, and

k is the safety factor, taken as 2 to 3.

The thickness of the layer of frozen ground under the tunnel

$$d_1 = \frac{-2\frac{R}{k}f + \sqrt{4\frac{R^2}{k^2}f^2 + \frac{R}{k}H(2a + b)^2}}{2R} \tag{23}$$

in which the notations are the same as in the formulas given above.

The following equipment is required for carrying out the refrigeration operations: (1) a refrigeration installation to generate cold, (2) a system of feeder pipes (brine pipes), and (3) a system of refrigerating pipes. The refrigeration installation consists of an ammonia or carbon dioxide compressor, a condenser, an evaporator, and a centrifugal pump for pumping the brine. Ammonia or carbon dioxide compressors are used for freezing the ground.

The refrigeration unit operates as follows: The ammonia gas enters from the evaporator into the compressor and is compressed to a pressure corresponding to that at which the gas changes into liquid state (9 to 11 atmospheres for carbon dioxide). The compressed gas flows into the condenser where, under the effect of cooling water, the ammonia passing through the coils transfers heat to the water and is liquified. The ammonia proceeds from the condenser through a regulating valve into the evaporator; during this process the pressure is lowered from 8 to 10 atmospheres to 1 to 1.5 atmospheres, which again results in a change from liquid into gaseous state, the heat required for this change being absorbed from the brine.

The required temperature of the frozen ground and the temperature of the cooling brine are determined from the hydrogeological data. For refrigeration at shallow depth, the temperature of the brine usually is -25° C, while the ground is cooled to about -15° C. For refrigeration at large depth, the temperature of the cooling brine is lowered to -45° C, and the ground is cooled to -25° C. Having established the temperature, the thickness of the ring and the volume of the frozen mass can be determined from Lamé's formulas.

The allowable compressive stress of the frozen sand, in relation to the ratio of the height of the uncovered part of the frozen ring to its inside diameter, is determined from the data of Table 10.

TABLE 10

TECHNICAL INDEXES OF FROZEN GROUND

Ratio of height of uncovered part of frozen ring to its inside diameter	1.71	0.43	0.33	0.17	0.17	0.17
Temperature of the frozen ring in degrees C	-10	-10	-10	-10	-20	-25
Allowable stress in kg/cm^2	43	86	103	124	150	175
Hydrostatic pressure in kg/cm^2	17	34	41	50	60	70
Limiting depth of the refrigerated sand in meters	140	280	330	400	480	560

Having determined the volume of ground requiring refrigeration, the necessary quantity of kilogram-calories can be computed from the formula

$$Q = (V - V_1)\, 1000 p\, (t_o - t_1) C + 1000\, V_1\, (t_o + 79 - 0.5 t_1) \qquad (24)$$

where

V is the volume of refrigerated mass in cubic meters,

V_1 is the volume of water in V,

p is the specific gravity of the ground,

C is the specific heat of the ground,

79 is the latent heat of liquefaction,

0.5 is the specific heat of ice,

t_1 is the mean temperature of refrigeration, and

t_o is the mean temperature of the ground prior to refrigeration.

Use can be made of the specific heat values given in Table 11.

TABLE 11

SPECIFIC HEAT OF SOILS

Name and Type	Specific Gravity	Specific Heat
Limestone	2.5 - 2.8	0.25
Clay	1.5 - 2.85	1.00
Sandy rocks	2.2 - 2.5	0.22
Sand	1.4 - 2.0	0.20
Dry stony soils	2.0 - 2.8	0.20

To determine the actual quantity, including losses, the value obtained from formula (26) should be doubled. The capacity of the refrigeration installation can be determined by means of the following calculations.

EXAMPLE: A shaft is refrigerated to a depth of 200 m. Inside diameter = 6 m, thickness of frozen wall = 2 m, mean temperature of ground = 14°C, freezing continues to $t_1 = -15^\circ$C, specific heat of the ground = 0.2, specific gravity of the ground = 2.7, porosity of the ground = 25 per cent. The ground is fully saturated with water.

SOLUTION: 1 m^3 of the ground contains 0.25 m^3 of water and 0.75 m^3 of soil particles.

Volume of cylindrical ring

$$V = \frac{\pi}{4} H (D^2 - d^2) = \frac{3.14}{4} \times 200 (10^2 - 6^2) = 10,000 \ m^3$$

Quantity of water in the cylindrical ring

$$V_1 = 10,000 \times 0.25 = 2500 \ m^3$$

Theoretical evaluation of the quantity of cold

$$Q = (V - V_1) \times 1000 p (t_o - t_1) C + 1000 V_1 (t_o + 79 + 0.5 t_1)$$

$$Q = (10,000 - 2500) \times 1000 \times 2.7 (14 + 15) \times 0.2 + 1000 \times 2500 (14 + 79 + 0.5 \times 15)$$

$$Q = 117,000,000 + 250,000,000 = 367,000,000 \text{ kg-cal}$$

Actual quantity, including losses

$$Q = 2 \times 367,000,000,000 = 734,000,000 \text{ kg-cal}$$

To determine the capacity of the refrigeration installation, it is necessary to know the number of holes as well as the diameter and thickness of the refrigerating pipes. If the bores are arranged along the perimeter of the ring, then the surface of the cooling pipe is

$$F = (d + 2\sigma)\, Hn$$

where

d is the pipe diameter in meters,

σ is the thickness of pipe walls in meters,

H is the depth of bores in meters, and

n is the number of bores.

The average transfer of cold from 1 m^2 of surface of cooling pipes to the soil is q = 200 to 240 cal per hr. Hence, the theoretical capacity of the refrigeration installation must be

$$N_m = Fq$$

Considering the losses occurring during the transfer of cold from the refrigerating machine to the bores and back, the actual capacity required is

$$N = 1.15\, N_m \text{ cal per hr}$$

The required duration of refrigeration is

$$T = \frac{Q}{N_m}$$

Ordinarily the holes are arranged around the trunk of the shaft at a distance of 3 to 3.5 m from the external circumference. In unstable soils frozen to a great depth, where considerable thickness of frozen wall is required, two or three frozen rings are used. The distance between the holes located on the circumference of the frozen ring depends on the time specified for the operations, the depth of freezing, the hydrogeological and physical properties of the ground, and the cost of drilling operations. In strong and stable, but highly water-bearing soils, the distance between holes is usually greater than 1 m, while in weak and unstable soils the holes are spaced 1 m apart or less.

Care should be taken during drilling operations to keep the bore strictly vertical since deviation of more than 1 to 2 per cent from the vertical may lead, in the case of deep bores, to formation of unfrozen spaces permeable to water and quicksand. If greater deviations do occur, it is occasionally perferable to drill supplementary bores, usually numbering about 10 per cent of the original bores, instead of straightening the crooked bores. Since the bores have to accommodate the refrigerating pipes, they should be not less than 150 to 200 mm in diameter.

The refrigerating pipes (Fig. 29) are made from gas pipes 75 to 100 mm in diameter. The lower end is welded together, while the upper end is equipped with a nipple through which passes a slender pipe 25 to 28 mm in diameter. A second, short pipe is welded to the nipple and carries the returning brine. The first pipe is connected to the feeder main, while the second pipe is connected to the return main. Thermometers are installed in both pipes for the purpose of observing the temperature of the inflowing and outflowing brine.

After all the bores have been drilled and the system of refrigerating pipes has been installed in the bores, these pipes are connected to the distributing and collecting pipes which, in turn, are connected to the mains leading to the refrigeration unit. The cooling brine is pumped into the bores, and the freezing operation is conducted by either the parallel or successive method. Figure 30 shows the process of gradual formation of the frozen mass by parallel freezing. The merit of this method is its simplicity. Some of its numerous shortcomings are: the difficulty of controlling the distribution of cold, nonuniform operation of the refrigerating machines working under varying temperature, the possibility that the solution would escape, and other damages to the refrigerating pipes. Moreover, when all the bores are cooled simultaneously along the entire contour, water pockets may form at the boundary between two frozen masses; the freezing of the water in these pockets surrounded by frozen masses becomes difficult because the water pockets oppose the expansion of the water during freezing. The resulting pressure in the water pocket may cause squashing of the refrigerating pipes.

In the successive method (Fig. 31) the cooling brine is admitted simultaneously only to separate groups of bores consisting of two or four diametrically opposite bores. The brine is circulated in these bores until the temperature in the adjacent bores has reached the freezing point, at which time the circulation of the brine is transferred to the latter bores. Thus, the gradual growth of the frozen layer is achieved by successive inclusion of new adjacent bores as soon as their temperature is lowered to the freezing point. The advantage of this method is the more complete control and regulation of the process, effected through measurement of the temperature of the ground at various depths in the bores in which brine is not yet circulating. In addition, the use of the successive method allows for a smaller capacity and a greater efficiency of the refrigeration unit.

It may be concluded that the parallel method is preferable for freezing at shallow depths, while the successive method is advantageous for deep shafts and at large flows of ground and saline water. Control of the freezing process is carried out in special bores by measuring the temperature of the ground or filling the bore with water. If the level of the water admitted into the bore is lowered to the level of the ground water, then complete freezing has not yet occurred. On the other hand, if the level of the water admitted is not lowered, then the mass is already sufficiently frozen.

In the refrigeration operations for shaft no. 9 of the Moscow subway, approximately 100 bores were drilled in three rows located 1 m apart and having a total length of 600 linear m. The bearing capacity of the ground frozen to -15° C was as follows:

114 kg/cm^2 for sand with water content of 18 per cent
113 kg/cm^2 for clay with water content of 24 per cent
62 kg/cm^2 for fine moraine sand with water content of 15 per cent
84 kg/cm^2 for coarse moraine sand with water content of 13 per cent

The excavation of the tunnel has shown that the radius of freezing reached 1.75 m. The entire mass above the arch of the tunnel has been frozen. The full volume of the frozen mass was 1000 m^3. The freezing cost 75 rubles per 1 m^3. The quantity of kilogramcalories per 1 m^3 amounted to 48,000.

D. Chemical Hardening

The chemical method of hardening the soil consists of successive forcing of two chemical solutions into the ground through a system of metal pipes. As a result of the physico-chemical processes occurring between the two solutions, a hydro-gel of silicic acid is formed, rapidly and firmly hardening the ground. The first solution is liquid glass (sodium or potassium salt of silicic acid), and the second solution is calcium chloride.

When the liquid glass is applied under pressure, it displaces the water from the ground and fills all the free pores between the soil particles.

Thus, after the first solution has been injected, a mass of soil saturated with liquid glass is formed around the injector. During the further application of calcium chloride solution under pressure, this solution displaces the silicate in the pores of the soil; however, part of the liquid glass remains on the sand particles and undergoes a chemical reaction with the calcium chloride. As a result of this chemical reaction, every sand grain becomes covered with silica gel and, due to the cementing properties of this gel, a firm homogeneous mass is formed. On account of the silica gel deposited on the sand grains, the strata are strongly sealed and the ground becomes impermeable to water (Fig. 32).

Only sandy, rather permeable soils can be rendered impermeable by the process described above. Loam and clay, with a percolation coefficient of less than 2 m per day, cannot be hardened in this way because it is extremely difficult for the solutions to penetrate into the ground.

This method is not recommended for strengthening coarse conglomerate soils having a percolation coefficient greater than 80 m per day because the resulting gain in strength is slight. The method of chemical hardening can be successfully applied to sandy soils composed of fine grains, medium grains or coarse grains, having a percolation coefficient of 2 to 80 m per day. The strength of the chemically hardened soil depends on the size and chemical composition of the particles, the chemical composition of the hardening materials, the concentration of the solutions, and the purity of the original materials. The temporal strength of the silicated soil varies between 20 and 80 kg per cm^2, occasionally amounting to 100 kg per cm^2.

The strength of the chemically hardened soil increases with time. The soil attains one-half its ultimate strength during the first two hours after the hardening process. Further intensive increase in strength occurs during the first ten days, and afterwards the increase proceeds much more slowly. Find sands attain the greatest strength, while coarse sands attain the least strength.

Chemically hardened ground is impermeable to water and has greater resistance to the corrosive effect of aggressive ground water than concrete made of ordinary Portland cement. Salt and acid solutions do not have any corrosive effect on chemically hardened soil, but alkaline solutions and free carbon dioxide corrode this soil. The resistance of chemically hardened soil to frost action is slight; hence, it is not recommended to harden the soil at the surface.

Table 12 gives the results of Prof. Gutman's tests on hardened soil specimens kept in various chemical solution.

TABLE 12

STRENGTH OF TEST SPECIMENS OF HARDENED SOIL

Solution in Which the Chemically Hardened Specimens Were Kept	Temporal Compressive Strength in kg per cm^2				
	7 days	28 days	6 mos.	1 yr.	2 yrs.
Magnesium chloride	18.9	20.4	21.2	16.3	22.4
Humus solution	29.9	26.0	25.1	18.6	19.2
Hydrogen sulphide	23.8	20.7	24.9	19.1	17.6
Magnesium sulphate	15.1	17.0	13.0	16.2	12.4
Air	26.5	19.0	17.2	19.2	17.4

In the process of preparing liquid glass, the molten glass is poured into cast iron molds where it cools and cracks into large chunks because of the difference in stress at the surface and in the interior; the large chunks are broken later into smaller pieces. The pieces of silicate are placed in autoclaves or silicate heaters and are converted into liquid glass of required concentration.

Liquid glass is a water solution of sodium silicate; its chemical formula is

$$R_2On \times SiO_2$$

where R_2O is oxide of sodium or potassium, and n is the quantity of silica.

The modulus of the liquid glass, that is, the ratio of the silica to the sodium oxide, SiO_2/NaO, has the greatest effect on the strength of hardening; this modulus has the value of 2.5 to 3.5. The hardened soil attains its greatest strength when the value of the modulus is 2.7 to 3.0. Any further increase in the value of the modulus does not result in increasing strength, but, on the contrary, causes a considerable decrease in strength.

The values presented in Table 13 characterize the effect of diluting the silicate with water on the strength of hardening.

TABLE 13

HARDENING STRENGTH IN RELATION TO DILUTION OF SILICATE WITH WATER

Percentage of Water by Volume of Molten Glass	Strength in kg per cm^2
0	71
20	56
40	49
60	9
80	3

Table 14 gives the limits of concentration established in practice in relation to the modulus of the liquid glass used.

TABLE 14

CONCENTRATION AT 18° C

Modulus	Specific Gravity	Degrees Baumé
2.60	1.48 - 1.53	47 - 50
2.75	1.42 - 1.48	43 - 47
3.00	1.40 - 1.45	41 - 44

The greater the viscosity of the liquid glass, the more difficult its penetration into the thickness of the soil and the greater the load on the pumps. On construction jobs, therefore, the tendency always is to dilute the glass to liquid consistency. This procedure is not permissible because it lowers the strength of the ground. To lower the viscosity, the glass is heated before it is pumped. The greatest decrease in viscosity occurs at a temperature of 40° to 50° C. Hence, taking into consideration the cooling occurring in the pipes, the glass is usually heated to 70° C.

The silicate (liquid glass) used for hardening the ground should satisfy the following technical conditions:

1. The modulus of the silicate solution, SiO_2/NaO, should be not less than 2.60.

2. The concentration of the silicate solution at modulus 2.6 should be not less than 47° to 80° Baumé, corresponding to a specific gravity not less than 1.48 to 1.49.

3. The quantity of materials undissolved in water should be not greater than 1 per cent.

4. The silicate of the solution should be transparent or only slightly turbid, colorless or faintly colored, the color ranging from greenish-yellow to brownish-yellow (black color is not allowable).

Liquid glass may be kept in wooden containers.

Calcium chloride, which is the second chemical material used in the hardening process, is a by-product in the manufacture of soda or Berthollet salt (potassium chlorate, $KClO_3$). The calcium chloride obtained during the manufacture of soda has a low concentration (8 to 10 per cent) and a considerable quantity of foreign matter. The manufacture of potassium chlorate yields calcium chloride of higher concentration (26 to 40 per cent) and purity; therefore, this solution is generally used for hardening purposes.

The calcium chloride solution must satisfy the following technical conditions:

1. The solution should contain not less than 350 g of $CaCl_2$ per liter, which corresponds to a concentration of specific gravity 1.26 or 30° Baumé.

2. The amount of $CaCO_3$ admixture should be not greater than 30 g per liter, the suspended particles weighing not more than 0.2 g.

Considering the strength of the resulting hardening and the cost of the solution, calcium chloride occupies an exclusive place among all the other chlorine salts of bivalent and trivalent metals. The following experimental results are an illustrative example:

$MgCl_2$ - 28 kg/cm^2

*NaCl - 35 kg/cm^2

$BaCl_2$ - 47 kg/cm^2

$AlCl_3$ - 5-8 kg/cm^2

$FeCl_3$ - 5-8 kg/cm^2

*The original text shows $NaCl_2$.

It is evident from these data that $BaCl_2$ gives the best results, but its salts are very expensive and, therefore, cannot be used on a large scale.

When calcium chloride is prepared in tanks having an acid intensity (pH = 4 to 5), explosions may occur when the tanks are opened, since chlorine dioxide may be formed, in the presence of admixtures of potassium chlorate and free hydrochloric acid, and may react on the metal to form free chlorine. Proper precautionary measures must be taken in such cases. Metal tanks are quite suitable for storing and transporting liquid calicum chloride.

The following equipment is required for the operations of chemical hardening of the ground: (a) injectors, that is, metal pipes which are inserted into the ground and through which the chemical solutions are injected, (b) a pipe network feeding the solutions to the injectors, (c) pump installations, and (d) auxiliary installations and silicate heaters. Figure 33 shows an injector used for injecting the solution.

The distribution system consists of the following: metal pipes 50 to 65 mm in diameter, tested at a pressure of 30 atmospheres; tees; measuring devices for determining the quantity of solutions discharged; and hose connectors. If the volume of work is small, the distribution system may consist of thick, high-pressure hose. In the winter the system should be heated to prevent the cooling of the solution flowing in the pipes or hose.

The couplings used are either American nuts designed by Roth or pneumatic railroad and trolley couplings. A distributor is installed near the place where the injectors are inserted; it services five to six injectors and consists of a 50-to 65-mm pipe the ends of which are connected, respectively, to the feeder pipe and to a removable hose. This pipe has a valve regulating the pressure in the distributor and a manometer measuring the pressure. The five or six branches, each 25 to 37 mm in diameter, leading from this pipe to the injectors are equipped with a valve, a flow meter, and a manometer, and end with a hose connecting to the injectors. Four or five injectors can be connected to each branch of the distributor, so that one pump can service 25 to 30 bores simultaneously. The disadvantage of this method is the impossibility of accurately measuring the quantity of reagents forced into the hole. Yet the accuracy of determining the amount of solution forced through the injector into every hole is very important for both hardening of the ground and the masonry; therefore, the meters are an essential part of such a distributor.

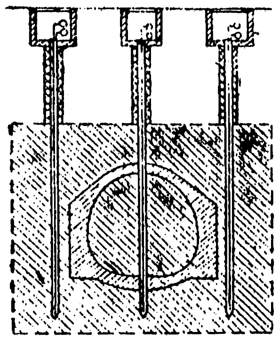


Fig. 28 – Diagram of Freezing for Subsurface Operations

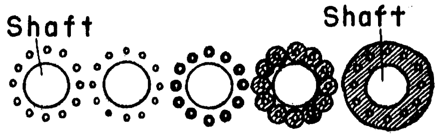


Fig. 30 – Frozen Ring Formed by the Method of Parallel Freezing

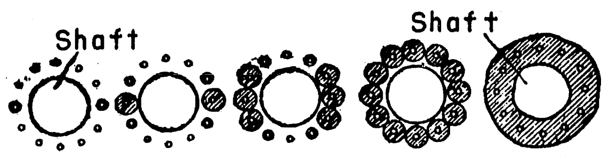
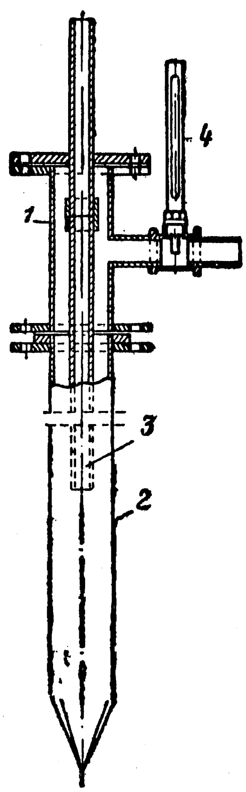


Fig. 31 – Frozen Ring Formed by the Method of Successive Freezing



1. Cap of refrigerating pipe
2. Refrigerating pipe
3. Feeder pipe 4. Thermometer

Fig. 29 – Installation for Freezing the Ground

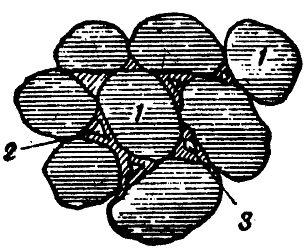
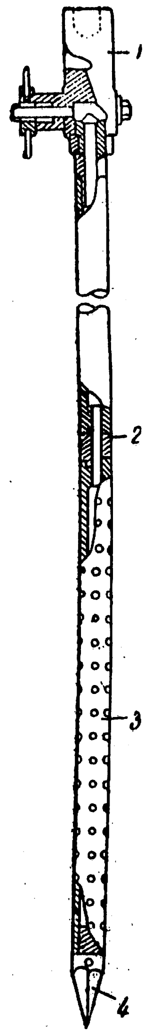


Fig. 32 – Deposition of Silica Gel in Sandy Soils

1. Grain 2. Free pore-passage after application of calcium chloride 3. Silica gel



1. Cap 2. Coupling nipple 3. Injector 4. Tip

Fig. 33 – Injector

The chemical solutions are pumped by high-pressure plunger pumps having spherical valves; the pumps can be driven by any prime mover. The capacity of the pumping installation is determined by the output required for simultaneous operation of the given number of injectors. The working pressure does not exceed 30 to 40 atmospheres. In the case of small volumes of work, the solutions may be pumped with hand pumps, that is, so-called hydraulic presses widely used in industry for hydraulic testing of steam boilers. One hydraulic press is sufficient for pumping 100 to 150 liters per hr at a pressure of 25 to 30 atmospheres, and it is serviced by one man.

Pumps of this type usually are assembled in groups of four to six and are operated by an electric motor. Multiple-piston pumps having a separate intake for each cylinder require a large amount of hose connecting the pumping installation with the site of operations and necessitate a complicated signaling system between the site of operations and the personnel servicing the pump. The use of multiple-piston pumps is recommended for operations of chemical hardening of shafts and tunnels where the quantity of reagents used is relatively small and the use of stationary pumps is not feasible. The maximum output of the pump is 30 to 40 liters per min at a pressure of 30 atmospheres; hence, one pump is sufficient for hardening 50 m^3 of ground per day. A Yjorsk pump requires a 4- to 5-kw motor.

When the reheated silicate is cooled, it tends to become crystallized and precipitates very fine particles of silica gel which adhere to the fine clearances of the pump. It is thus necessary to rinse the pump and the other apparatus with hot water once every day and after each interruption of pumping.

The injectors can be readily driven into the ground and withdrawn by a driving hammer or a perforator. For this purpose, a special tip fitting snugly into the cap socket is inserted into the driving hammer or into the drill holder of the perforator. The withdrawal can be carried out with the same tools if a special clamp is used on which the driving hammer and perforator act in the reverse directions.

The injectors can be forced into the ground by special electric drivers; however, these drivers are not suitable for the job if the injectors are to be driven in an inclined position or horizontally. If electric drivers are used, the injectors have to be withdrawn with the aid of a hoist or winch.

A silica heater is required for the operations of chemical hardening. The most suitable heater is that designed by Nojnikov. It consists of a wooden tank having the shape of a truncated cone of the following dimensions: bottom diameter = 2300 mm; top diameter = 2100 mm; effective height = about 2500 mm. The tank is made of planks 50 to 80 mm thick and is equipped with a cover. Inside the tank, at a height of 50 to 60 cm from the bottom, is mounted a grate made of boiler steel 6 to 8 m thick. Perforated steam pipes, bent in the shape of a circle, are located under the grate. Samples of the liquid glass are drawn from several cocks installed in the tank, and the finished product is removed through a discharge cock 100 mm in diameter.

In order to prepare the plans for the chemical hardening operations, it is necessary to have the data of hydrogeological investigations, laboratory analyses of the ground and water, and experimental studies conducted at the site of the planned hardening operations. In addition, consideration must be given to the requirements which the strengthened ground is to meet, to the functional purpose of the structure, and to the objective accomplished by the process of chemical hardening. In preparing the plan, it is necessary to determine the radius of the hardening action and the volume of the hardened mass. The radius of the hardening action denotes the distance reached by the injected chemical materials. This radius varies within the following limits, depending on the coarseness of the sand and the percolation coefficient: 0.20 to 0.25 m for fine sands with a percolation coefficient of 2 to 5 m per day; 0.25 to 0.50 m for medium sands with a percolation coefficient of 5 to 15 m per day; 0.50 to 0.60 m for coarse sands with a percolation coefficient approaching 80 m per day.

In planning the operation, consideration should be given to the fact that the liquid glass is injected in several stages during which the injector is gradually lowered into the ground to the required depth, whereas the calcium chloride is injected during the withdrawal of the injector. The injectors should be driven into the ground to a depth not greater than 6 to 8 m because it is difficult to maintain the original direction at large depths. The quantity in liters of liquid glass injected into the ground is

$$V = 6AP$$

where $\underline{A}$ is the volume in cubic meters of soil requiring hardening and $\underline{P}$ is percentage porosity of the soil, determined in the laboratory.

The quantity of calcium chloride injected into the ground is 20 per cent greater than the quantity of liquid glass. Thus, the quantity of liquid glass and calcium chloride injected into the ground can be determined with sufficient accuracy for each stage. A different injection procedure is recommended if considerable seepage velocities occur, since the danger arises that the liquid glass injected in the first stage will be carried away before the calcium chloride is forced into this space. In such a case the calcium chloride is forced into each interval immediately after the injection of the liquid glass; that is, liquid glass is forced into the injector driven to a depth corresponding to the first stage, and immediately afterwards calcium chloride is forced into the injector. Then the injector is driven to the next stage and the pumping operation is repeated.

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ABSTRACTS OF SCIENTIFIC RESEARCH WORK FOR 1945

of the

OBRUCHEV INSTITUTE OF FROST SCIENCE

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INTRODUCTION

In 1945 the Institute of Frost Science investigated three problems: (a) genesis of permafrost and theory of freezing process, (b) conditions for stability of structures in regions of permafrost, and (c) seasonal freezing of soil and ground, and seasonal reservoirs of cold.

With regard to the first problem, investigations have been conducted with the objective of determining the nature of the freezing process and the laws of the dyanmics of the phase composition of frozen grounds. This was done for the purpose of developing methods for adjusting the properties of various grounds during their freezing and thawing.

In the experimental field, the following studies are noteworthy:

1. The phase composition of frozen grounds (particularly the nonfreezing water component).

2. The thermal constants of grounds.

3. The freezing temperature of grounds.

4. Preliminary work on possible application of roentgenographic methods to studies of frozen, freezing, and thawing grounds.

Because of the scientific newness of the questions under consideration, the main objective of the experimental studies of 1945 was to establish methods for determining the various constants of frozen grounds. It was found experimentally that the temperature of freezing and overcooling is lowered with increasing density, with decreasing moisture, and when the absorbing complex of the grounds is saturated with cations. Lowering the temperature of the surrounding medium results in lowering the freezing point, but lessens the overcooling. The experimental results radically change the established notion of the nature of frozen grounds, and must be considered when solving practical problems relating to design and construction of foundations for structures.

Considerable progress in the general field of the given problem was achieved by the investigation of the spacial extent of permafrost. These investigations were most extensive in the region where the Yakutsk frost station was active. The systematic drying of lakes and active layers of plowed areas in that region caused the problems of water supply to become particularly serious in recent years.

In 1945 the completed investigations and the compilation of all other data available for the region of permafrost in USSR enabled V. F. Toomel to draw a new permafrost map to a scale of 1:10,000,000. The boundaries of permafrost are shown more accurately on this map, and an attempt has been made to indicate the territories characterized by difference in thickness of permafrost. The latter fact constitutes a definite achievement in comparison with previous maps.

With regard to the second problem, investigations were conducted with the objective of establishing rational methods for design and construction of various engineering structures to be erected on permafrost. Direct studies of construction in the field were made by the frost stations of the Institute and particularly by its Yakutsk station.

In addition to observations of mine construction in the Varkutsk region of continuous permafrost, the Institute has performed extensive work on the Kol Peninsula in response to requests by the Narkomchemprom that the Institute of Frost Science participate in solving problems related to operation of the mine. These problems arose with the occurrence of icing in the mine during 1941-42; the icing reached such proportions that any further normal operation of the mine became impossible. Long observations made it possible to establish the causes of the icing in the mine and to evolve methods for its elimination.

A special phase of this problem was the utilization of the winter cold for construction purposes, which was most effectively realized in the case of isothermal ice storehouses.

With regard to the third problem, a series of methodological and surveying operations have been carried out. These operations constitute a continuation or completion of the series of the preceding year.

A B S T R A C T 1

Adrianov, P. I., THERMAL PROPERTIES OF SOILS AND GROUNDS

This paper presents the current theory of the phenomena, the methods of measurement, and the characteristics of all typical soils of the European USSR. The author's original concepts are contained in the chapters dealing with theory and methods. For the first time the quantitative characteristics of thermal capacity, heat conduction, and temperature conductivity are given for 70 typical soils of the Soviet Union. The soils are in their natural formation according to genetic levels, and the characteristics are determined for a depth of 1 m under two moisture conditions--maximal molecular and full moisture capacity. The chapters dealing with temperature, heat capacity and thermal conduction contain illustrative material, a large number of tables, and a bibliography.

ABSTRACT 2

Bojenova, A. P., MIGRATION OF WATER IN FROZEN GROUNDS IN RELATION TO THE DEGREE OF THEIR MOISTNESS AND COMPACTNESS

The migration of water in frozen grounds has been investigated under the following conditions:

a. Different degrees of saturation with water, but similar porosity of the ground.

b. Different degrees of saturation with water at various porosities of the ground.

In the first case, with decrease in saturation the ground changes from the two-phase state (ground skeleton plus water), to the three-phase system (ground skeleton plus water plus air). In the second case, the ground remains a two-phase system at all degrees of saturation with water.

It was found that the intensity of migration of the water decreases with decrease in saturation of the grounds. If the size of porosity does not change with decrease in saturation and the ground changes to the three-phase system, then the presence of air pores in the ground system exerts the predominant influence on the intensity of migration of the water during freezing. The moisture limits at which the migration of the water occurs intensively are narrow. On the other hand, if the porosity size decreases with decrease in saturation and the ground remains in the two-phase state, then the intensity of the water migration depends on the quantity of free water contained in the system. The moisture limits at which the water migration is intensive are wider than in the case of the three-phase state of the ground.

A B S T R A C T 3

Bojenova, A. P., THE EFFECT OF HEAT-INSULATING COVERS ON THE MIGRATION OF WATER IN FROZEN GROUNDS

An investigation was made of the effect of heat-insulating covers on the migration of water in frozen sandy and loamy soils. The observations were made on monoliths of 175 by 20 by 8 cm.

The grounds were moistened until they were fully saturated. The monoliths were cooled only from the top. Individual parts of the monoliths were covered with "Torfoleum," with metal, or were left uncovered. As a result, the temperature regime of the different parts varied. The sand and loam monoliths were subjected to freezing simultaneously. The effect of heat-insulating covers on the migration of water is different for sand and for loam soils. In sandy soil the migrating stream proceeds from the colder layers towards the warmer layers. Consequently, an accumulation of water occurs below the heat-insulating layer, accompanied by the formation of ice lenses at the boundary separating the insulator from the soil. The parts that are not covered with insulation lose their water. In loamy soil, in contrast, the migrating flow proceeds from the warm layers towards the layers of more intensive freezing; consequently, accumulation of moisture occurs in the upper zone of the parts that have no heat-insulating covers. The parts covered with insulation lose the water.

A B S T R A C T 4

Bojenova, A. P. and Grigorieva, V. G., INVESTIGATION OF THE FREEZING TEMPERATURE OF GROUNDS

Experiments on loam have been conducted for the purpose of determining the influence of the following factors on the freezing temperature and overcooling temperature of grounds: (a) degree of compactness and moisture content of the ground, (b) temperature of the surrounding medium, (c) cation composition of the absorbing complex of the ground, and (d) preliminary cooling of the ground in the air-dry state to a temperature of about -20° C.

The following results were obtained:

1. The freezing temperature and the temperature of overcooling during freezing are lowered with increasing degree of compactness of the loam and with decreasing moisture content.

2. The freezing temperature of the loam is lowered when the temperature of the surrounding medium is lowered; its overcooling during freezing, however, is lessened.

3. Preliminary cooling of the loam in the air-dry state to a temperature of about -20° C results in lower temperatures of both freezing and overcooling of the loam.

4. Both the freezing temperature of the loam and its temperature of overcooling during freezing are lowered when the absorbing complex is saturated with the cation Na.

ABSTRACT 5

Grave, N. A. and Chernukhov, P. G., SOME OBSERVATIONS OF PERMAFROST, GEOLOGY, AND HYDROGEOLOGY IN THE REGION OF THE KHALMER-U RIVER

A new source of coking coal of great industrial importance has been discovered recently at the headwaters of the Karataikha and Kara Rivers, located at the extreme northeastern part of European USSR in the basin of the Khalmer-U River. In the summer of 1944 the authors carried out field studies in that region primarily for the purpose of determining the permafrost and hydrogeological conditions of the region. Parallel with these studies, some observations were made of the geology of quaternary deposits. These deposits are represented by moraine on the eluvium of the original soils, covered by lake deposits corresponding to the period of the postglacial optimum. The latter deposits are overlaid by alluvium in the valleys and by diluvial surface loams at the divides.

The thickness of the permafrost was found to be 118 m, according to the available section, but in many places it is apparently considerably thicker. Directly under the permafrost were found reservoirs of water under pressure.

ABSTRACT 6

Dementiev, A. I., Rosenthal, N. I., and Ianovski, V. K., METHODS FOR ANALYZING ENGINEERING EXPERIENCE ON PERMAFROST

Evaluation of the methods and determination of the conditions necessary for stable construction in regions of permafrost are possible only if the available engineering experience is studied and taken into account. However, such an analysis cannot be made without a procedure which is well planned and thoroughly worked out. Factual data large in scope but not compiled in accordance with a specific procedure loses greatly in value. Important omissions in one case and noncomparable data in the other case make it impossible to draw well-founded conclusions from the factual material.

The present paper deals with the procedure for evaluating data of construction on permafrost. It is given in the form of a questionnaire containing all the questions necessary for investigating the completed structures. In addition to detailed hydrogeological and geographical characteristics of the area under construction, the report should include the detailed thermometric characteristics of the area and a complete description of the type of permafrost encountered in the area. Information must be given about all the aspects of the engineering projects under consideration--constructional, technological, and chronological--including all the particulars pertaining to both the period of construction and operation. Data should be presented about all the sanitation and service auxiliaries, as well as the method and time of construction.

ABSTRACT 7

Dostovalov, V. N. and Korkina, P. I., INVESTIGATION OF THE DIELECTRIC CONSTANT AND SPECIFIC RESISTANCE OF FROZEN AND UNFROZEN GROUNDS IN RELATION TO MOISTURE AND TEMPERATURE

The first cycle of orientational experiments was conducted in 1945 in the frost laboratory of the Institute of Frost Science. The resulting curves, showing the variation of the dielectric constant with freezing of moist soil, characterize the freezing process in many details that could not be discerned by any other means. It is particularly noted that, in addition to the freezing of the main mass of gravitational water at temperatures near 0° C (about -7° to 13° C), the remaining water apparently located in the thinner capillaries freezes also.

The report on this work was prepared by V. N. Dostovalov. It discusses the objective of the investigation and describes the experimental procedure, apparatus, and results; it draws conclusions from the obtained data and specifies further phases of the investigation.

A B S T R A C T 8

Efimov, A. I., FORMATION OF THERMOKARST LAKES IN THE YAKUTIA REGION

The article is based on permafrost and hydrogeological studies made in 1945 in the Churapcha region at the Leno-Amginsk Divide on a territory of extensive formation of areas of ground ice in the form of layers 15 to 20 inches in thickness. Local melting of the ground ice results in the formation of a large number of thermokarst lakes.

The following groups of lakes exist in the region:

1. Lakes of river origin which are basically old river beds.

2. Lakes of thermokarst origin, having a short yearly contact with the surface streams.

3. Lakes which at present have lost their contact with surface streams because the streams carried no water for many years.

4. Lakes of thermokarst origin, having no contact with streams and feeding on water flowing down mountain slopes.

5. Lakes of thermokarst origin outside the range of surface flow, located on elevated, flat, contour elements. This group of lakes can be classified as (a) forming lakes, (b) formed lakes, and (c) drying lakes. The final stage of these lakes, as well as the lakes of groups three and four in some cases, is represented by dry depressions.

Each group of lakes has specific genetic characteristics manifested by the difference in size and in the chemical composition of the water. This fact determines the unequal importance of each group of lakes as a source of water supply.

The arid climate of the Central Yakutia region (annual precipitation about 200 mm, evaporation about 350 to 400 mm) causes drying of the lakes that are insufficiently replenished by surface waters. The lakes of all groups have a high mineral content, except the lakes of groups two and three which are characterized by a relatively low mineral content (up to 1 gram per liter) and which have a more constant water supply because they are refilled periodically with water from the connecting streams during floods. However, even these lakes cannot serve as sources of supply for large centers of population, in which case it is necessary to plan for utilization of the waters located under the permafrost layer. The formation of thermokarst lakes in the Central Yakutia region has not been studied sufficiently. On the other hand, such lakes are the only available sources of water supply in a number of regions. It is essential to expand the investigation of the regime of these lakes in the immediate future.

ABSTRACT 9

Efimov, A. I., THE REGIME OF SUPRAPERMAFROST WATERS WITHIN THE BOUNDARIES OF WARM STRUCTURES (EASTERN TRANSBAIKAL).

The regime of the semifrozen suprapermafrost layer carrying water, adjusted to the gravel-conglomerate stratum of the quaternary deposits and occurring at a depth of 2 to 5 m, is described on the basis of materials collected during an expedition. The water-carrying layer is underlaid by permanently frozen, massive Jurassic clays. In the winter it acquires pressure due to the deep freezing reaching 4 to 5 m in the region around Chita. Within the boundaries of structures with large emanation of heat, where there is no seasonal freezing, the level of the pressurized water-carrying suprapermafrost layer rises 1 to 1.5 m in the winter. The rise in level inside structures is quite characteristic for Eastern Transbaikal and lasts from January until May. The unfrozen ground under the structure, where the water level rises, extends over the entire boundary of the structure and sometimes even beyond it.

Periodic wetting and drying of the ground under the foundations, due to seasonal variation in the level of the suprapermafrost layer, is one of the factors contributing to the weakening of the bearing capacity of the ground supporting the foundations. This results in deformation of the structures, which is considerable at times.

The problem of taking measures against the rise in the water level within the boundaries of structures is quite complicated, since the regime of the semifreezing permafrost layer varies with the seasons of the year. For this reason, it is desirable to avoid the use of areas developing a semifreezing suprapermafrost layer for erection of large structures giving off heat.

ABSTRACT 10

Efimov, A. I., Kachurin, S. P., and Soloviev, P. A., HYDROGEOLOGICAL AND PERMAFROST SURVEY OF THE VICINITY OF THE CHURAPCHA SETTLEMENT (YaASSR)

Permafrost investigations were conducted in the region between the rivers Lena and Amginsk for the main purpose of locating sources of water supply for the regions suffering from lack of water as a result of the drying of the thermokarst lakes. Plane tables were used in the aerial surveying of the permafrost and hydrogeological conditions. This survey speeded up the work, made it more accurate, and also made it possible to draw a series of permafrost and geomorphological maps of the region to a scale of 1:60,000.

Four morphological types of topographic surfaces were determined; they have the following specific features:

1. Areas between depressions (5 per cent of the area surveyed).
2. Depressions of 5 to 18 m in depth (33.5 per cent of the area surveyed).
3. Thermokarst lakes (1.4 per cent of the area surveyed).
4. Erosional lakes (0.1 per cent of the area surveyed).

The following section (from top to bottom) is the result of a detailed analysis of the upper part of the quaternary stratum of thickness 50 to 70 m.

a. A cover of loam and sandy loam, 2 to 5 m thick.

b. Ground ice of firn (glacier snow) origin, without horizontal divisions and containing a considerable quantity of mud particles distributed in the ice in the form of obliquely inclined thin layers. The ice is absent in the depressions formed as a result of its melting, and frequently in the areas between the depressions also. This ice is 15 to 20 m thick.

c. A layer of loam 5 to 6 m thick, replacing the ground ice and occurring in the ice.

d. Interstratification of the sandy part of the loam with the remains of shells. This layer is 13 to 20 m thick.

e. A layer of sand more than 6 m thick, in some places changing into gravel covering Jurassic sandstones.

It was established that formation of ground ice depends on its hypsometric state.

Several types of contour were observed; they had different permafrost conditions and dissimilar courses of the freezing process. These types are as follows:

1. Heights between depressions, where the frozen layer is shallow over large areas, is stable or growing, and is the oldest.

2. Beds of depressions and river valleys, where temperature conditions vary, depending on the age of the beds, but intensive growth of the frozen layer occurs throughout. Such growth does not occur in grounds having higher temperatures, preserved from the thawing of the lakes.

3. River beds and lake bottoms, with layers of thawed ground more than 10 to 20 m thick occurring directly at the surface or under a thin layer of soil that had time to freeze.

4. Hollows of newly forming lakes, in which thawing of the frozen layer occurs.

Degradation of permafrost in the Churapcha region is a local phenomenon related to the presence of thermokarst reservoirs and manifested over an area occupying only 3 per cent of the territory involved. The well-developed thermokarst lakes in this region can be classified in five groups differing with respect to origin and importance as water supplies. It was found that most of these lakes are rapidly drying out.

Suprapermafrost waters are absent. It is indicated that the ground ice can be utilized by creating artificial thermokarst lakes. The subpermafrost waters, exploration of which is being planned, must constitute the main source of water supply for the populated points between the rivers Lena and Amginsk.

A B S T R A C T 11

Zhukov, V. F., SPECIAL ASPECTS OF CONSTRUCTION OF FOUNDATIONS FOR POWER PLANTS IN THE REGION OF PERMAFROST

This report on soil and foundation structures required for erection of power plants in the region of permafrost refers to a series of phenomena, the neglect of which tends to cause disturbance of the state of permafrost in the region of the substructures, resulting in disturbance of the stability of the structures and their deformation. The main factors affecting the state of the permafrost layer are: the temperature regime of the superstructure, the methods of construction, and the manner in which the structures are used. Power plants give off a large quantity of heat; therefore, plans for their erection in the region of permafrost are evolved under consideration of several special conditions.

The following particular aspects must necessarily be taken into consideration during the work for constructing the foundations:

a. The necessity of strict adherence during construction operations to the construction principle incorporated in the design and to its execution in the course of the work.

b. Consideration of the permafrost factors characteristic for each area under construction.

c. The necessity of maintaining the time schedule adopted for the organization of the work, and the need for readjusting the construction procedures in case of deviation from the initial schedule.

If the work schedule is changed, it may occur that the methods stipulated in the organizational project of the work are no longer permissible. In the case of construction based on the principle of preserving the permafrost, for instance, it is permissible in the winter to prepare all the foundation pits, pour the concrete for the foundations in place, and freeze the ground. In the summer, however, it is necessary to build the concrete foundations above ground (in a prefabricated manner) and to excavate the pits only before the prepared foundations are to be lowered into the pits. After the foundations have been erected, the pits should immediately be well filled with soil. Freezing of the soil under the foundations can be carried out only when the next winter season arrives. Construction principles other than usual and presence of permafrost compel the engineers to change the ordinary methods applied to earthworks and to use steam and electricity to thaw out the solidly frozen ground.

The method of loosening the frozen ground with a pickax can be improved also. For this purpose, it is necessary to pay attention to the shape of the sharp point of the pick and to the manner in which the blow is directed.

The sharp edge of the pick should have a groove on the outer side, and the blows of the pick on the ground should be directed along the ice seams in the soil. The ground splits most readily along these seams, and the required expenditure of muscular energy is slight.

In the text of the article the analysis of the special aspects of foundation construction is supplemented by examples of construction of several power plants erected in the region of permafrost.

ABSTRACT 12

Kachurin, S. P., DIRECTIONS FOR INVESTIGATION OF PERMAFROST UNDER CONSIDERATION OF THE SURFACE CONTOUR OF THE EARTH

In this paper brief directions are given regarding permafrost and geomorphological investigations in the regions of permafrost. The need for such directions is determined by the fact that permafrost investigations differ considerably from purely geological, soil, geographical, and other investigations. They require much more labor and time for drilling deep bores and holes and for obtaining geothermal and physico-technical materials, together with geological, hydrogeological, and other data. For this reason, the instructive directions allow for maximal utilization of every test hole. The availability of the directions should be of assistance in the following:

a. Obtaining data that are as much as possible similar in context and quite comparable for various geographical regions.

b. Studying various forms of contours and processes in a more complete way and in accordance with the same procedure.

c. Establishing and evaluating the effect of the contour on permafrost and, in contrast, the dependence of the contour shape on the occurrence of permafrost.

d. Obtaining data for theoretical analyses as well as for satisfying the requirements of applied considerations.

The article presents and discusses the following aspects: the characteristics of the basic forms of macrocontour, mesocontour, and microcontour; directions regarding methods and subjects for investigation; the scope and types of physico-mechanical analysis of soils as well as the objective and scope of laboratory analyses, experimental works, and the method for preparing reports.

The directions given were worked out under consideration of the existing instructions and questionnaires on "Analysis of a Permafrost Section," prepared by the Institute of Frost Science.

ABSTRACT 13

Kachurin, S. P., TYPES OF PERMAFROST IN THE TRANSBAIKAL REGION

This paper summarizes some results of studies of various types of permafrost in the eastern Transbaikal region. The work is based on the author's personal investigations (1941-43) as well as bibliographical sources. In view of its various types of permafrost and their correlation with the contour conditions of the locale, the Transbaikal constitutes a classical region. Attention was called to this fact by earlier investigators (V. A. Obruchev, I. Krashennikov, A. V. Lvov) with reference to particular regions.

The reasons for such well-defined interdependence between permafrost and contour are the following special geographical conditions of the Transbaikal region:

1. Continental climate.
2. Well-defined contours of mountains, valleys, and plains.
3. The bounding location of Transbaikal with respect to the entire region of permafrost in USSR.
4. The fact that this area of the region of permafrost in USSR has been investigated a little more thoroughly than other areas, although these investigations were mainly secondary studies made in the course of other investigations for applied purposes.

Several characteristic types of permafrost, differing from each other in a number of symptoms, occur on the vast territory of this region extending more than a thousand kilometers from west to east as well as from north to south.

The following are the main morphological types of permafrost:

1. Permafrost continuous in area and monolithic in depth.
2. Permafrost with separate islands of unfrozen ground.
3. Islands of permafrost among unfrozen grounds.
4. Occasional areas containing ice lenses (sporadic icing).

The scheme indicated is rather a preliminary one; all the basic types of permafrost are interconnected, with mutual transitions depending on local conditions. Definite construction principles correspond to each of the given types of permafrost. The principle of destroying the permafrost in the foundations of structures has the most general and wide application.

ABSTRACT 14

Kachurin, S. P., PHYSICO-GEOGRAPHICAL SURVEY OF SOUTHEASTERN TRANSBAIKAL AND THE LEFT BANK OF THE UPPER AND MIDDLE REACHES OF THE AMUR RIVER

The survey represents a short introductory part to a series of articles prepared for the military engineering committee; it contains a special description of the permafrost and engineering conditions.

The territory investigated is a relatively narrow strip in the southern part of eastern Transbaikal and the left bank of the Amur River. It is divided by the author (conditionally) into several parts having more or less similar geographical aspects. The basic objective of the survey is to give a general picture of the orography, geology, hydrography, vegetation, and other elements of the landscape, typical for the regions investigated. The basic materials for the survey were publications, mainly those of geological and geographical content, as well as some personal observations of the author. The paper includes a list of the references used.

ABSTRACT 15

Koloskov, P. I., SURVEY OF SEASONALLY FROZEN GROUNDS IN THE TERRITORY OF USSR FOR ZONING PURPOSES, ACCORDING TO AVAILABLE DATA

The plan for the study of seasonally frozen grounds, initiated in 1944 by the Institute of Frost Science, has become well defined. It was found technically feasible to divide the work into two parts, general and regional. The work of the general survey comprises the following items:

a. Basic aspects and perspectives of the study of seasonally frozen ground.

b. Procedure for obtaining and evaluating the characteristics of seasonally frozen ground from indirect data on climate and other phenomena, when data from the following direct observations are unavailable or insufficient.

1. Mean annual temperature of the ground.
2. Mean annual amplitudes of the ground temperature.
3. Climatic indexes of the moisture of the ground.
4. Lowest monthly average of the ground temperature.
5. Highest monthly average of the ground temperature.
6. Depth of freezing of the ground under natural conditions.
7. Depth of freezing of the ground under conditions of bare surface.

c. Standardization of the procedure and technique involved in preparing regional maps connecting adjacent regions.

d. Directions for further methodological operations.

Some of these studies were submitted for publication.

ABSTRACT 16

Koloskov, P. I., **MAP OF DEPTHS OF WINTER FREEZING OF THE SOIL IN EUROPEAN USSR AND KAZAKHSTAN UNDER CONDITIONS OF BARE SURFACE**

One of the problems of the Institute of Frost Science, Department for the Study of Seasonally Frozen Grounds, has been the preparation of a map of the depths of winter freezing under conditions of bare surface. It became necessary to expedite the preparation of the map because the Narkomstroi needed the information for evaluating data on the depth of foundations.

Lack of data from direct observations of the depths of freezing of the soil under the conditions in question made it necessary to obtain such data from the formula

$$h = k\sqrt{\frac{\Sigma(-\,t^{o})}{\sqrt{tw}}}\ *$$

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*The formula given in the original text reads

$$h = k\sqrt{\frac{\Sigma(-\,t^{o})}{\sqrt{t + w}}}$$

which is dimensionally incorrect; therefore, it is given here in a corrected form corresponding to that of the author's formula appearing in the next abstract.

in which $\underline{h}$ is the depth of freezing in centimeters under conditions of bare surface, $\Sigma(-t^{o})$ is the yearly sum of the monthly averages of the negative temperatures of the air, $\underline{t}$ is the mean annual temperature of the air, $\underline{w}$ is the climatological index of humidity, and $\underline{k}$ is a coefficient determined from observations.

The magnitude of the coefficient $\underline{k}$ was determined from data of five meteorological stations which made observations of the temperature of the soil under conditions of bare surface. The value of this coefficient varied from 61.5 (Poltava) to 77.6 (Leningrad), the mean value being 71.5. A map drawn to a scale of 1:5,000,000 was prepared from the data computed from the formula given above, using the approximated value of 70.0 as the magnitude of the coefficient.

The depths of freezing in the European part increase to a greater extent from south to north. In the territory of Kazakhstan the increase in depth of freezing from south to north occurs more abruptly.

ABSTRACT 17

Koloskov, P. I., DEPTH OF WINTER FREEZING OF THE SOIL UNDER CONDITIONS OF NATURAL AND BARE SURFACES

No direct data were available regarding the depth of freezing under conditions of either natural surface or bare surface. It was necessary, therefore, to obtain approximate characteristics by means of computations.

The depth of freezing under natural conditions was computed in accordance with the formula

$$h = k\frac{\alpha 2t}{\sqrt{tw}} + 40$$

in which $\underline{h}$ is the depth of freezing in centimeters, $\underline{t}$ is the mean annual temperature of the soil, α is the mean annual amplitude of the temperature of the soil at a depth of 40 cm, $\underline{w}$ is a climatological index of moisture, and $\underline{k}$ is a coefficient determined from observations. Data about the temperature of the soil, collected by more than 200 meteorological stations, have been used to determine $\underline{k}$. The average value of the coefficient $\underline{k}$ was found to be 65.0.

The depth of freezing under conditions of bare surface was calculated in accordance with the formula

$$h = k\sqrt{\frac{\Sigma(-t^{o})}{\sqrt{tw}}}$$

in which $\Sigma(-t^{o})$ is the yearly sum of the monthly averages of the negative temperatures of the air, t is the mean annual temperature of the soil, and the meaning of the magnitudes h, w, and k is the same as in the preceding formula. The value of the coefficient k was determined from data of five stations and was equal to 70.0. Two maps drawn to a scale of 1:5,000,000 have been prepared on the basis of the values obtained in this manner. A third map, combining these two maps, was prepared for the purpose of indicating regions.

The territory is zoned in accordance with the depth of freezing under conditions of bare surface; the zones are divided into regions in accordance with the depth of freezing under natural conditions.

ABSTRACT 18

Kostetzkaya, T. P., ROENTGENOLOGY OF ICE

The article relates the procedure for evaluating the experimental data obtained from investigations of the structure of ground-ice of Yakutia and of artificial ice (for the purpose of verifying the correctness of these data with those well established in literature).

In the analyses, the ice is regarded as one of the links of the following structural pattern: Co α (hexagon) - ZnS (wurtzite) - ice, patterned in accordance with the densest hexagonal arrangement of atoms. This pattern is compared with the structural arrangement Co ρ (cubic) - ZnS (sphalerite) - diamond, based on a dense cubical arrangement of atoms.

Proceeding from the assumption that the oxygen lattice of the ice is basically a wurtzite one, preliminary theoretical analyses were carried out on the intensive interference of the structures of cobalt, wurtzite, and ice.

ABSTRACT 19

Krasnov, K. M., PERMAFROST IN THE BASIN OF THE ARKAGALA RIVER (HEADWATERS OF THE KOLYMA RIVER)

The data used in this article were collected by the author during expeditionary explorations of various kinds. These data characterize primarily the depth at which the upper boundary of the permafrost occurs. This depth varies widely, depending on the composition of the active layer. The temperature distribution at various depths of a test section, within the range from 0.4 to 6.0 m, is given in accordance with observations covering a yearly cycle.

According to data obtained from test bores, the depth of the lower boundary of the permafrost reaches 162 m. In some cases this depth apparently is much greater, since the temperature in a bore prepared for geothermal tests still was - 3.0° C at a depth of 125 m.

ABSTRACT 20

Krilov, M. M., FROZEN-EARTH DAM WITH WINTER REFRIGERATION AND BRINE (EUTECTIC) COOLING

This paper presents a project for a dam designed to create a water reservoir for the town of Nordvik of the Glavsevpromput.

Construction of ordinary earth dams is not feasible under conditions of permafrost having large ice content because of thawing and deformation of the foundation, resulting in unallowable seepage through the dam, the foundation, and the abutments. The problem of constructing dams in regions of extreme north and permafrost cannot be solved satisfactorily by use of wooden, concrete, and reinforced concrete structures because of severe climatic conditions. It is advantageous to utilize the strength and impermeability of frozen ground for the construction of dams. In this case, the long, severe winter becomes a favorable factor for both construction and subsequent operation of the refrigerated dam. In order to make the frozen-earth dam durable for many years, it is necessary throughout the year to maintain a negative temperature in the definite contour of the core and base of the dam as well as in its shore parts. This temperature must be at least 1.5° to 2° C lower than the thawing temperature of the ground composing the body and base of the dam. The hydrotechnical problem of the stability of the frozen-earth dam can be solved by using a refrigeration technique utilizing winter cold.

The paper proposes an original dam with a ventilation gallery into which are placed low-temperature ice or eutectic mixtures (brines) freezing or melting at a well-defined constant temperature; for example, the eutectic of table salt with a freezing point of -21.5° C. The presence of a ventilation gallery makes it possible: (a) to build a frozen-earth dam during the winter, cooled to about the average temperature of the month of March; (b) to carry out the charging with low-temperature ice and to achieve control over the state of the core and base of the dam; (c) to divert the water and to accomplish local freezing of the seepage area in case of partial thawing and penetration of the water through the frozen part of the dam.

Experimental construction and observation of frozen-earth dams will make it possible to solve important problems of constructing frozen-earth dams in the more southern regions of permafrost. It is contemplated to use air cooling of the dam with winter cold by means of artificial ventilation, and to introduce low-temperature ice charges and mechanical air cooling by means of air coolers and cooling wells, without using bores and brine cooling. The feasibility of constructing frozen-earth dams is confirmed by successful construction of ice storehouses which have an annual heat circulation that is less favorable than that of a dam having no internal inflow of heat.

A B S T R A C T 21

Krilov, M. M., EXPERIMENTAL CONSTRUCTION AND OPERATION OF ICE STOREHOUSES

Large construction of ice storehouses in the Ukraine was started in 1945. Of course, the farther south, the more complicated becomes the problem. The typical methods of construction and operation of ice storehouses, suitable for the interim region, have to be revised for the southern region. The first, most substantial recommendation prescribes building the storehouses deeper in the ground. The second suggestion concerns the organization of freezing the ice in the nighttime, since in the daytime this procedure is ineffective in the south. The third suggestion refers to the speedup in constructing the movable sheathing. This factor makes it possible to speed up the work on preparation of the ice-and-snow cover and to decrease the required amount of structural material. The fourth suggestion concerns the strengthening of the brine cooling with timely preparation of the saline ice field, a procedure which would tend to lessen the thawing of the ice storehouse at the bottom, thus decreasing its settling. The fifth suggestion points out the need for protecting the insulating layer of cinders by adding a layer of soil and sawdust, or peat, and by sodding or seeding the surface with grass. The sixth suggestion concerns the additional protection at the top, near the ice part of the entrance, by means of a shield of a shutter-like cover filled periodically with ice. The seventh suggestion indicates the necessity for maintaining a temperature of -1.5° to -2.0° C in the ice storehouses in the south during the summer, in contrast to the temperatures of -0.5° C suitable for the interim region. The lower temperature is essential for the preservation of the storehouses and for storing valuable goods at a lower humidity of the air.

In small, single-chamber storehouses of 16- to 20- ton capacity used in the Ukraine, goods are stored in separate box-like containers. In this way goods are isolated from each other under conditions of temporary storage at increased brine cooling.

ABSTRACT 22

Koodriavtzev, V. A., PERMAFROST INVESTIGATION AT THE BESTIAKH-PAVLOVSK COVE (IaASSR)

Permafrost investigations of the right bank of the Lena River near Yakutsk have established the general character of freezing near this large river. It was found that the body of permafrost on the peninsula constitutes a frozen peninsula admidst unfrozen ground, separated from the permafrost body of the bank by a gap of unfrozen ground under the old layer. The frozen ground on the peninsula is a continuation of the body of the bank and tends to grow.

ABSTRACT 23

Melnikov, P. I. and Soloviev, P. A., PERMAFROST CONDITIONS AND PRINCIPLES OF CONSTRUCTION OF THE ZHATAISK COVE

A detailed picture of the permafrost conditions of the youngest river terraces, where interaction occurs between permafrost and river water, has been obtained from investigations made in the region of central Yakutia. The development of frozen ground is closely related to the age of the terraces. The proximity of water reservoirs produces mainly local effects. The frozen ground is currently still developing, extending to the terraces as they are formed. The present occurrence of frozen ground and the progress of the process given above are complicated by the counteracting influence of the river and lake water, which occasionally causes local degradation of the permafrost.

A B S T R A C T 24

Mishkovskaya, G. A., MORPHOLOGICAL TYPES OF SEASONALLY FROZEN GROUND IN KAZAKHSTAN

The abstracted article was prepared under the supervision of P. I. Koloskov in accordance with his method (P. I. Koloskov, "Survey of Seasonally Frozen Grounds in the Territory of USSR for Zoning Purposes, According to Available Data"), and constitutes a continuation of his work of 1944. All of the 1:5,000,000 maps attached to the article were prepared by the author, including the maps (originally drawn by Koloskov in 1944) that were revised because it became possible to utilize new supplementary data about the temperature of the soil. The paper consists of six maps and the corresponding text. These maps verify the established relationships and illustrate the suggested relationships given by Koloskov in his article of 1944.

The map of the depths of winter freezing of the soil under natural surface conditions shows that the maximum depth of freezing (more than 250 cm) of the soil occurs in eastern Kazakhstan (the region of western Preebalkhash, Lake Zaisan, and Black Irtish). From this region, the depth of freezing decreases towards the west, north, and south. The zero isoline of depth of freezing passes through the eastern shore of the Caspian Sea (the Mangishlak Peninsula) and through southern Kazakhstan (headwaters of the Sir-Daria River).

The map of distribution of the depths of winter freezing of the soil under conditions of bare surface is more simple, since in this case the dependence of the depth upon the snow cover is eliminated. The general decrease in freezing of the soil proceeds from north (more than 350 cm) to south (less than 100 cm), with small local variations.

The map showing the difference in depth of freezing of the soil under conditions of natural surface and bare surface made it possible to establish the following facts: in plains (including the rolling Kazakh country), the depth of freezing of the soil decreases with increasing thickness of the snow cover; in mountainous regions, in contrast, as the absolute heights increase and the thickness of the snow cover grows, the depth of freezing under natural conditions approaches that existing under conditions of bare surface.

Zoning of the Kazakhstan Territory is carried out in accordance with the basic morphological index of seasonally frozen ground, that is, the depth of freezing. The zones are obtained by dividing the territory according to the depth of freezing of the soil under conditions of bare surface, using 50-cm intervals. In this way, seven zones were obtained within the range of depth of freezing from less than 100 cm to more than 350 cm. The zones are subdivided into regions according to the depth of freezing of the soil under natural conditions, using 50-cm intervals. Accordingly, the nature of the snow cover determines the 58 regions obtained. The regions corresponding to each zone are described in detail. The characteristics of the basic soil and climate factors influencing the development of frozen ground are given.

ABSTRACT 25

Nersesova, Z. A., Shimanovski, S. V., and Korotkova, O. N., PHASE COMPOSITION OF FROZEN GROUNDS

The dilatometric method for determining nonfreezing water in the ground has been checked by means of special tests. It was found that this method does not give sufficient accuracy and reproduction of results, and cannot be used, therefore, for studies of the phase composition of frozen grounds. Fedosov's dilatometer may be used as a device for determining the specific weight of frozen soils.

A number of methods are available for determining the nonfreezing water in disperse systems. The method most suitable for the study of phase composition of frozen grounds is the calorimetric method which measures the quantity of heat absorbed during melting of the ice in the ground; the melted ice corresponds to the amount of free (freezing) water contained in the ground. In comparison with the dilatometric method, the calorimetric method has many advantages with regard to the original theoretical premises and the methods of procedure.

The calorimetric method for determining the quantity of nonfreezing water in soils has been worked out in the laboratory. A calorimeter was constructed permitting the use of large specimens of soils and giving sufficient precision of the temperature measurements. The results of the first series of tests on the calorimetric determination of nonfreezing water allow for a positive evaluation of this method.

A B S T R A C T 26

Paramonova, L. G., MORPHOLOGICAL TYPES OF SEASONALLY FROZEN GROUNDS IN EUROPEAN USSR

This work consisted chiefly of preparing a series of maps of European USSR to a scale of 1:5,000,000 under the supervision of P. I. Koloskov and in accordance with the method evolved by him. A number of maps, prepared in 1944 by P. I. Koloskov to a scale of 1:12,600,000, were redrawn to a larger scale.

The following eight maps were prepared:

1. Map of mean annual temperatures of the soil.

2. Map of mean annual amplitudes of the temperature of the soil at a depth of 0.4 m.

3. Map of minimal mean monthly temperatures of the soil at a depth of 0.4 m.

4. Map of maximal mean monthly temperatures of the soil at a depth of 0.4 m.

5. Map of depth of freezing of the soil under conditions of natural surface.

6. Map of depth of freezing of the soil under conditions of bare surface.

7. Map showing the difference in depth of freezing of the soil under conditions of natural surface and bare surface.

9. Map of the regions of seasonally frozen ground, obtained from maps 5 and 6.

A report on the work accomplished has been prepared. It contains a brief description of the maps and a preliminary analysis of the map showing the regions of seasonally frozen ground. The work is to be continued in 1946.

A B S T R A C T 27

Popov, A. I. and Dostovalov, V. N., THE OB EXPEDITION FOR PERMAFROST SURVEY

The Ob expedition was organized for the purpose of determining the basic laws, the extents, and the physical state of permafrost in the central and western parts of the western Siberian plain.

The practical importance and timeliness of such a work were determined by surveys and plans for a new railroad through the north of the western Siberian plain. The right tributary of the Ob River, the Nazim River, flowing into the Ob at a point somewhat above the mouth of the Irtish, was chosen as the object for the 1945 investigation.

The new method of field investigations of permafrost, applied in the Ob expedition, was that of electrical sounding of the frozen strata by means of direct current. This method yielded highly positive results and facilitated rapid determination of permafrost conditions. The results of the field investigations and the preliminary examination of the samples have established the basic laws of permafrost in the regions investigated, in relation to various combinations of existing physico-geographical conditions (snow cover, exposure of slopes, vegetation cover, lithology, and topography). These results also determined the major indexes for the physical state of the permafrost. In the region investigated, permafrost was found only in clays on slopes of divides facing north. No permafrost was found in peat bogs situated on flat terraces above the river terraces.

The results of the Nazim investigations show that the southern boundary of permafrost in this locale occurs farther south than existing maps indicate.

ABSTRACT 28

Sedletzki, I. D. and Kostetzkaya, T. P., REVIEW OF LITERATURE ON THE STRUCTURE OF ICE

The paper is mainly a compilation of experimental data obtained by a number of authors from detailed roentgenographic studies of ice. The period covered is 1917 (Rinke) to 1935 (Burton and Oliver). Included in the paper is a comparative table of values of the distance between planes (d_{hkl}), obtained by John (1918), Dennison (1921), and Oliver (1935).

The method of preparing test samples is discussed, and it is noted that all the constants and results concerning the fact that ice has an hexagonal structure with compact packing were made on the basis of experiments on artificial ice. Roentgenographic studies of the structures of natural ice and ground ice, which are of particular interest to frost scientists, have not been made as yet; meanwhile, the variety of crystallographical and morphological data on the structure of ice indicates the existence of several modifications of ice.

Interference maxima for roentgenograms of water were determined (Meyer and Stuart), parallel with the studies which established the basic distances between planes of the ice lattice. A comparison of the results shows that similarity exists between the roentgenograms of ice and water; this fact indicates that the water retains the basic structure of the ice.

The bibliography on the structure of ice and water lists only the fundamental references used in this article.

ABSTRACT 29

Sidorov, M. F., OBSERVATIONS OF TEMPERATURE REGIME AND OPERATING CONDITIONS OF THE ANADYR COAL DEPOSIT

During the period from 1936 to 1942, the author observed the temperature and operating conditions of the coal deposit in the region of the town of Anadyr. This deposit is exploited for local needs by means of drift mining utilizing very crude methods. The coal mined is located in the permafrost layer, the temperature of which is -5.5° C and lower.

The mine is practically never shored, but the frozen mass is sufficiently stable so that the roofs seldom cave in. Mining of the coal is done in the winter months when the walls of the tunnels and drifts get ventilation colling. In the summer the entrance to the mine is closed, and the mine is used for cold storage of food products. The temperature of the air in the mines is determined primarily by the temperature of their walls. Cold flows into the mines from the colder surrounding masses of permanently frozen rocks.

ABSTRACT 30

Toomel, V. F., MAP OF PERMAFROST IN USSR

Much new factual material on permafrost in USSR has been collected in the last ten years. This material makes it possible to revise, with greater precision, the former concepts of the permafrost boundaries and the nature of the permafrost layers. Hence, it became possible to draw a map to a scale of 1:10,000,000; this scale is larger than that used for the maps prepared by M. I. Sumgin and based on the same principles.

In many instances the new data made it possible to define the boundaries of separate permafrost islands with greater precision, to supplement the boundaries of the temperature zones of the permafrost layer, presented in 1935 by M. I. Sumgin, and to establish these boundaries with greater accuracy. In addition, the first attempt has been made to delineate the range of permafrost according to the greatest thicknesses of the permafrost layer in individual regions.

In determining the individual boundaries in territories for which little factual data are available, the author has guided himself by known laws of permafrost of a definite nature and thickness in relation to the absolute elevation of the locale.

A B S T R A C T 31

Toomel, V. F., PERMAFROST CONDITIONS IN MANCHURIA

A schematic map of permafrost in Manchuria has been drawn to a scale of 1:2,500,000 on the basis of limited, often indirect, data taken from literature. The explanatory note on the map gives supplementary information about the conditions of winter freezing and summer thawing of soils in separate permafrost regions defined according to the expected maximal thicknesses of the permafrost layer.

A B S T R A C T 32

Toomel, V. F., TENTATIVE DIRECTIONS FOR PERMAFROST SURVEYS

Heretofore diverse methods were used for studying permafrost distribution in space and properties of permafrost layers. The tentative directions for permafrost survey are intended to correct this inconsistency to some extent.

Developing the aspects discussed in the author's paper "On Permafrost Survey," published in the Reports of the Academy of Sciences, Geographical and Geophysical Series, No. 2, 1945, the author notes in particular the method for determining locations for bearing sections, surfaces and areas, and the means for obtaining the contour of the selected types of areas. The required supplementary observations are specified. The scope of the necessary field operations is indicated in each case. Thus, the directions regulate the collection of field data for permafrost maps. In view of the newness of the case, the directions were presented as tentative. They were tested in 1945 at several field jobs and did not evoke any major objections.

The areas having similar permafrost characteristics and magnitude of permafrost layers were determined on the basis of the concept that, within the limits of certain geomorphological elements of the same age, the active layer and the first few meters of the permafrost layer definitely correspond to the current physico-geographical conditions.

A B S T R A C T 33

Toomel, V. F., Kachurin, S. P. and Popov, A. I., PERMAFROST CONDITIONS IN THE SOVIET AND FOREIGN FAR EAST

Brief descriptions of the permafrost conditions in various physico-geographical regions were prepared upon request of a government department interested in the matter. Within the scope of the studies, the descriptions present an analysis of the permafrost layer, the active layer, and the supra-permafrost layers. In addition, the penetrability of the locale is indicated, and the conditions for construction and operation of transportation facilities are specified.

The regions described are shown on several pages of a map drawn to a scale of 1:1,000,000. On the respective pages, V. F. Toomel plotted the schematic boundaries of permafrost regions in extent of permafrost and thickness of permafrost layers. Various data on permafrost, giving a rather non-uniform illustration of the territory under consideration, have been utilized for plotting these boundaries. Nevertheless, these data make it possible to detect certain relationships between the development of seasonally frozen ground and permafrost in the border regions of the territories in which these phenomena occur and the geographical aspects of the locale. The following geographical aspects were utilized: elevation of the locale, relief and soils, and data about climate and vegetation.

A B S T R A C T 34

Tiutiunov, I. A., THE PROBLEM OF STAINS-MEDALLIONS IN THE ANADYR REGION

This article was written on the basis of the author's personal observations in the Anadyr region during 1943-45. It contains a brief survey of some existing theories on the problem of stains-medallions and presents a new explanation for the occurrence of medallions in the Anadyr region.

The formed core of unfrozen ground is regarded as a closed system functioning with transfer of heat to the surrounding medium. However, only a definite part of the heat given up by the system is converted into work of displacing the core in space. The displacement of the core in space is not of volcanic nature but is cyclic.

A detailed analysis of this problem explains a number of occurrences and makes it possible to regulate the process of migration, and the phenomena connected with it, during erection of surface structures.

ABSTRACT 35

Ool, I. F., SEASONALLY FROZEN GROUND IN CENTRAL TRANSBAIKAL

The climatic properties of Transbaikal determine the conditions for the formation of seasonally frozen ground. These properties are: (a) low winter temperatures; (b) large relative and absolute amplitudes of the temperature variation between January and July, which amounts to 45° to 51° C with respect to the mean monthly temperatures; (c) numerous clear and sunny days in winter, accompanied by relatively little cloudiness; (d) existence of winter calms; (e) strong winds during May, June, September, and October; (f) small quantity of precipitation in general and the very thin snow cover (5 to 15 cm) which forms in the middle of November and often settles on ground that is dry and frozen already. These factors create favorable conditions for deep freezing of the ground in winter, the depth of freezing attaining maximal values for the given latitudes.

Very little published material on this aspect of Transbaikal is available. The explorations of the Transbaikal expedition of the Institute of Frost Science, carried out in 1941-43 in the Chita region near the town of Romanovka (northeast of Chita), at the Doman station, and in other places, furnish much data, especially about the seasonal freezing of the ground during complete biennial cycles of observation. The main results of these investigations are as follows:

1. Systematic observations at a series of points have verified the available scattered data on the general depth of winter freezing of the ground; this depth ranges from 3 to 5 m.

2. Ground (alluvial) waters and their winter regime were found to have a considerable effect on the seasonally frozen ground.

3. It was established that the depth of seasonally frozen ground at the base of buildings and structures is greatly affected by the closeness or remoteness of heating installations (heating main, for example) and by the heat inside the buildings.

4. The negative temperatures of the upper permafrost layers adjacent to the seasonally frozen ground were lowered when the areas were located under cold (unheated) buildings or structures.

ABSTRACT 36

Chekotillo, A. M., CURRENT MEASURES AGAINST ICING IN THE U.S.A. AND CANADA

This paper compiles and classifies the material published in various American and Canadian publications on the problem of measures against icing. It deals mainly with the experimental operation of the Alcan Highway built in 1942-43. Each of the anti-icing measures applied is compared with analogous procedures used in USSR. It is found that the American researchers (Leffingwell, Taber, Eager, and Pryor) have worked out only the general aspects of the theory of the icing processes. This directly accounts for the fact that passive methods, directed not at the causes of icing but at the liquidation of its effects, are predominant in the practice of anti-icing measures in the U.S.A. and Canada. In general, the American engineers and technicians are proceeding correctly with respect to measures used against icing, but they lag 15 to 20 years behind the USSR engineers in this field. However, in aspects of mechanization, America is more advanced than the USSR.

ABSTRACT 37

Shvetzov, P. F., SUBPERMAFROST WATERS AND THE GIGANTIC ICINGS OF NORTHEASTERN YAKUTIA

The article establishes the existence of extensive gigantic icings on the large territory of the Yan-Kolyma mountainous region and notes that this natural and historical phenomenon does not occur on plateaus and among older mountain systems. The ice fields of these icings attain an area of 150 sq km and a volume of 500,000,000 cu m.

Abstract 37, Shvetzov

On the basis of studies of the gigantic icings of the Taskhai-takh range and observations of the glaciers of the Verkhoyansk Range, the author attributes the origin of these icings to springs of ground water occurring near the icings. These springs have a large and constant discharge (nearly 1500 liters per sec throughout the year), a constant temperature, a low mineral content, and a large quantity of gas. Similar nonfreezing springs of ground water occur near large icings of other mountainous regions (the Momsk Range, the basin of the Omolan River, the headwaters of the Kolyma River). The temperature of the water of these springs is usually 0.4° to 0.5° C. throughout the year.

Taking into consideration the constant temperature and discharge of the springs, the complete freezing or absence of water in most small and medium rivers in the winter, the poorness of the alluvial water tables under river beds, and, moreover, the hypsometric aspects of the mouths of some of the springs (1000 to 1200 m above sea level on the upper parts of the mountain slopes), the author classifies these as springs of subpermafrost waters. The subpermafrost water tables are adapted to sandstones, shales, and effusive Mesozoic rocks which are greatly disturbed and fissured owing to the intensive occurrence of the Austrian orogenic phase. The existence of deep tectonic flaws and fractures in the rocks accounts for the fact that a permafrost layer 150 to 250 m thick becomes permeable, at least to growing and continuously active large springs.

It is noted that subpermafrost waters constitute the only source of water supply for the population and enterprises of many regions of northeastern Yakutia. This fact was confirmed by test drilling for subpermafrost waters at the Kolyma headwaters. The large factories and towns of this region have already obtained the required amount of subpermafrost water from discharging apertures.

It is the author's opinion that the gigantic icings indicate the existence of subpermafrost waters and special kinds of water reservoirs, and that the outlets of the springs of subpermafrost waters are indicative of the particular geological structure of recently formed regions.

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Academy of Science USSR

ICINGS AND COUNTERMEASURES

by

A. M. Chekotillo

Department of Highways, NKVD USSR

Moscow 1940

LIST OF ILLUSTRATIONS

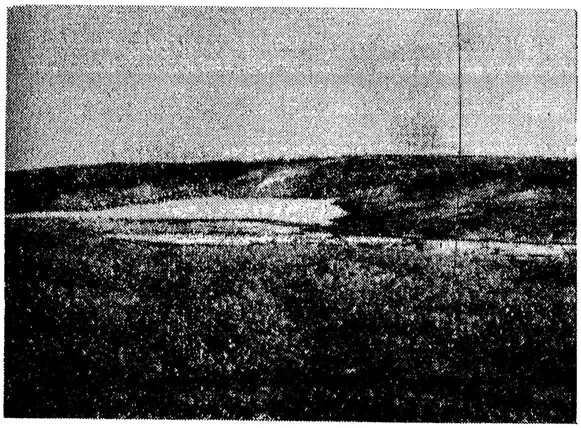


Fig. 1 – Remainder of Icing (Taryn) on the Yamna River, Taimyr Peninsula, Siberia

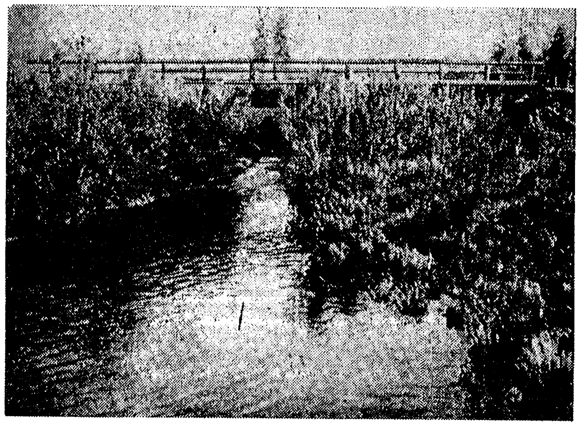


Fig. 2 – Berkakit River in Siberia on which Icings are not Formed

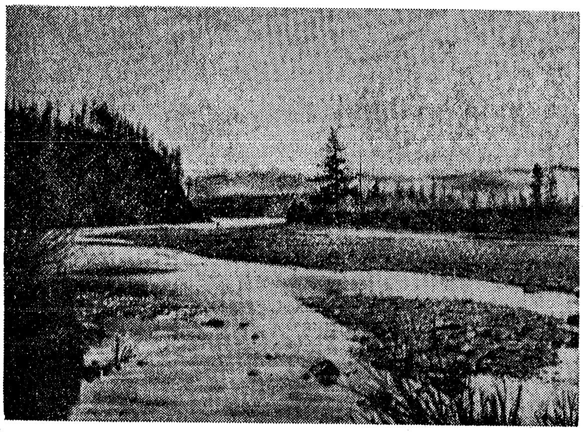


Fig. 3 – Icing on Shallow River with Gravel Bed

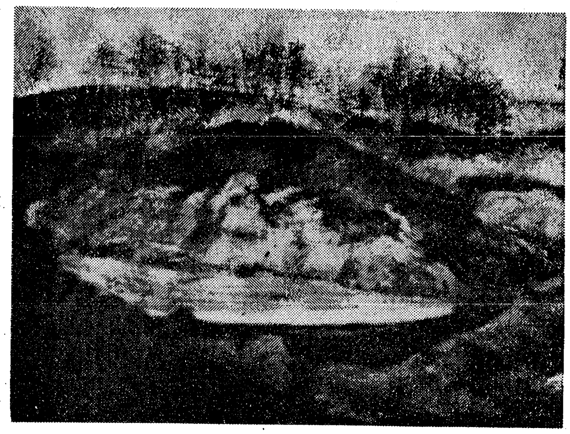


Fig. 4 – Seasonal Icing Mound, Taimyr Peninsula, Siberia

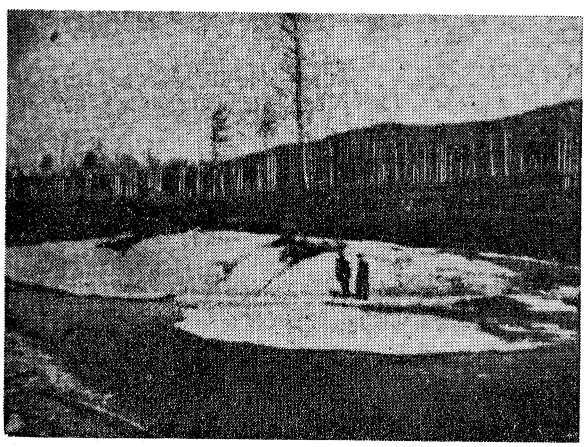


Fig. 5 – Front View of Icing Mound on the Amazar River in Siberia

Fig. 6 – Side View of Icing Mound on the Amazar River in Siberia

CHAPTER I

ICING PROCESSES AND PHENOMENA

A. General Data

The term "icing" refers to a mass of ice formed during the winter by successive freezing of sheets of water seeping from a river, from the ground, from a spring, or from a combination of such sources. Accordingly, the icings are designated as river icings, ground icings, or mixed icings. They usually form irregular sheets or fields, mounds attaining large dimensions (20 m in height and 50 m or more at the base), or incrustations along slopes. The surface of the icing is usually very uneven and its form depends on hydrological and climatic conditions.

Most icings melt during the summer and reappear the following winter. Some large icings do not melt completely during the summer, which results in remnant ice fields called "taryns" (Fig. 1).

Conditions favorable for the formation of icings are:

1. Presence of ground water in the active layer.

2. Low temperature of the air and only thin snow cover during the early part of the winter.

3. Proximity of the permafrost table to the surface of the ground.

4. Thick snow cover during the latter part of the winter.

B. River Icings

River icing is formed when the freezing of a shallow portion of a stream impedes the flow at the upstream side. This increases the hydrostatic pressure of water above the ice barrier and the water is forced to break through to the surface, where it freezes and gradually forms the icing. The water may emerge from a fissure in the ice or may reach the surface as a seep along the bank of the river. River icings, therefore, may not only cover the stream channel but may also extend over a considerable portion of the flood plain. For the same reason, river icings may form at a considerable distance from the stream.

River icings occasionally attain a thickness of 4 m and cover an area of many square miles. Some streams and rivers freeze through to the bottom, yet icings are not formed (Fig. 2). Shallow rivers with gravel beds, having small branches and tributaries, expose a large surface to freezing action, their beds freeze readily so that the flow stops completely, and icings are formed during the first frost (Fig. 3).

River icings also occur in the form of icing mounds when the river water cannot seep readily through the banks (Figs. 4, 5, and 6). These mounds explode when their internal pressure becomes excessively high. Figure 7 shows types and shapes of icings formed in the permafrost region by springs, rivers, and ground water.

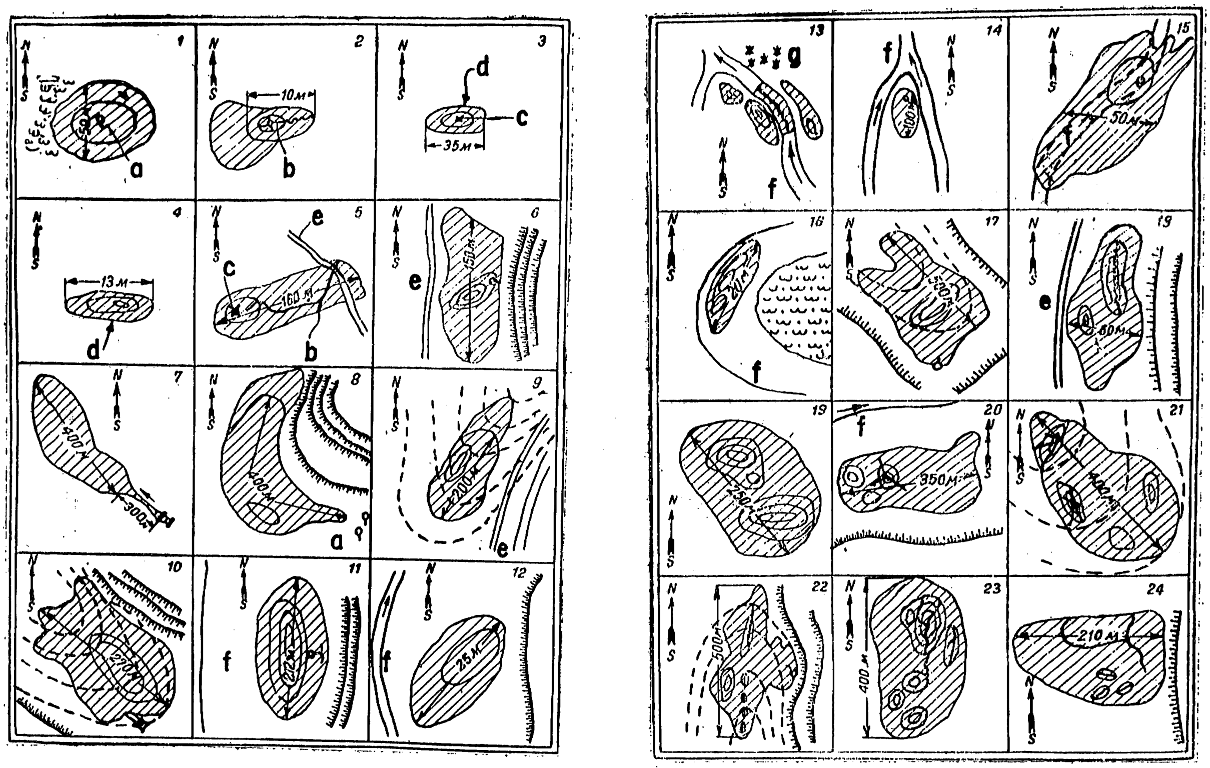
C. Ground Icings

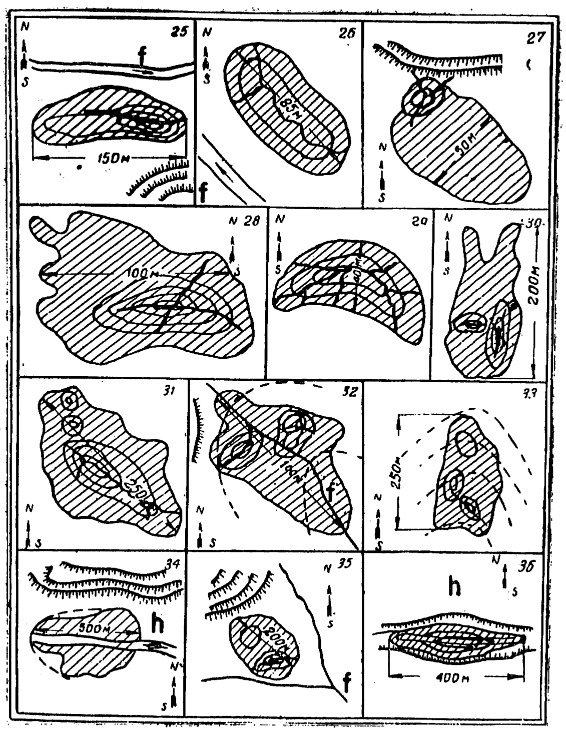
Ground icings are formed when the winter freezing of the ground penetrates to the permafrost. The ground water in the active layer is forced to the surface along a path of least resistance, spilling over the surface and freezing. Similarly, the freezing of water issuing from a spring forms an icing. These icings are more likely to develop along a mountain slope where the strata dip in the same direction as the slope of the mountain. They are less frequent along a slope underlain by strata dipping into the mountain.

Ground icings usually assume the form of mounds and frequently occur at the upstream part of a valley, where the ground flow is constrained and the freezing action is stronger. The pressure produced by the freezing of water in the active layer increases with increasing freezing action until the mound cracks and the unfrozen water flows out, forming ice layers near the mound. Icings are most intense in the southern permafrost region and attain their maximum spread when the snow cover is at its maximum (Fig. 8). Figures 9 and 10 show typical icing mounds. The icings appear in the form of incrustations when the ground water emerges to the surface along a natural slope or an artificial cut.

D. Icings Formed by Springs

Icing mounds form near springs fed by subpermafrost or suprapermafrost waters. These mounds do not have any natural surface layer, but are pure ice. Their size depends on the relative discharge of the springs. The icings attain extensive proportions if the region has thick snow cover and the discharge from the spring is large. Figure 11 shows a typical icing mound formed by a spring. One icing, located in the Momy River valley in Siberia,





a. Spring

b. Fissure

c. Hole

d. Icing Mound

e. Road

f. Stream

g. Mounds

h. Stream Bed

Road acts as frost belt.

Fig. 7—Types and Shapes of Icings Formed by Springs or Rivers

Fig. 8 – Icing Mound in the Southern Permafrost Region

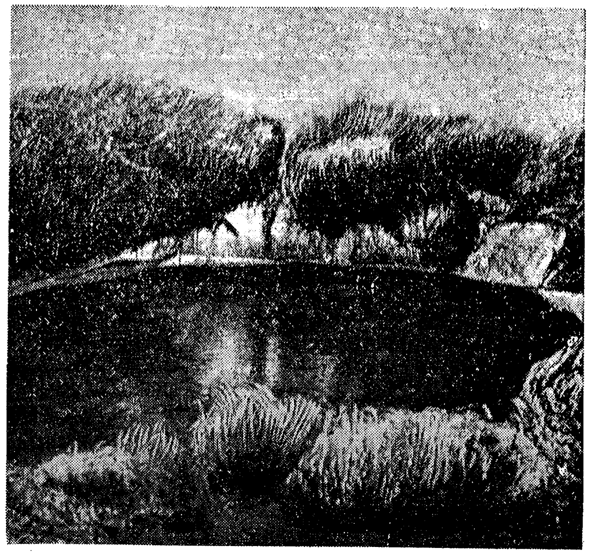
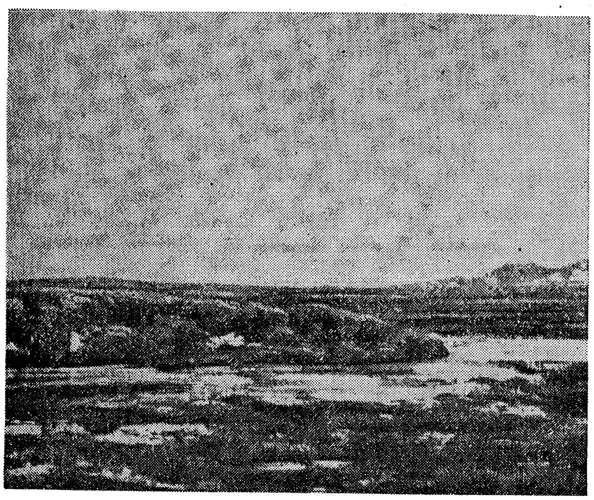


Fig. 9 – Details of an Icing Mound, Taimyr Peninsula, Siberia



The mound is a typical ground icing.

Fig. 10 – Eroded Mound of Seasonal Icing, Taimyr Peninsula, Siberia

The mound is 52 m long, 16 m wide, and 7 m high.

Fig. 11 – Icing Mound Formed by a Spring

is 26 km long, 6 to 8 km wide, about 4 m thick, and covers an area of 48,000 to 54,000 acres. Some icings, fed by subpermafrost water flowing through tectonic flaws, are located at the feet of mountains and form a mass of several million cubic meters.

The formation of icings fed by constant discharge of subpermafrost water depends on intensity of frost, thickness of snow cover, and time of snowfall. The suprapermafrost icings depend also on quantities of precipitation in the summer; their formation is more intensive after a rainy summer.

Figure 12 shows the rate of growth of icings formed by river and ground waters in the permafrost region. Figure 13 gives a typical section through an icing mound.

E. Mixed Icings

Mixed icings are those formed by both ground and river waters. Such icings usually occur near small streams with poorly defined beds. The formation of these icings is affected primarily by the river water because this water is in contact, through the talik, with the ground water contained in any of the layers relative to the permafrost.

F. Theory of Icing

The earliest theory of icing phenomena produced by river waters, evolved by Podyakonov in 1903 and later modified by Sumgin, establishes the following relationship between the variables involved:

$$R = P \frac{c}{d} Q \frac{a}{N + M} \frac{1}{t} \qquad (1)$$

where

- R is the rate of growth of icing,
- P is the frost intensity,
- c is the thermal conductivity of channel bed,
- d is the thickness of snow cover,
- Q is the discharge of stream,
- a is the width of valley,
- N is the effective cross section of the alluviums,
- M is the effective cross section of the stream channel, and
- t is the critical temperature of water with respect to formation of icings.

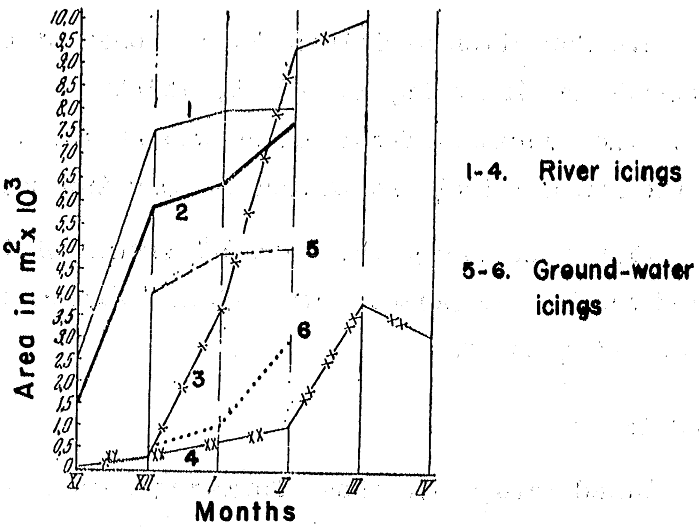
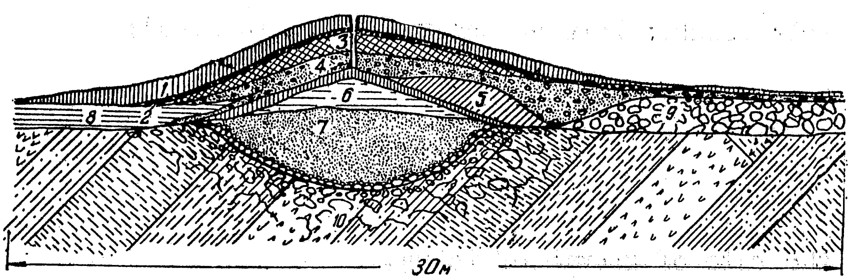


Fig. 12 — Rate of Growth of Icings at Various Points on the Amur-Yakutsk Highway, Winter 1927-28



1. Ice 2. Vegetative cover 3. Sod 4. Loam, sandy loam, pebbles 5. Coal vein 6. Water 7. Red sand 8. Clay and loam 9. Coarse sand 10. Rock

Fig. 13 — Cross Section through an Icing Mound at Strelka on the Amur-Yakutsk Highway

This formula is definitely inadequate because it does not include many of the important factors influencing the icing processes and does not apply to ground icings. Recently M. I. Sumgin developed a new theory based on the study of icing mounds and formations in the permafrost region. The correlated results of this study are presented in Table 1.

Sumgin's theory attributes the icing phenomena to stresses produced in the ground during freezing. According to this theory, pressure in freezing ground is caused either by increase in volume when water is converted into ice or by the force of crystallization of ice. The initial pressure may cause the masses of as yet unfrozen water and ground to move laterally in one or the other direction as well as upward. Thus, the ground that offers least resistance to this pressure will be forced to move in the direction of lesser resistance, producing either a vertical heave or a lateral dislocation.

Another theory, presented by Taber, postulates that the pressure effects which accompany the freezing of ground are due entirely to the force of crystallization of ice. According to Taber, "heaving is upward because that is the direction of heat conduction rather than because it is the direction of least resistance" and "the pressure is developed only in the direction of crystal growth, which is determined chiefly by the direction of heat conduction and the availability of water."

The principles of Sumgin's theory are illustrated in Fig. 14 and can be described as follows. The layers overlying the permafrost of a given area in the permafrost region consist of the water-bearing layer and the active layer. These layers will freeze to some depth below the surface, depending on the intensity and duration of frost action. It is assumed that at a given instant after the beginning of freezing, the active layer in the central part of the area under consideration has frozen to a depth $\underline{H}$, while the region bounding this central part (assumed to be circular with diameter 2R) has frozen to the depth of the permafrost and merged with the permafrost (Fig. 14a). The latter occurrence may be due to the fact that the bounding region has better conductivity than the central part, or it lacks vegetative and snow cover. If freezing continues, the water-bearing layer will begin to freeze and the freezing may reach a depth H', in which case the volume of the frozen part of the water-bearing layer would be $\pi R^2(H' - H)$. Assuming that all the water in the layer of thickness H' - H freezes, the resulting increase in volume is

$$V = \beta \, w q \, \pi R^2 (H' - H) \qquad (2)$$

TABLE 1

CHARACTERISTICS OF ICING AND SWELLING PHENOMENA

Types of Surface Formations Resulting from Stresses in Freezing Ground	Relief of Terrain Where Formations Occur	Nature of Stresses in the Ground	Medium Transmitting the Stresses	Masses Undergoing Displacement	Range of Stress Action
1. Spots-Medallions (polygons). 2. Peat mounds without ice lenses.	Level places (even at high elevations) and flat slopes.	Hydrostatic pressure. Molecular forces acting during expansion of freezing water.	Semi-liquid masses of slud.	Slud (solifluctional ground) and some water.	Shallow layer of slud; small radius of action.
3. Ground icings without mounds. 4. River icings without mounds.	Slopes; often steep slopes. River beds and coves.	Hydrostatic or hydrodynamic pressure, or both.			
5. Peat mounds with ice lenses. 6. Boolgoonyakhs*	Level places (even at high elevations) and flat slopes. Wet, swampy places. As yet unknown. Occasionally form among small lakes.	Hydrostatic pressure. Molecular forces acting during expansion of freezing water.	Water saturating sandy or coarse grounds. Slud may or may not be present.	Water. In special cases masses of slud move with the water.	Deepest water-bearing layers; large radius of action.
7. Ground icings with mounds, ice lenses, and ice fields. 8. River icings with mounds and ice fields.	Slopes or bases of slopes. Beds and coves of rivers and streams.	Hydrostatic or hydrodynamic pressure, or both. Molecular forces acting during expansion of freezing.	Water saturating sandy or coarse grounds. Slud may or may not be present.	Water. In special cases masses of slud move with the water.	Deepest water-bearing layers; large radius of action.

*Frost mounds (pingos, hydrolaccoliths); mounds, usually of considerable size and many years duration.

where β is the expansion coefficient of water converted into ice, w is the percentage moisture (by weight) in the water-bearing ground, and q is the unit weight of the dry material. This increase in volume produces large stresses in the ground, resulting in hydrostatic pressure causing swelling of the ground and movement of water and of some soil material toward the forming mound. The initial dimensions of the mound depend on the magnitude ΔV, while its final dimensions depend on both the hydrostatic pressure and the quantity of water flowing from the surrounding water-bearing layer towards the mound. Growth of the mound ceases when the water-bearing layer has frozen completely.

Occasionally a mound is formed at a point distant from the unfrozen water-bearing layer (Fig. 14b). The water reaching such a point is pumped from the surrounding active layer by hydrostatic pressure. During winter freezing, the water from the water-bearing layer flows to the second terrace and then across the gravel layer to the road; the mound may form on the road or near it.

The relative dimensions of an icing mound can be determined from certain known magnitudes with the aid of the following theoretical considerations. If the mound has the shape of a right cone with radius of base r and height h (Fig. 14c), then the volume of the cone is $V = \pi r^2 h/3$, and it is equal to ΔV. That is,

$$\beta w q \pi R^2 (H' - H) = \frac{\pi r^2 h}{3} \tag{3}$$

If the expansion coefficient is taken as $\beta = 0.09$, then

$$0.27\, w q R^2 (H' - H) = r^2 h \tag{4}$$

which is the equation for the icing mound.

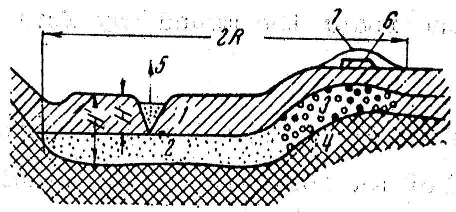
The magnitudes w and q can be readily determined from a sample, while r and h can be measured. Hence the unknowns R and H' - H can be found from the relationship

$$R^2 (H' - H) = \frac{r^2 h}{0.27\, w q} \tag{5}$$

Thus, the magnitude R can be found from measurements of H and H' when the mound begins to form and when it attains its maximal dimensions respectively.

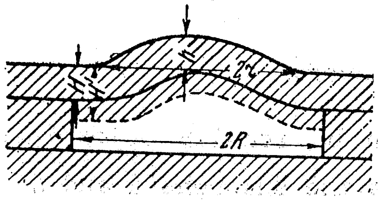
H= depth of freezing at a given instant
H'=final depth of freezing
2R= diameter of unfrozen layer

(a). Volume increase caused by freezing of water in layer of depth H'–H produces pressure resulting in swelling of ground.



1. Impermeable active layer
2. Alluvial layer
3. Dry layer of coarse soil
4. Permafrost
5. Stream
6. Road on second terrace
7. Icing mound formed on the second terrace

(b) Water from the alluvial layer passed through the coarse layer and formed the mound.



h = height of cone
r = radius of base

(c) Section facilitates determination of relative magnitudes shown.

Fig. 14 – Diagrams Illustrating Formation of an Icing Mound

The mound will not form if $H' - H = 0$, that is, if the freezing does not reach the water-bearing layer. Such condition may arise during exceptionally warm winters with normal snow cover or during exceptionally snowy winters with normal temperature.

In Eq. (5), if

$$\frac{r^2 h}{0.27\ wq} = B \tag{6}$$

then

$$R^2\ (H' - H) = B \tag{7}$$

If $H = 0$,

$$R^2\ H' = B \tag{8}$$

This formula shows that the mound begins to form the instant the ground surface freezes. Under natural conditions this situation occurs when the water-bearing layer begins at the surface of the ground or during freezing of pure water.

The value of $\underline{R}$, which is the radius of the water-bearing region feeding the mound, and the value of $H' - H$ can be determined from Eq. (7).

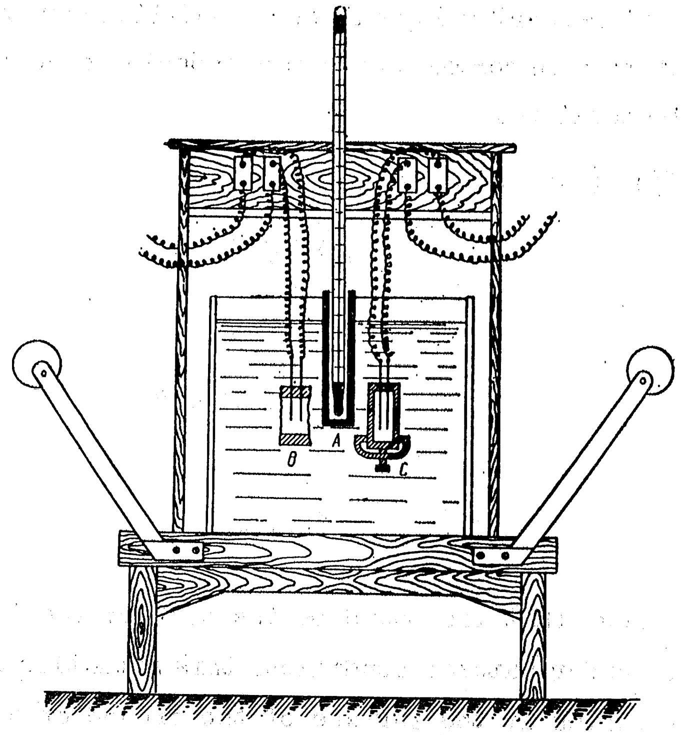
$$R = \sqrt{B\ (H' - H)} \tag{9}$$

and

$$H' - H = \frac{B}{R^2} \tag{10}$$

Actual mounds differ in shape from that of the theoretical right cone; they are trapezoidal (as in the case of stream valleys when the valley widens downstream), ellipsoidal or, in most cases, irregular. Nevertheless, Eq. (4) may be used as a sufficient approximation for any mound.

The theoretical considerations given above do not quite apply to "boolgoonyakhs" and to mounds connected hydraulically to adjacent mounds. The boolgoonyakh phenomena have not been sufficiently investigated yet.



A. Thermometer B. Rubber cylinder C. Metal cylinder

Fig. 15 — Petrov's Electrical Device for Testing Artificial Icings

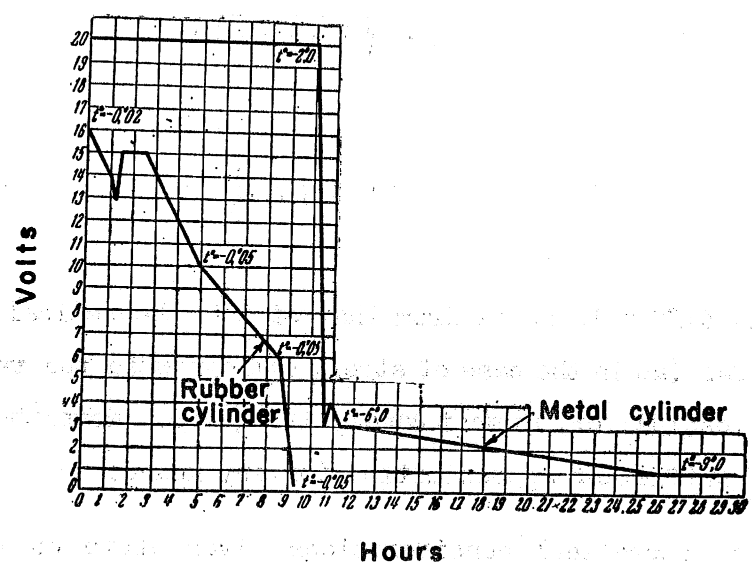


Fig. 16 — Curves Showing Relation between Duration of Freezing and Voltage in Petrov's Experiment on Artificial Icings

CHAPTER II

PRESSURE FORCES IN ICINGS

In 1930 V. G. Petrov conducted the first measurements of the pressure forces developed in an icing. He selected a site near Strelka on the Amur-Yakutsk Highway, where every winter a large icing formed and the stresses in the icing produced an icing mound. Special electrical measuring devices were used.

To check the results of these field tests, Petrov made a series of laboratory experiments on artificial icings, using instruments similar to those of the field tests. The design principles of the measuring instrument are based on the following properties of water:

a. Water is a conductor when in liquid state and a nonconductor when in solid (ice) state.

b. Conversion of water into ice causes an increase in volume.

c. Increase in pressure lowers the freezing point of water.

The experimental installation is shown diagrammatically in Fig. 15. The measuring device consists of the following five basic parts:

1. Rubber cylinder filled with water and connected to insulated conductors.

2. Metal cylinder filled with water, closed hermetically, and connected to insulated conductors.

3. Thermometer placed in the icing at the same depth as the cylinder.

4. Voltmeter and battery.

5. Vessel (25 cm in diameter and 25 cm high) with steel sidewalls (0.8 cm thick) and wooden bottom (4 cm thick).

The procedure is as follows: the vessel containing the cylinders and thermometer is filled with water and exposed to natural frost. The conductors attached to cylinders B and C lead to a control board located in a room. The circuit contains a voltmeter, battery, and commutator directing the current to either one of the two cylinders. The voltmeter registers the instant at which the water in the cylinders freezes, while the thermometer indicates temperature.

It is assumed that the water in the metal cylinder, which is not affected by the strains in the surrounding icing, will freeze at 0° C; the water in the rubber cylinders, which is affected by the pressure in the surrounding medium, will freeze at a relatively lower temperature. Experimental results proved the correctness of this assumption. The water in the metal cylinder froze at -0.10° C, while the water in the rubber cylinder froze at -0.50° C. The lower freezing point (-0.10° instead of 0°) in the case of the metal cylinder is attributed to the presence of salt in the water; hence, -0.10° C is taken as the freezing point of this water. This temperature difference of 0.4° C is produced by a pressure of 52 atmospheres within the icing if it is assumed that a pressure of 1.3 atmospheres is required to lower the freezing point of the water by 0.01° C.

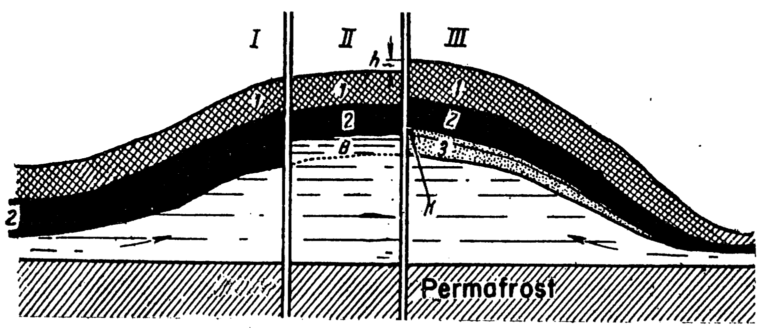
Figure 16 shows the results of Petrov's experiment; these results are in accord with those of the field tests. The curves show the relation between duration of freezing and voltage. The electrical resistance of the water in the cylinders increases with increasing formation of ice crystals; hence, the voltage decreases. Theoretically, these curves should decrease gradually from initial voltage to zero volts as the freezing action continues. In practice, however, the drop is not gradual but occurs in successive stages.

Proceeding from the assumption that the thawing action within the icing increases with increasing pressure, the process of icing formation can be divided into three distinct stages and explained as follows (Fig. 17).

It is assumed that at a given instant of the first formative stage the icing has an internal pressure of 130 atmospheres. This pressure lowers the freezing point to -1° C. If the lower layer that is not directly subjected to external frost has a temperature of -1° C, then physical equilibrium exists and freezing ceases.

It is assumed that in the second formative stage the internal pressure has increased to 260 atmospheres, which corresponds to a freezing point of -2° C. If the lower ice layer and the water below it exist at a temperature of -1° C, which is the temperature prevailing in the first stage, then the ice crystals of the lower layer will tend to melt under the effect of the high pressure. The water formed from the melting ice occupies a smaller volume than the ice, and existing pressure forces water from the surrounding water-bearing layer into this vacant space. Thus, the quantity of water in the icing increases.

The third formative stage occurs when the pressure acting on the lower ice layer is the same as in the second stage (260 atmospheres) and when the temperature of this ice layer and of the underlying water has reached -2° C as a result of increasing frost action. If frost action continues, the layer of water underlying the lower ice layer will freeze and expand in volume. This expansion must result in a rise in level of the passive ice layers; that is, it produces swelling. Thus, frost action on the outer and peripheral ground surrounding the icing causes periodic melting and freezing of the ice within the icing; this results in increased pressure within the icing and formation of a mound.



I. First Stage	II. Second Stage	III. Third Stage
1. Upper, passive ice layer with higher minimal temperature.	1. Upper, passive ice layer under increased pressure.	1. Upper, passive ice layer raised a distance h by ice wedge K.
2. Lower, passive ice layer with lower minimal temperature.	2. Lower ice layer, melting at increased pressure caused by increased frost action, yields water and space admitting additional water (B).	2. Lower, passive ice layer raised a distance h by ice wedge K.
Arrows indicate the pressure produced by the ground water the freezing of which proceeds from the periphery of the icing toward its center.		3. New, active, lower ice layer is formed from the water and increases in thickness by the magnitude K.

Fig. 17 — Petrov's Diagram Illustrating Three Stages in the Formation of an Icing

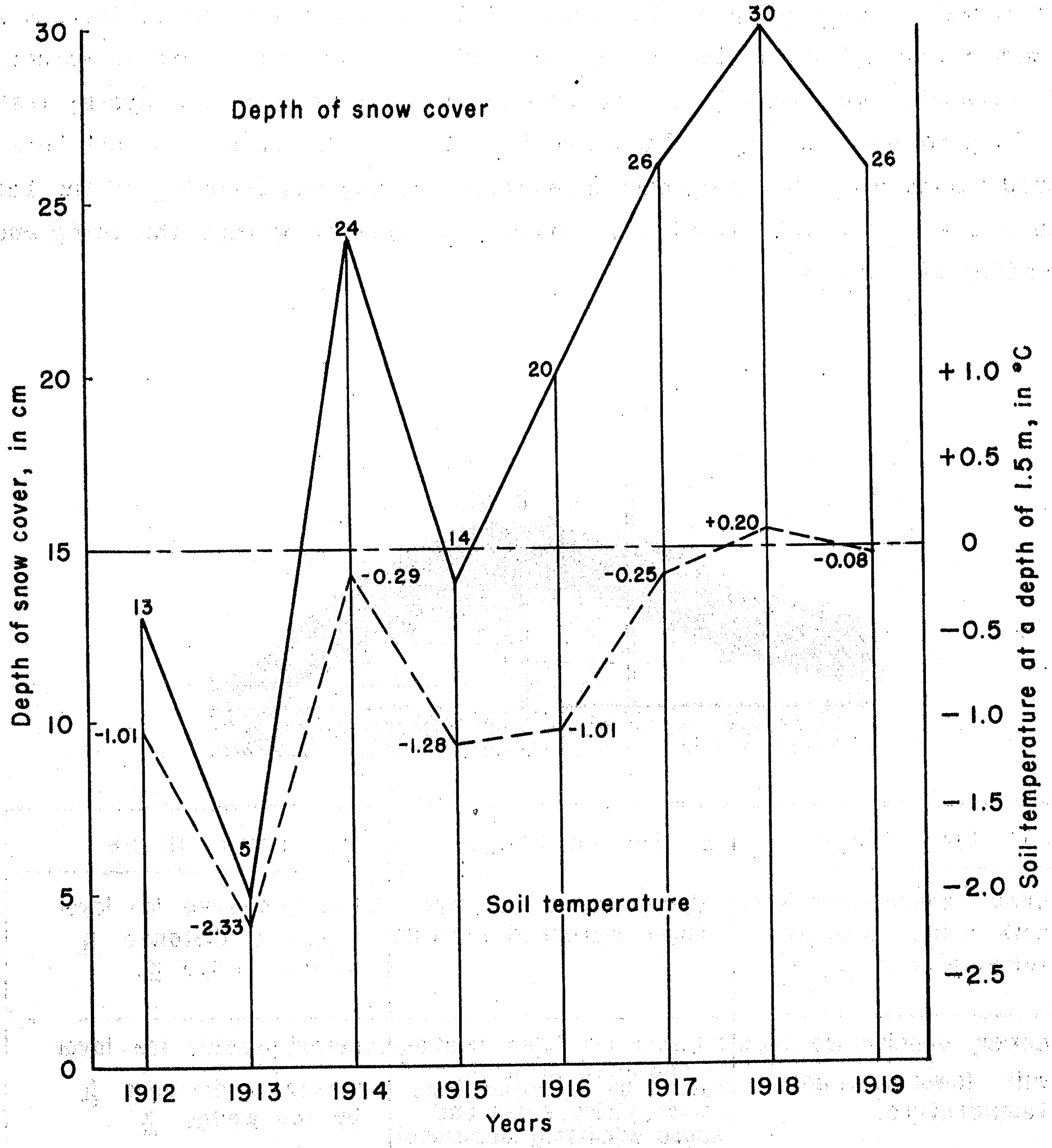


Fig. 18 — Soil Temperature at a Depth of 1.5 m and Depth of Snow Cover, Bomnak Meteorological Station, Siberia

CHAPTER III
EFFECT OF SNOW COVER

In southern Siberia, icings are most intense and attain their maximum spread in March when the cold cover is at its maximum. Snow acts as an insulating blanket, due to the air spaces between the snow particles, preserving in the underlying ground the low temperatures which were caused by the December and January frosts.

The insulating effect of snow is strikingly illustrated by the observations of P. I. Koloskov in 1915 at Gosh, Siberia (Table 2).

TABLE 2

INSULATING EFFECT OF SNOW

	Jan.	Feb.	March
Thickness of snow in cm	19	24	38
Temperature on the surface of snow in °C	-47.8	-40.5	-31.0
Temperature beneath the snow in °C	-19.2	-14.9	- 8.9

A slight difference in thickness of the snow cover may have a far-reaching effect on the behavior of an icing. V. G. Petrov states that one large icing which has been appearing regularly for many years failed to form during a winter with unusually heavy snowfall. M. I. Sumgin presents data (Table 3) obtained at the Bomnak Permafrost Station in Siberia, illustrating how a slight difference in thickness of the snow cover can affect the temperature of the ground. The corresponding curves are given in Fig. 18.

TABLE 3

EFFECT OF SNOW COVER

	Dec. 1913	Dec. 1914	Dec. 1918
Thickness of snow in cm	5	24	30
Temperature of the air in °C	-5.2	-4.7	-4.7
Temperature of the ground 1.5 m below surface in °C	-2.33	-0.29	+0.20

In areas with heavy snowfall during the early part of the winter, river and ground icings usually appear late in the winter, early in the spring, or may not appear at all. On the other hand, in areas with less than 1/2 m of snow or none at all, icings are likely to appear as early as December.

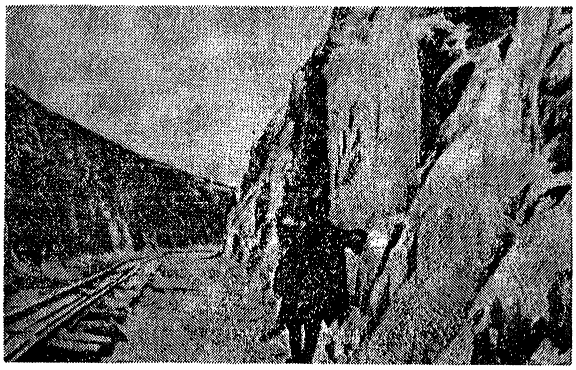


Fig. 19 – Icing in a Rock Cut

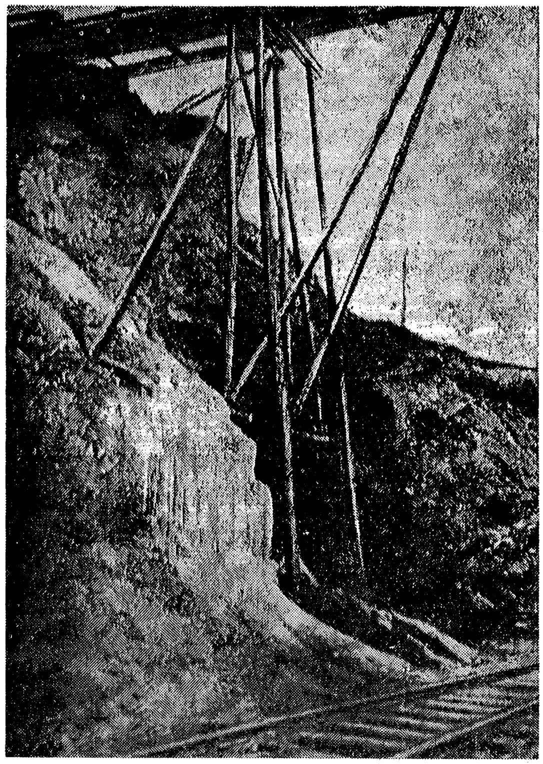


Fig. 20 – Icing on the Slope of a Cut

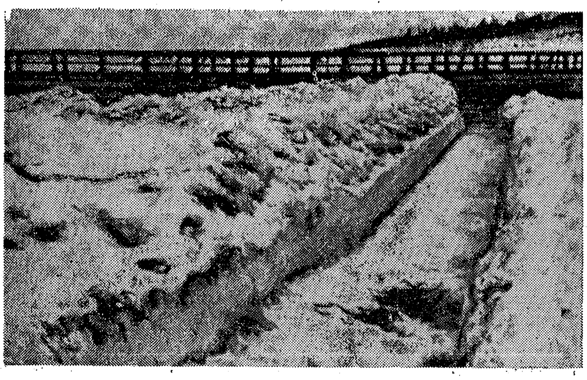


Fig. 21 – Icing Overflowing a Bridge across the Ayan River on the Amur–Yakutsk Highway

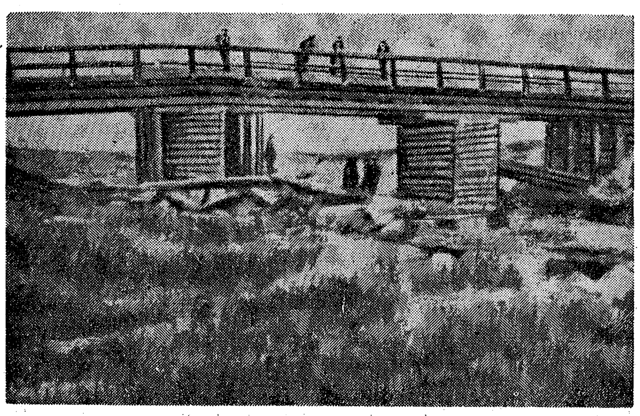


Fig. 22 - Wooden Bridge Deformed by Icing at the Nikolkin Kliutch River, Amur–Yakutsk Highway

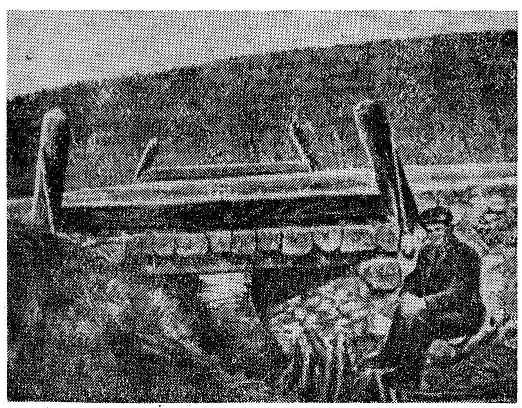


Fig. 23 – Typical Small Bridge at an Icing Formed by a Spring, Amur– Yakutsk Highway

CHAPTER IV

DESTRUCTIVE ACTION OF ICINGS

Both river and ground icings may cause extensive damage to bridges, roads, tunnels, buildings, and other engineering installations. Ground water seeping through cracks in the slope of a road cut, or surface water that is improperly diverted from the cut, will form icings endangering the roadbed and hampering traffic (Figs. 19 and 20). River icings are especially destructive to bridges and roads. The bridges may be deformed or completely destroyed by icings which occasionally attain such proportions that they engulf the bridge and overflow the road (Figs. 21, 22, and 29). The greatest damage to bridges and culverts is usually caused by explosions of icing mounds formed upstream (Fig. 55). This damage can be prevented only by proper diversion of the ground flow. It is recommended that bridges across small streams or across ditches have a minimum clear span of 3 m and a clearance of 1.5 m above the surface of the water (Fig. 23). Heating or insulation of the channel is feasible under these conditions.

Icings in tunnels may be formed from ground water seeping through the walls, arch and bed, from condensation of air and steam, or from freezing of the drainage ditch in the tunnel. Icicles formed in the tunnel occasionally attain a length of 1.5 to 2.0 m and a thickness of 40 to 50 cm (Fig. 24). Icings in a tunnel (Fig. 25) tend to interfere with traffic; current measures against these icings, none of which is entirely rational or preferable, include removal of the ice at regular intervals, heating the tunnel by steam pipes laid between the tracks, insulation of the outer surfaces of the walls and arch, and application of artificial freezing. Buildings erected on permafrost cause the underlying active layer to thaw out or lessen the effect of frost action on this layer; consequently, the ground water can readily seep beneath or near the buildings and form icings (Figs. 26 and 27) that are difficult to eliminate or to render harmless.

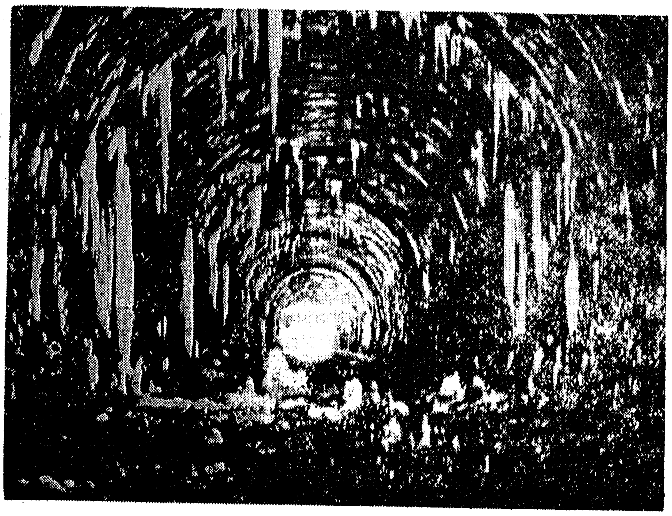


Fig. 24 — Icicles in a Tunnel

Fig. 25 — Icing Beginning to Form in a Tunnel

Water seeping through unfrozen ground beneath the house fills the building with ice.

Fig. 26 — Icing in Barracks

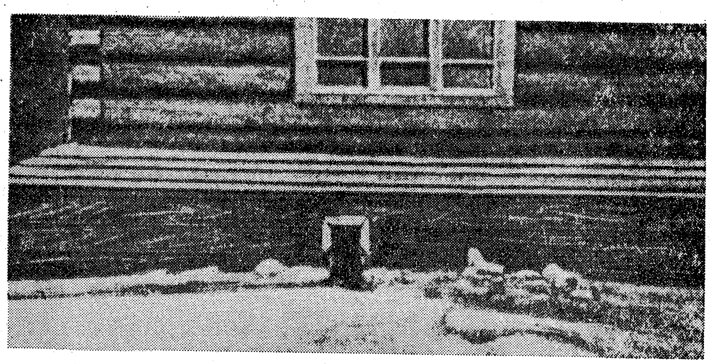


Fig. 27 — Icing Formed by Water Emerging through a Vent

CHAPTER V

MEASURES AGAINST ICINGS

A. General Characteristics

The destructive action of icings on bridges, roads, and other engineering installations can be eliminated or ameliorated by either the passive or the active method. The passive method consists of measures against the effects of existing icings, while the active method is used to eliminate the causes of icing formations at a given site.

The passive method includes the following five distinct measures:

1. Removing the icing ice.
2. Diverting the water feeding the icing.
3. Constructing barriers against the icing.
4. Enlarging the cut in which the ice forms.
5. Relocating the site of the structure.

The active method includes the following four effective measures:

1. Draining the site of the icing.
2. Constructing frost belts.
3. Deepening and straightening the river channels.
4. Insulating the stream channels.

Effectiveness of design and construction of engineering installations in the permafrost region depends on proper survey of the region under consideration. The data should include observations made during both summer and winter and should furnish the following information:

1. Causes of icing formation.
2. Origin of water feeding the icing (river, suprapermafrost, or subpermafrost).
3. Topography of the region.
4. Structure of river valley.
5. Character and amount of river deposits.
6. Character and amount of vegetation cover.
7. Period and intensity of frosts.
8. Period and intensity of snowfall.

B. Removing the Icing Ice

Cutting away and removing the icing ice forming on roads, in cuts, and near bridges constituted one of the earliest passive methods (Fig. 28). This method is relatively impractical and not entirely effective. The highways in Alaska are closed to traffic from about October to June because of icings, and the bridges are occasionally completely covered by the icings (Fig. 29). To prevent the destructive action of the icings and to keep the roads open, it is necessary to cut away and remove the ice. This can be done with the aid of any mechanical or other means available.

C. Diverting the Water Feeding the Icing

Suprapermafrost waters feeding an icing can be diverted to a suitable spot by ditches insulated to keep the temperature of the flowing water above freezing (Fig. 30). The insulation usually consists of planks covered with a 50-cm layer of peat and moss. This current practice appears to be unsatisfactory because the insulating cover delays the thawing of the ditches in the spring when they are needed most. It is recommended, therefore, that the diversion (as well as drainage) ditches be uncovered during summer and covered during winter (Figs. 31 and 32).

Explosion of icing mounds and resulting damage to the engineering installation can be prevented by puncturing the upstream face of the mound and diverting the emerging water through a suitable ditch. Such an operation has to be repeated at regular intervals, depending on local conditions.

An early method utilized actual heating of the water in the ditch of a railroad cut (Fig. 33). The icing water can be diverted also through troughs formed between two adjacent ties by removing some of the gravel (Fig. 34). This procedure necessitates relatively continuous removal of the ice forming in the trough and at its downstream side. Surface water and ground water at shallow depths are prevented from reaching the roadbed in the cut by a cutoff ditch. Figures 35 and 36 show typical insulated drainage and diversion ditches used on the Amur-Yakutsk Highway.

Water of a river icing is diverted as follows: two holes are made in the ice at the upstream and downstream sides of the river at a distance of about 5 to 6 m from the bridge, and these holes are connected by a channel cut in the ice. This channel is kept open by cutting away the ice as soon as it begins to clog the channel.

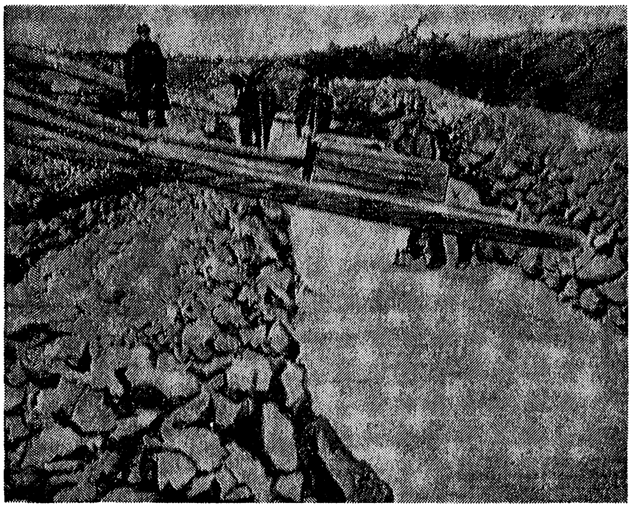


Fig. 28 — Removing Excess Icing

Fig. 29 — Bridge in Alaska Covered with Icing to Top of Railings

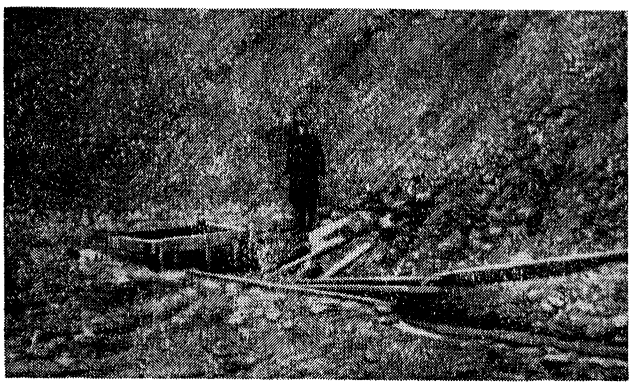


Fig. 30 — Tapping a Spring Issuing from the Slope of a Cut

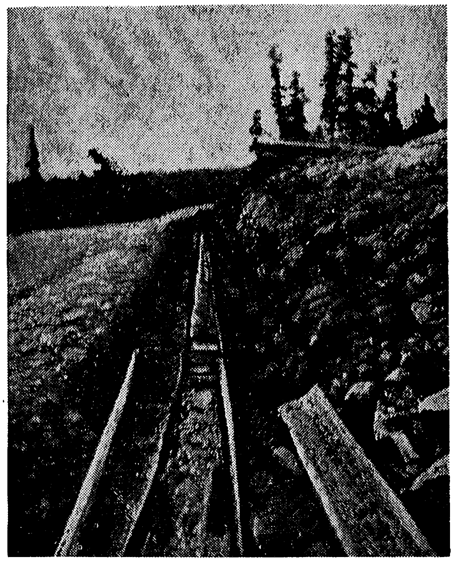
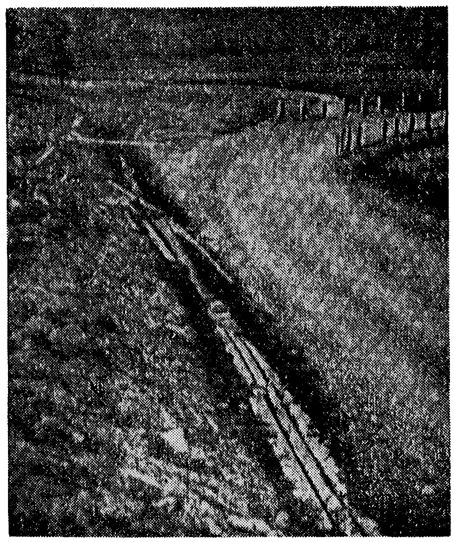


Fig. 31 — Insulated Ditch Uncovered During Summer, Amur-Yakutsk Highway



Insulation not yet in place.

Fig. 32 — Insulated Ditch Covered During Winter, Amur-Yakutsk Highway

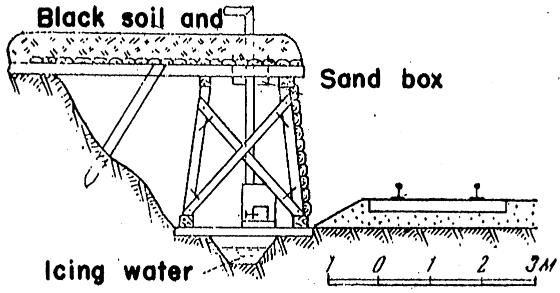


Fig. 33—Insulated Ditch in a Cut

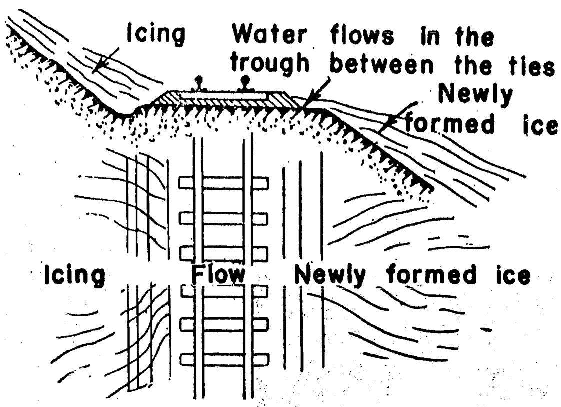


Fig. 34—Draining the Icing Water in a Cut by Ballast Troughs

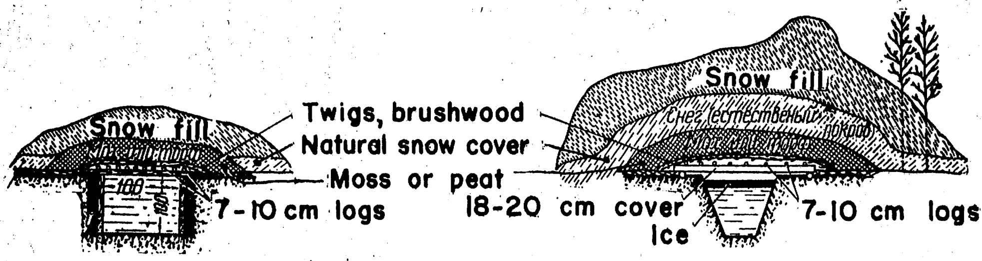


Fig. 35—Insulation of Drainage Ditches, Amur-Yakutsk Highway

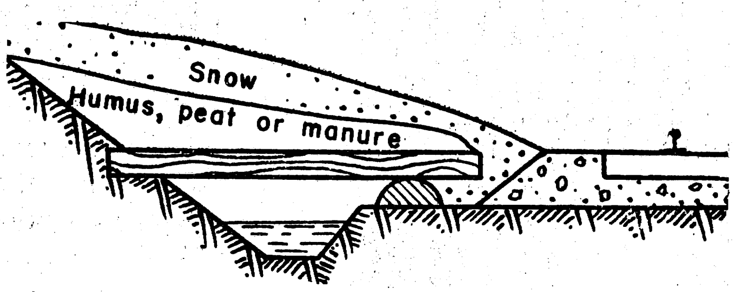


Fig. 36—Diagram Showing Insulation of a Ditch for Drainage in Winter

D. Fences and Barriers

Fences and barriers may be used to stop the advance of an icing towards the structure (Figs. 37 and 38). These barriers are erected separately or in addition to other protecting means. Stone and coarse soil are the most suitable materials, but snow, logs, and other available materials can be used. Normally the barrier is 3 m high and its width at the base must be not less than half its height.

E. Widening of Road Cuts

If water seeps through cracks in the rocky slopes of the cut, an icing forms and overflows the road, disturbing the safety and continuity of traffic. Widening the cut on the side of the icing may eliminate this undesirable effect (Fig. 39). However, such a procedure is not always rational and economical, in which case it may be necessary to divert the water feeding the icing or to construct a frost belt.

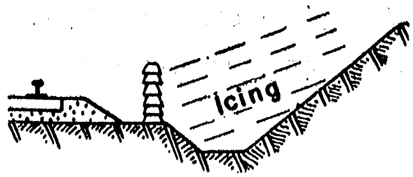
F. Relocating the Site of the Structure

A structure erected on permafrost may cause formation of icing and may be subjected to the destructive action of the icing if the structure penetrates the ground-water table or if it is located on a frozen suprapermafrost layer. The layer beneath the structure would thaw and the water would emerge and form the icing (Figs. 26 and 27). Under these circumstances, it may be advantageous from both the technical and economical viewpoints to relocate the structure on a site where no danger of icing formation could arise.

G. Drainage

Drainage is a basic, active measure against icings. If areas with excessive moisture are drained and seepages are checked, icings cannot form in the area and it can be used as a site for the projected structures (Fig. 40). The type and dimensions of the drainage ditches depend on the relief of the terrain and the hydrological and climatic conditions of the region. Normally, shallow cutoff and diversion ditches are satisfactory. Trapezoidal ditches 2 m deep, 0.8 m wide at the bottom, and 5 m wide at the top are commonly used in the permafrost region.

Where the permafrost table is fairly close to the surface, the composition of the upper part of permafrost must be taken into consideration.



Barrier prevents icing water from reaching the roadbed in the cut.

Fig. 37—Barrier Constructed from Used Railroad Ties

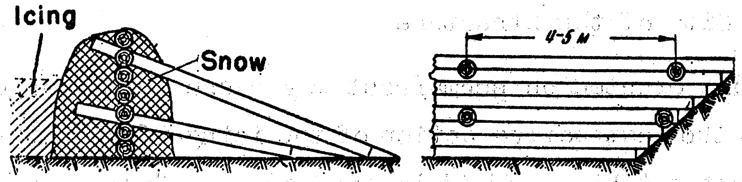


Fig. 38—Timber Barrier Against Icing, of Type Used on the Amur-Yakutsk Highway

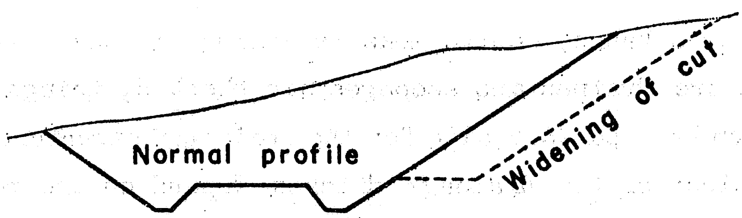


Fig. 39—Diagram Showing Cut Widened Towards the Icing

Where permafrost is composed of silt and fine sand with a considerable proportion of ice, drainage will not give satisfactory results because the thawed ground will become plastic or semifluid. Under such conditions, drainage ditches will slump and erode, eating into the surrounding ground and endangering the newly-erected structures. Apparently there is no fully satisfactory method of drainage under these conditions.

The critical time when drainage should function properly is during the spring. Current practice of covering the ditches with insulating layers of peat and moss appears to be unsatisfactory because this cover delays the thawing of ditches in the spring when they are most needed. It is recommended that drainage should be stopped in the autumn, when it is not required, and speeded up in the spring. It is also recommended that drainage ditches be filled with material of high heat conductivity, such as boulders, gravels, or coarse sand, capped with a layer of clay to prevent silting.

Drainage measures should be carried out considerably in advance of building, as there is always the possibility of some unexpected, unfavorable effect on the ground that has been selected as the site.

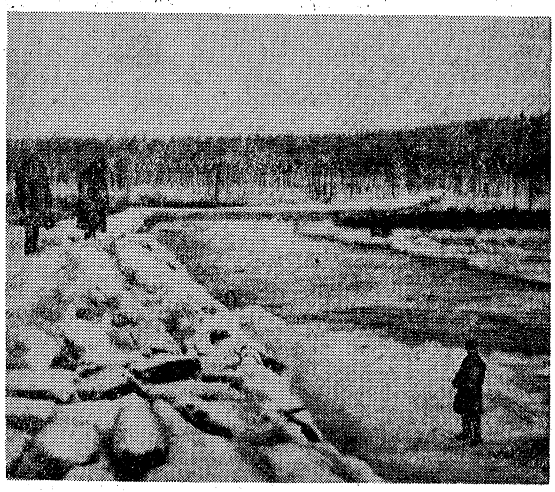
Excessive moisture of the ground may be due to inflow of surface waters or to underground seepage. Elimination of surface water does not present any serious problem, but water percolation underground is more difficult to control. It has to be intercepted and diverted into deep wells or sumps. Sinking of wells requires particularly thorough investigation of the hydrological conditions. Existence of strong seepage from below the permafrost may result in rapid filling of the well and may even cause an overflow which would greatly aggravate the situation.

H. Frost Belts

A satisfactory preventive or corrective measure against icings, known as the frost belt, has been developed by V. G. Petrov. It consists of a specially constructed ditch, the function of which is to drain the water and, primarily, to cause early and complete freezing of the active layer at a distance from the road or structure sufficient to prevent damage by the icing. Vegetation is removed and a ditch about 1 m deep is dug some distance from the road or structure. Subsequent freezing of the ground is most intense along this ditch. The freezing will reach the permafrost level and form a

Drainage made it possible to plant an orchard on this site.

Fig. 40 — Site where Icing Mound Would Form before Drainage of Area



Icing water flowing from beneath the snow stops at the belt and freezes.

Fig. 41 — Effect of Frost Belt

frozen wedge across the unfrozen layer. This wedge prevents the ground water under the ditch from percolating into the area near the road or structure. As a result, an icing will form within the confines of the frost belt (Fig. 41) at the given distance uphill and away from the structure. Such a belt is called a permanent or ground frost belt; it is constructed normal to the course of flow. The action of the frost belt is illustrated in Fig. 42.

The ground frost belt should be adequately wide (sometimes 5 to 10 m, depending on local conditions) and 0.5 to 1.0 m deep. Occasionally it is judicious to construct two or more belts (Fig. 43) approximately parallel to each other; the width of each belt may be reduced in this case. The belt may be 1 m deep and only 5 m wide if it is bordered on the uphill side by an additional strip, called a wing, 10 to 15 m wide and stripped of vegetation and sod. In such a case, the profile of the frost belt resembles the outline of a dipper (Fig. 44). The belt should be located 50 to 100 m from the road and should be as long or slightly longer than the icing which is to be eradicated. The spoil (ice as well as ground) accumulated during construction operations should be piled on the downhill side of the belt; if any snow is present, it should be scraped uphill and deposited near the upper edge of the belt.

Frost belts are used extensively to protect roads and bridges against the destructive effects of icings. Seepage of ground water or river water from beneath the ice may freeze on or near a road in successive sheets of ice that may attain a thickness of several meters. This occurrence is attributed in part to the disturbing effect of the construction process on the regime of the active layer and the permafrost. During construction, the vegetation and snow covers are removed and the exposed ground is tramped down; this accelerates the freezing of the ground beneath the road and forms a frozen wedge causing the seepage to emerge to the surface through the adjacent undisturbed ground. Along the new Amur-Yakutsk Highway in Siberia, more than 100 large icings occurred within a distance of about 700 km. Stretches of road, each several hundred meters long, have been rendered entirely impassable because of these icings. Both the river and ground icings along the road showed a remarkable alinement with the roadbeds, following every curve of the road, as shown in Figs. 45 to 48.

If large icing mounds form above the frost belt (Fig. 49), they should be punctured and drained before they explode and cause damage. They

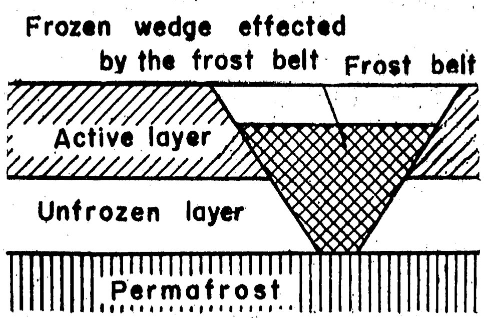
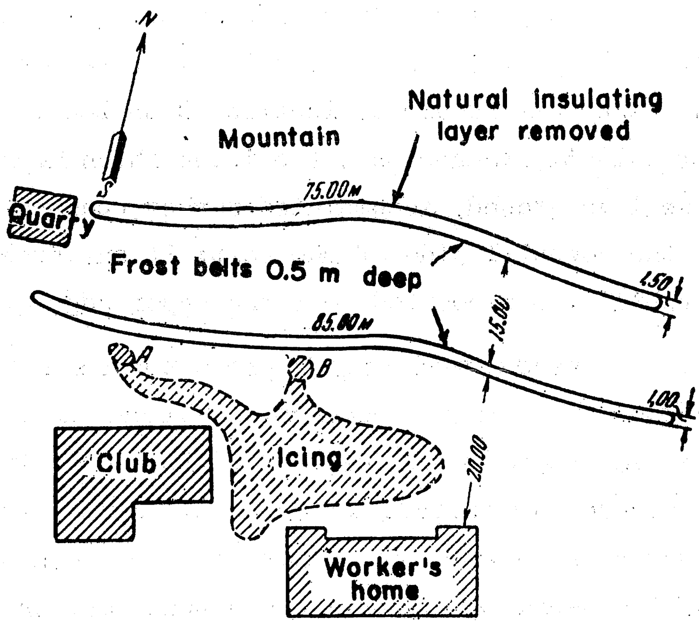


Fig. 42 — Diagram Illustrating Effect of Permanent (Ground) Frost Belt



A, B. Piles of lumber waste
Water flowing from beneath
the piles formed the icing.

Fig. 43 — Frost Belt for Ground Icing

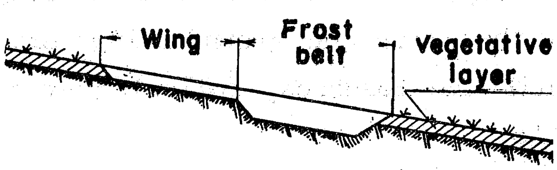
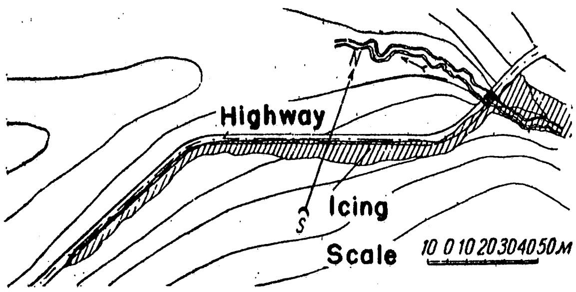
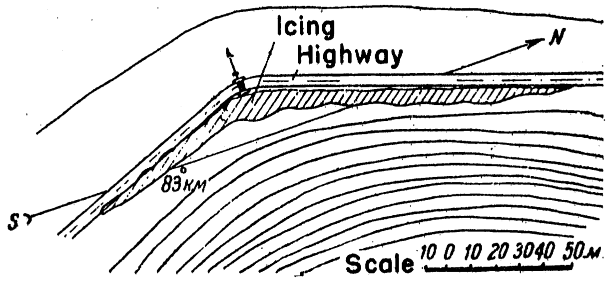


Fig. 44—Diagram Showing Arrangement of Frost Belt with a Wing for Ground Icing

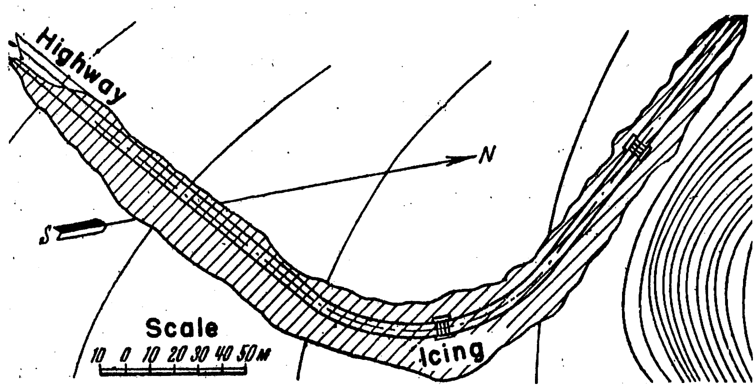




Construction of highway causes formation of icing which closely follows course of highway.

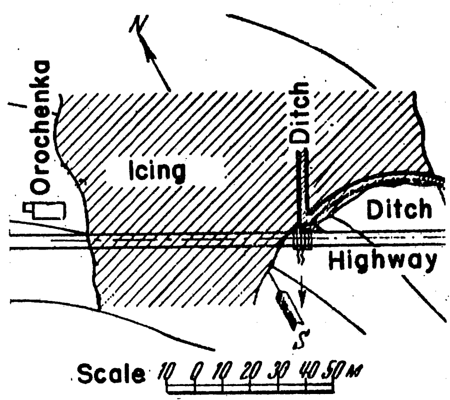
Fig. 45 – Plan of Icing on the Amur-Yakutsk Highway near Sosnovsk

Fig. 46 – Plan of Icing along the Amur-Yakutsk Highway



Construction of highway causes formation of icing which closely follows course of highway.

Fig. 47 – Plan of Icing along the Amur-Yakutsk Highway



One of the ditches stopped advance of icing.

Fig. 48 – Plan of Icing on the Amur-Yakutsk Highway near Orochenka

may be punctured either by simply chopping the ice, by blasting, or by melting the ice with thermite.

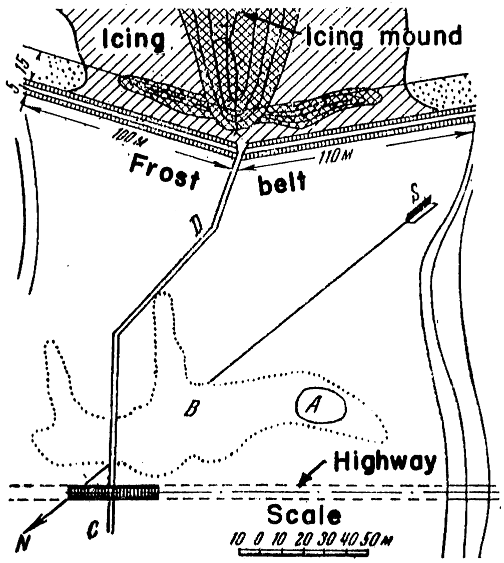
To be effective, a frost belt should be constructed early in the winter before the first snowfall, preferable before the beginning of freezing weather. The snow should be kept off the frost belt until the end of January. After January, the snow may be left on the frost belt, as it will tend to preserve the low temperature of the ground.

The frost-belt method gives satisfactory results only if the icing is fed by surficial water. The method is likely to be less effective or may fail altogether if the icing water comes from a deep source. It is suggested that icings fed by deep waters can be better controlled by diversion ditches. Hence, since the source of water forming the icing is not always readily determined, planning of preventive measures should be preceded by a thorough study of the icing.

A slightly different procedure is used in the case of a frost belt designed to offset the destructive effects of a river icing. One type of belt, used when the road crosses a shallow river which completely freezes in winter, consists of a ditch located 200 to 300 m upstream of the road and parallel to it (Fig. 49). The ditch in the river ice should be 3 to 5 m wide and about 2 m deep. Excavation of the ice is usually started as soon as the river ice is sufficiently thick to permit operations and is gradually continued until the river bed is reached. The ends of the ditch are dug in both banks of the stream; they are usually constructed during the summer. The spoil is deposited on the downstream side of the ditch to form a dike retaining the icing water and preventing inundation of the road (Fig. 50). The upstream wall of the ditch should have a mild slope.

Should an excessive amount of water accumulate in such a frost belt, it may become necessary to cut a drainage ditch in the ice downstream and under the bridge. Cribbing set in the bed and some distance above the road may also act as a frost belt and shift the formation of icing upstream and away from the road.

Destructive river icing may also be averted by building snow banks on ice across the river at shallow places and near rapids above the road. At the same time, a hole should be chopped in the ice below the road to drain the water, thus removing the possibility of creating hydrostatic pressure. A



A. Slight swelling of swampy ground B. Site of former icing mound
C,D. Drainage ditch along one of the stream beds

Fig. 49 — Effect of Frost Belt

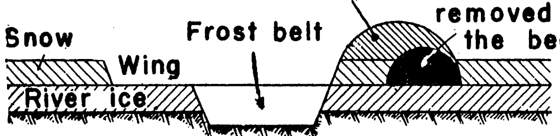
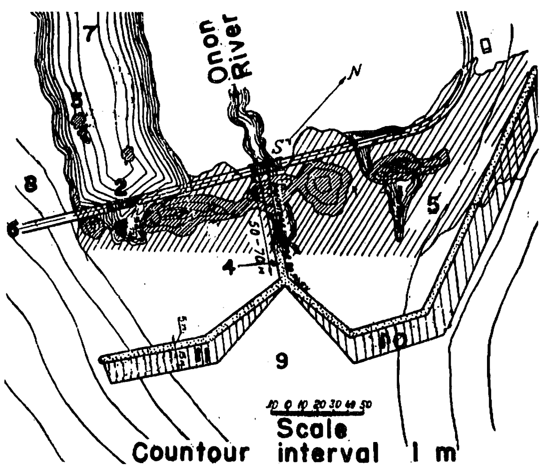


Fig. 50 — Diagram of Frost Belt with a Wing for River Icing



Flooded area with slabs of ice as a result of the explosion of an icing mound.

1. Site of explosion 2. Bridge washed out 3. Slabs of ice from exploded mound 4. Drainage ditch excavated in ice 5. Icing 6. Highway 7. Icing water 8. Woods 9. Frost belt 10, 11. Wing

Fig. 51 — Plan of Icing along the Amur-Yakutsk Highway at the Onon River and Plan of a Projected Frost Belt

mild river icing may be diverted from a road by sufficiently high earth-filled shoulders (levees).

The behavior of a river icing, its destructive effect on a road, and the plan of a projected frost belt to remedy the situation are illustrated in Fig. 51 showing the Onon River icing on the Amur-Yakutsk Highway, 124 km north of Skovorodino, Siberia. The Onon River valley is about 250 m wide at the point where the highway crosses it. The bottom of the valley is bounded on both sides by steep slopes with small mountain brooks flowing into the Onon River. Because of their rapid flow, these tributaries freeze in the winter much later than the Onon River. In the bottom of this valley, permafrost begins at 0.6 m below the surface; on the sides of the valley, however, permafrost is encountered at a depth of 1.5 m. During the winter, the percolating ground water beneath the frozen surface is confined in a narrow passage or blocked and trapped where the active layer freezes through to the permafrost. Such an obstruction to the underground seepage of water was created along the sides of the highway where the surface has been scraped or tramped down, permitting the ground underneath it to freeze to a greater depth much earlier than in the adjacent undisturbed ground.

Six icing mounds formed along this part of the road (Fig. 51). After preliminary cracking and trembling, the second mound from the south (indicated by 1) suddenly exploded. Large slabs of ice (2 m thick and up to 19 m long) were thrown out and carried 120 m down the valley across the road by a rushing torrent 75 m wide. The water which poured out from the exploded mound spread down the valley for a distance of about 5 km. A small highway bridge in the path of this torrent was shaved off to its foundation, shrubs were flattened down to the ground, and the bark on large trees was badly scarred by the moving pieces of ice. This catastrophic action lasted only a couple of hours. Prior to the explosion, the sixth icing mound, about 150 m to the northeast, was also showing signs of unrest, but the explosion of the second mound relieved the underground hydrostatic pressure and caused the sixth mound to subside and to cease cracking.

The Onon River icing described above clearly demonstrates the importance of hydrostatic pressure in the formation of mounds and their destructive effect on engineering structures. The projected frost belt is intended to eliminate this icing at the road by facilitating its formation at the site of the belt.

The position and number of frost belts for a given icing are determined in each case in accordance with local conditions. However, to be effective the belt must extend at least the entire length of the icing and must be sufficiently distant from the structure endangered by the icing. The results of inadequate length of the belt and its closeness to the structure are demonstrated in Fig. 52.

The inadequacy of Petrov's frost belt method is that, after one or two years of successful operation, the level of the permafrost table will develop a sag corresponding to the surface profile of the ditch, and the percolation of water will continue unchecked to the fill of the roadbed or to the building. To avoid this, it is recommended to fill the ditch or cover it with some insulating material in the spring and to reopen it in the fall, as well as to clean the ditch after each snowfall until February. Such maintenance of the frost-belt ditch is undoubtedly effective, but it is costly.

A variant of frost belt, proposed by Bykov and Kapterev, appears to be most satisfactory. Instead of the regular ditch, a trench 1 to 1.5 m wide is cut across the waterbearing layer to a depth of about 2 to 2.5 m. This trench is filled with clay, or with water-saturated silt or clay, and is thoroughly tamped. Then a row of pile planks is driven into this ground, and both sides of the exposed planks are covered with fill. This fill will cause the permafrost table to rise; as a result, the planks will be firmly anchored. The water percolating down the slope will be forced to the surface by the frozen ground and the impervious wall of the trench, and will form an icing on the uphill side of the barrier.

An effect similar to that produced by the ground frost belt can be achieved by constructing a protective embankment or fill of insulating material (peat or moss) or spoil above the active layer (Fig. 53). This embankment is less costly than the regular frost belt and requires little maintenance. The permafrost surface beneath the insulating fill rises until it merges with the lower surface of the active layer, thus forming an impermeable wedge barring the percolating water. The fill should be 1 m high if made of peat, and 2.0 to 2.5 m high if made of spoil. The total height can be reduced if the spoil covers a 50-cm layer of peat or moss. The embankment should be constructed in the spring; it achieves full effectiveness in two or three years. The use and relative effectiveness of protective embankments have not been studied sufficiently and cannot be definitely recommended as yet.

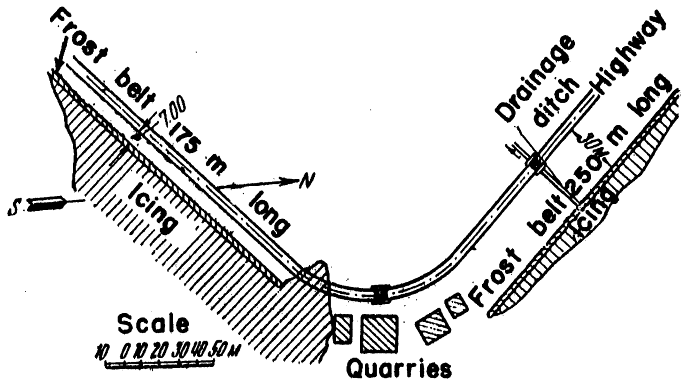
A variant of the ground frost belt and the protective embankment is the seasonal or snow frost belt. This belt consists of a strip of ground from which the snow is cleared and piled on the downstream side of the strip. The area free from snow freezes more rapidly than the adjacent areas under snow cover, thus facilitating the formation of the frozen wedge.

The principle of the snow belt was discovered at the Kitayanka icing on the Amur-Yakutsk Highway. As shown in Fig. 54, an ordinary pathway (1), formed in the snow by a sled hauling water from an ice hole (5), prevented the advance of the icing toward the highway (9). The pathway acted as a frost belt. A second pathway (2), formed in the same way as the first one, had a similar effect on the icing. Subsequently, the engineers experimented by removing the snow from a series of strips normal to the flow and succeeded in causing the icing to recede to within 1 km from the highway.

The effect of the seasonal frost belt is demonstrated in Figs. 55 and 56. The action of such a belt can be intensified by removing the vegetation cover as well as the snow from the strip; then the belt is a modified form of the regular ground frost belt which includes a ditch excavated in the ground beneath the cleared strip. Numerous belts of the snow type were used on the Amur-Yakutsk Highway. These belts can be constructed more rapidly and cheaply and constitute a more elastic measure against icings than ground frost belts because they can be readily erected in numerous rows, if necessary, and are not limited to definite minimum distances from the engineering installation. Moreover, their effectiveness is generally comparable to that of ground frost belts if the snow is removed from the strips after each snowfall and the drifts are cleaned during the interval between snowfalls. The seasonal belts must have drainage ditches insulated for winter drainage; the ditches serve to drain the suprapermafrost waters and the water issuing from exploding icing mounds ordinarily forming above the belts.

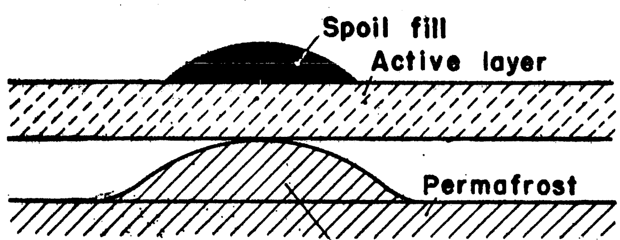
I. Deepening and Straightening the River Channels

River icings tend to form on sandbars, in shallows, along small branches of the river, and wherever depth and velocity of flow are relatively small. Hence, formation of icings at such points can be prevented or ameliorated if depth and velocity of flow are increased. This is accomplished by cleaning, deepening, and straightening the main channel (Figs. 57 and 58). These measures eliminate the sandbars and shallows as well as the flow in the



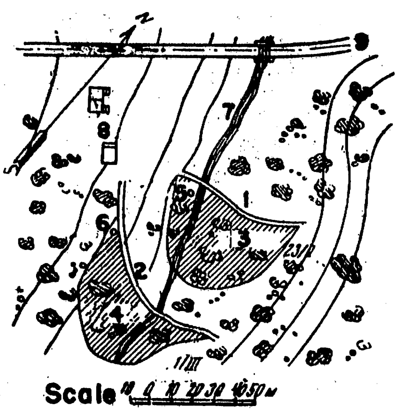
Gap between belts and insufficient distance between the shorter belt and the highway (7 m) make possible advance of icing toward structures and highway.

Fig. 52 — Plan of an Incorrectly Designed Frost Belt



Permafrost surface beneath the embankment is raised.

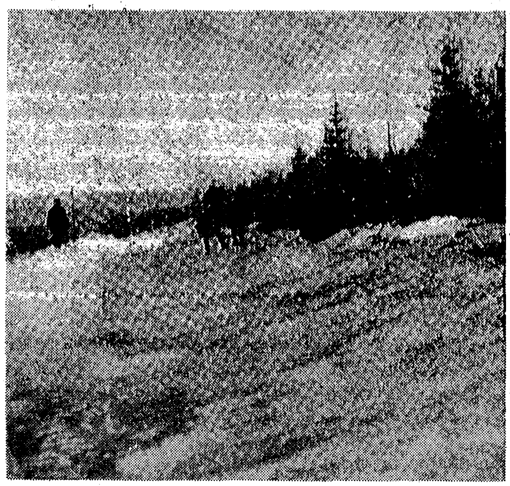
Fig. 53 — Diagram Illustrating Effect of Protective Embankment



1, 2. Pathway
3, 4. Icing
5, 6. Ice hole
7. Kitayanka River
8. Station Kitayanka
9. Highway

Pathways 1 and 2 prevent advance of icing toward the highway.

Fig. 54 — Plan of Icing at the Kitayanka River, Amur-Yakutsk Highway



Advance of icing is stopped at the belt.

Fig. 55—Seasonal Frost Belt in Action

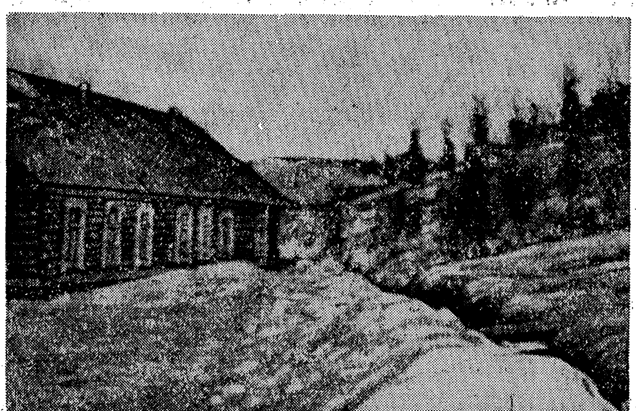


Fig. 56—Effect of Seasonal Frost Belt at Strelka, Amur-Yakutsk Highway

small branches, thus producing the desired greater depth and higher flow velocity which retard freezing and icing formation. If deepening and straightening the channel prove insufficient to prevent formation of icings in a given case, additional measures such as frost belts or insulation of the channel can be used.

Care must be taken that the vegetation cover on the banks of the river is disturbed as little as possible during the operations of deepening. Should this cover be removed, it must be replaced by new vegetation and sod to maintain the insulation of the active layer.

J. Insulating the Stream Channels

Inadequate snow cover causes stream beds under bridges to freeze earlier and more intensely than the surrounding ground covered with snow. The resulting icing formed at a bridge may cause deformation or complete destruction of the structure. It is essential, therefore, to prevent or ameliorate the formation of such an icing. This can be accomplished by a frost belt or other active methods. Engineers on the Amur-Yakutsk Highway made a comparative study of the various methods used on the highway and of the respective results. They took into consideration the effect of period and intensity of snowfall and the fact that frost belts proved unsuitable for relatively deep upper permafrost surface under the river bed; they concluded that the most rational and economical method is to insulate the river channel near and under the bridge. This procedure results in free flow of the stream and ground waters in the region of the bridge, reduces the freezing of the water-bearing layer which remains unfrozen during the entire winter, and thus offsets the formation of destructive icing near the bridge.

The effectiveness of the preventive method of insulating the beds can be increased by constructing a row of snow embankments across the water course at shallow places and near rapids upstream of the bridge. The first snow belt should be located about 100 m from the bridge, depending on local conditions. These embankments facilitate continuous movement of the water under the bridge during the winter. At the same time, holes should be chopped in the river ice downstream of the bridge to drain the water obstructed by downstream sandbars or shallows, thus preventing the occurrence of hydrostatic pressure and its undesirable effect.

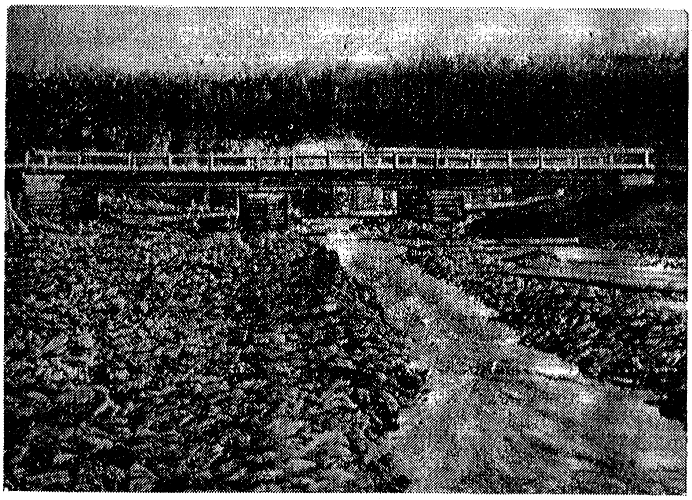


Fig. 57 – Deepened and Straightened Channel of a River at the Amur-Yakutsk Highway

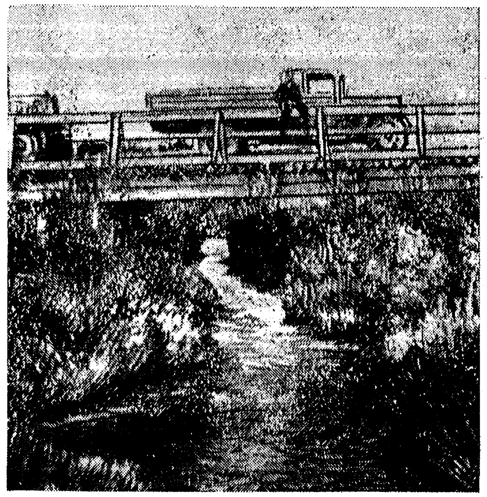
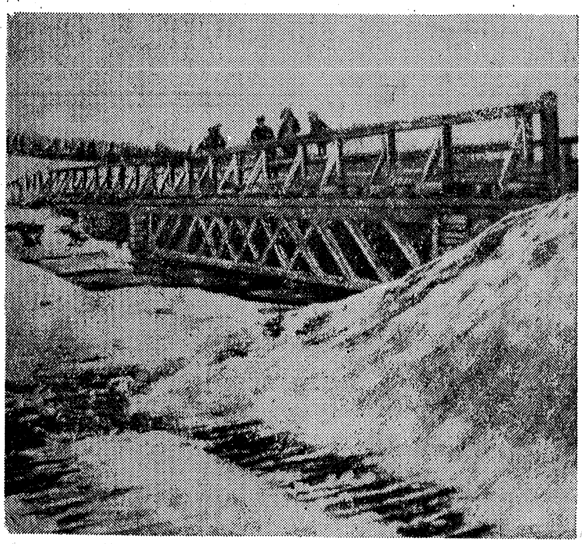
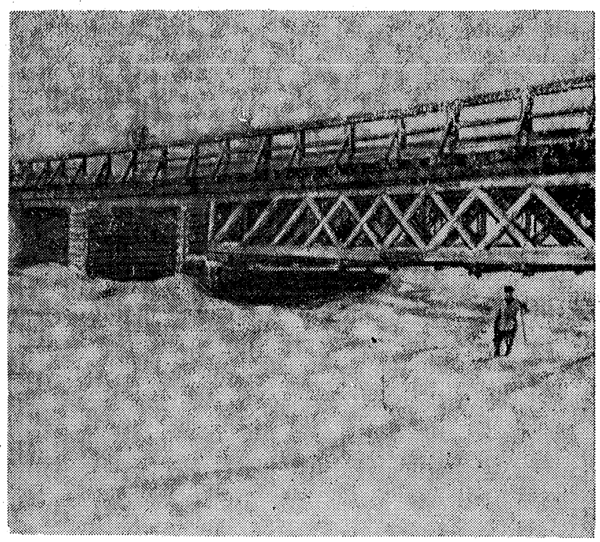


Fig. 58 – Deepened Channel of the Taloy River at the Amur-Yakutsk Highway



Icing reaches bridge structure.
Part of the Ice has been cut away and removed, May 1934.

Fig. 59 – Bridge across the Ayan River at the Amur-Yakutsk Highway Prior to Insulation of River Channel



Prevention of icing accomplished by insulation of river channel. March 1935.

Fig. 60 – Same Bridge after Insulation of River Channel

Figure 59 shows a bridge and the icing formed near it prior to insulation of the channel. The icing extends over an area of 14,000 m^2, is about 1 m thick, and reaches the lower chord of the truss. Part of the ice has been cut away and removed. A large icing mound is located 10 m upstream. Figure 60 shows the same bridge after insulation of the channel. The icing has disappeared. The insulation consisted of logs covered with brushwood overlaid by a snow fill and extended 100 m on each side of the bridge.

Use of the insulation method is recommended in cases where frost belts are likely to be insufficiently effective. The method is highly satisfactory in the northern regions with ample snowfall; it requires practically no winter maintenance.

The insulating cover usually consists of two layers. The lower layer is composed of logs, brushwood, conifer branches, and similar materials; it should be 0.3 to 0.5 m thick, depending on the materials used. The upper layer is an untamped snowfill 0.5 m thick. The lower layer serves merely as a base for the snow layer which acts as the main insulator. In regions of relatively shallow snow cover, the lower layer is covered by an additional layer of peat or moss (Fig. 61). Care must be taken that the peat is not compacted and is not mixed with fine soil during the operation, so that it would retain a maximum of insulating air. The insulating cover should be installed in the autumn before the beginning of frost and snowfall. The part of the cover under the bridge should extend across the entire width of the bridge and a minimum of 1 m into each bank. The effectiveness of this insulation method is enhanced if the insulation is extended along the channel to points where the banks are steep and covered with considerable vegetation.

Small streams and ditches can be insulated by a layer of air formed under an ice crust. The method is quite effective. The stream is temporarily dammed until the water rises to a predetermined level. Then the water is allowed to freeze until an ice crust 12 to 15 cm thick is formed, after which the dam is removed and the water level is lowered to normal. Thus, an air layer 0.2 to 0.3 m thick is formed, and its insulating properties are sufficient to prevent freezing of the flowing water. The effectiveness of such an air layer is increased by an additional layer of peat, moss, snow, or similar materials of low heat conductivity, covering the ice crust (Fig. 62). This method functions satisfactorily if care is taken that the discharge end of the insulated ditch remains unfrozen and does not create hydrostatic pressure.

In regions where strong winds prevail, special snow fences are erected along the entire length of the insulated channel or ditch in such a way that the snow will accumulate on the insulated bed. These fences may be replaced by conifer branches stuck in the snow along the bed, as shown in Fig. 63.

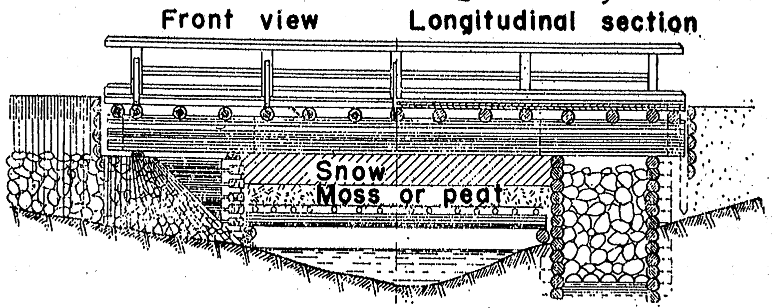
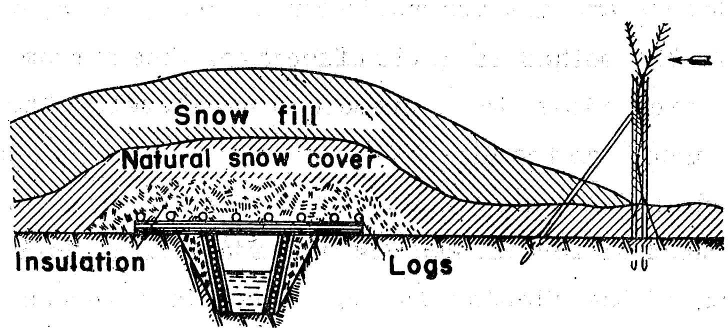


Fig. 61 – Insulation of River Beds under Bridges, Amur-Yakutsk Highway

Fig. 62 – Use of Air Layer for Insulation of a Small Ditch



Arrow indicates direction of prevailing winter winds.

Fig. 63 – Use of Snow Fences for Snow Fill on an Insulated Ditch

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