AD-A035 905　COLD REGIONS RESEARCH AND ENGINEERING LAB HANOVER N H　F/G 13/13
DAMS ON PERMAFROST (PLOTINY NA VECHNOY MERZLOTE),(U)
NOV 76　G F BIYANOV
UNCLASSIFIED　CRREL-TL-555　NL

1 OF 3
AD
A035905

CRREL

Draft Translation 555
November 1976

12

DAMS ON PERMAFROST

G.F. Biyanov

CORPS OF ENGINEERS, U.S. ARMY
COLD REGIONS RESEARCH AND ENGINEERING LABORATORY
HANOVER, NEW HAMPSHIRE

Unclassified
SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

REPORT DOCUMENTATION PAGE	**READ INSTRUCTIONS BEFORE COMPLETING FORM**
1. REPORT NUMBER Draft Translation 555	2. GOVT ACCESSION NO.
	3. RECIPIENT'S CATALOG NUMBER
4. TITLE (and Subtitle) DAMS ON PERMAFROST	5. TYPE OF REPORT & PERIOD COVERED
	6. PERFORMING ORG. REPORT NUMBER
7. AUTHOR(s) G.F. Biyanov	8. CONTRACT OR GRANT NUMBER(s)
9. PERFORMING ORGANIZATION NAME AND ADDRESS U.S. Army Cold Regions Research and Engineering Laboratory Hanover, New Hampshire	10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS
11. CONTROLLING OFFICE NAME AND ADDRESS	12. REPORT DATE November 1976
	13. NUMBER OF PAGES 234
14. MONITORING AGENCY NAME & ADDRESS (if different from Controlling Office)	15. SECURITY CLASS. (of this report)
	15a. DECLASSIFICATION/DOWNGRADING SCHEDULE

16. DISTRIBUTION STATEMENT (of this Report)

Approved for public release; distribution unlimited.

17. DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report)

18. SUPPLEMENTARY NOTES

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

DAMS
EARTH DAMS
CONCRETE STRUCTURES
COLD WEATHER CONSTRUCTION
FOUNDATIONS

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

Problems of building dams of natural materials on permafrost under severe climatic conditions in the Far North are discussed on the basis of experience in this country and abroad. Cases of damage to dams and sluiceways and its repair are described. Requirements for surveys, planning, and research in connection with building water-related structures on permafrost are presented. The book is intended for engineers specializing in water engineering matters and can be useful to students in institutions of higher learning and technical

Next

cont

schools concerned with this subject.

(14) CRREL-TL-555

(11) Nov 76

(12) 237p.

DRAFT TRANSLATION 555

(6)

ENGLISH TITLE: DAMS ON PERMAFROST

FOREIGN TITLE: (PLOTINY NA VECHNOY MERZLOTE)

AUTHOR: G.F. Biyanov

(10) Gavriil Fedorovich Biyanov

SOURCE: Moscow, Energiia, 1975, 184p.

CRREL BIBLIOGRAPHY ACCESSIONING NO.: 30-1751

Translated by Office of the Assistant Chief of Staff for Intelligence for U.S. Army Cold Regions Research and Engineering Laboratory, 1976, 234p.

NOTICE

The contents of this publication have been translated as presented in the original text. No attempt has been made to verify the accuracy of any statement contained herein. This translation is published with a minimum of copy editing and graphics preparation in order to expedite the dissemination of information. Requests for additional copies of this document should be addressed to the Defense Documentation Center, Cameron Station, Alexandria, Virginia 22314.

037100

1473

UDC 627.824.3:624.139

Problems of building dams of natural materials on permafrost under severe climatic conditions in the Far North are discussed on the basis of experience in this country and abroad. Cases of damage to dams and sluiceways and its repair are described. Requirements for surveys, planning, and research in connection with building water-related structures on permafrost are presented.

The book is intended for engineers specializing in water engineering matters and can be useful to students in institutions of higher learning and technical schools concerned with this subject.

Gavriil Fedorovich Biyanov

DAMS ON PERMAFROST
(Plotiny na vechnoy merzlote)

Sent to the printer 18 June 1975

FOREWORD

The opening of new regions in the Far North and the development of regions already in use can only be achieved if power is made available, i. e. if hydro, thermal, and nuclear power plants are built. The severe climate, the presence of permafrost, the poorly developed transportation links with industrially developed regions of the country, and other conditions determine the way in which water-related construction is done: dams built of natural materials are used in power projects and to build reservoirs for drinking water and industrial water supplies and for other economic purposes related to water.

A special characteristic of dam planning and construction is the need to take into account and predict changes in the temperature conditions of the dam itself and its permafrost foundation during the period when it is in use, and this determines the basis on which the dam is built: as a frozen dam or an unfrozen dam.

A number of earth dams have been built in the Soviet Union. The technique of stockpiling cohesive soils in winter, which was first developed and used in the construction of the dam for the Viluy Hydroelectric Power Station represents the state of the art and is responsible for the excellence of the dams of the Khantayka Hydroelectric Power Station, the Bilibino Nuclear Power Station, and others.

The construction of particular dams on permanently frozen ground has been discussed in the works of G. A. Borisov and G. Ya. Shamura, A. I. Kalabin and A. Ye. Vedernikov, S. G. Svetkovaya and V. S. Timofeychuk, and others.

In this book the author has attempted to generalize and analyze experience in the construction and use of earth dams and sluiceways on permafrost. The book reflects the experience of the author over many years in the construction of the Vilyuy Dam and a number of other dams in Yakutia. In preparing the manuscript the author also used information from the Institute of Permafrostology, Siberian Division, USSR Academy of Sciences, the Leningrad Division of the S. Ya. Zhuk Gidroproyekt Institute, institutes of the Dal'stroyproyekt, the Yakutniiproalmaz, the Viniprozoloto, the B. Ye. Vedeneyev All-Union Scientific Research Institute of Halurgy, the Krasnoyarsk Promstroyniiproyekt, and the Construction Administration of Vilyuygesstroy.

The author expresses his thanks to Cand. Tech. Sci. Yu. N. Myznikov and Eng. V. I. Makarov for their advice during their review of the manuscript. The author gives special recognition to Eng. L. N. Toropov, who took on the difficult task of editing and thereby greatly improved the book.

Please direct comments and suggestions to:

113114, Moscow, M-114,
Shlyuzovaya kab., 10,
Izd. "Energiya."

The Author

Chapter I

CLIMATIC GEOGRAPHY OF PERMAFROST REGIONS

1-1. AREAL EXTENT OF THESE CONDITIONS

Permafrost is found mainly in the polar and subpolar high latitudes, in some places extending down to a latitude of 50° or more. Permafrost also occurs in high mountains in the Temperate Zone. It is most widespread in the Northern Hemisphere, encompassing a considerable portion of the continents of Eurasia and North America, the islands of the Arctic Ocean, Greenland, and Iceland.

Data on the distribution of permafrost around the world are given in Table 1-1 [Ref. 70].

Table I-1

Distribution of Permafrost

a - Страна или территория	b Площадь, млн. км²	c - Процент от площади страны (территории)
d - СССР	10,5	47
e - Канада	5,7	52
f - Монголия	0,8	—
g - Аляска (США)	1,5	79
h - Гренландия	1,6	100
i - Антарктида	13,5	100

Key: a - Country or area, b - Area (millions of km^2), c - Percent of the area of the country (or territory), d - USSR, e - Canada, f - Mongolia, g - Alaska (USA), h - Greenland, i - Antarctica.

Extreme cooling of the surface layers of the earth in polar regions and adjacent areas results from the fact that solar heating is inadequate in these regions as compared with the middle latitudes, i. e. from a cumulative deficit of heat in the polar and subpolar regions [42, 70, and 73]. The first permafrost scientist, M. I. Sumgin, was of the opinion that permafrost is a relict phenomenon which appeared when the temperature of the earth dropped during the Ice Age.

The southern boundary of permafrost in this country runs from northwest to northeast through the Kola Peninsula, then along the tundra/forest-tundra boundary to the intersection with the Ural Mountains, then into Western Siberia across the middle reaches of the Ob' and Yenisey Rivers to the mouth of the Angara. There it turns sharply southward, curving around the mountain massifs of the Altay and Sayan, and continuing across Mongolia. The permafrost boundary again reenters the territory of the USSR on the east in the Amur River basin, encompassing a large area to the shores of the Sea of Okhotsk, and the Kamchatka Peninsula beyond (see Figure 1-1).

The southern boundary of the permafrost zone is shown only roughly on a small-scale map. Between the zone of seasonally frozen ground and the permafrost zone there is a transitional zone consisting of islands of permafrost among areas of unfrozen ground; this zone in places is as much as hundreds of kilometers wide.

In addition to the general characteristics of the permafrost zone, such as severe climatic conditions, which make economic exploitation difficult, their remoteness from the main industrial centers of the country and the poor transportation facilities between them, the relatively low population, low economic development, etc., these regions vary widely in climatic conditions, ground freezing, engineering geology, and hydrological conditions.

It is desirable for convenience to consider construction conditions in the Far North by individual zones: the European North, the Western Siberian North, the Yenisey North, the Eastern Siberian North, the Far Eastern North.

Climatic conditions in the different permafrost zones depend on their geographical position, i. e. on their closeness or remoteness with respect to seas and oceans as well as their elevation. These regions are characterized by heat and cold which paralyzes everything, freezing winds and fogs, the polar night and whiteouts, midnight sun, and other extreme natural phenomena.

The European North has a severe continental climate with winter temperatures lower as we move from west to east. The average January temperature at Syktyvkar is -15.2°C, at Ukhta -17.6°C, and at Vorkuta -20.6°C. The July temperatures are +16.6°C, +16°C, and +12.4°C, respectively.

The Western Siberian North is under the influence of cold arctic air masses from the Kara Sea. The mean January temperature at Surgut is -20.9°C and 20.1°C at Berezovo; the July temperature at those places is +17.2°C and +15.6°C, respectively, with mean annual temperatures of -2.8°C and -3.4°C.

The Yenisey North has a strongly continental climate; the winter is long and severe. The mean temperature in January over a large part of the area is about -30°C, while the mean temperature in July at Dudinka is +12°C. The mean annual temperature at Dudinka is -10.7°C, and that at Igarka is -9.3°C. A large amount of snow falls here during the winter season and strong winds are reported. Over a large part of the area snow is typically moved about by the winds and accumulates on slopes of northern exposure, where it may reach depths of 2-3 m.

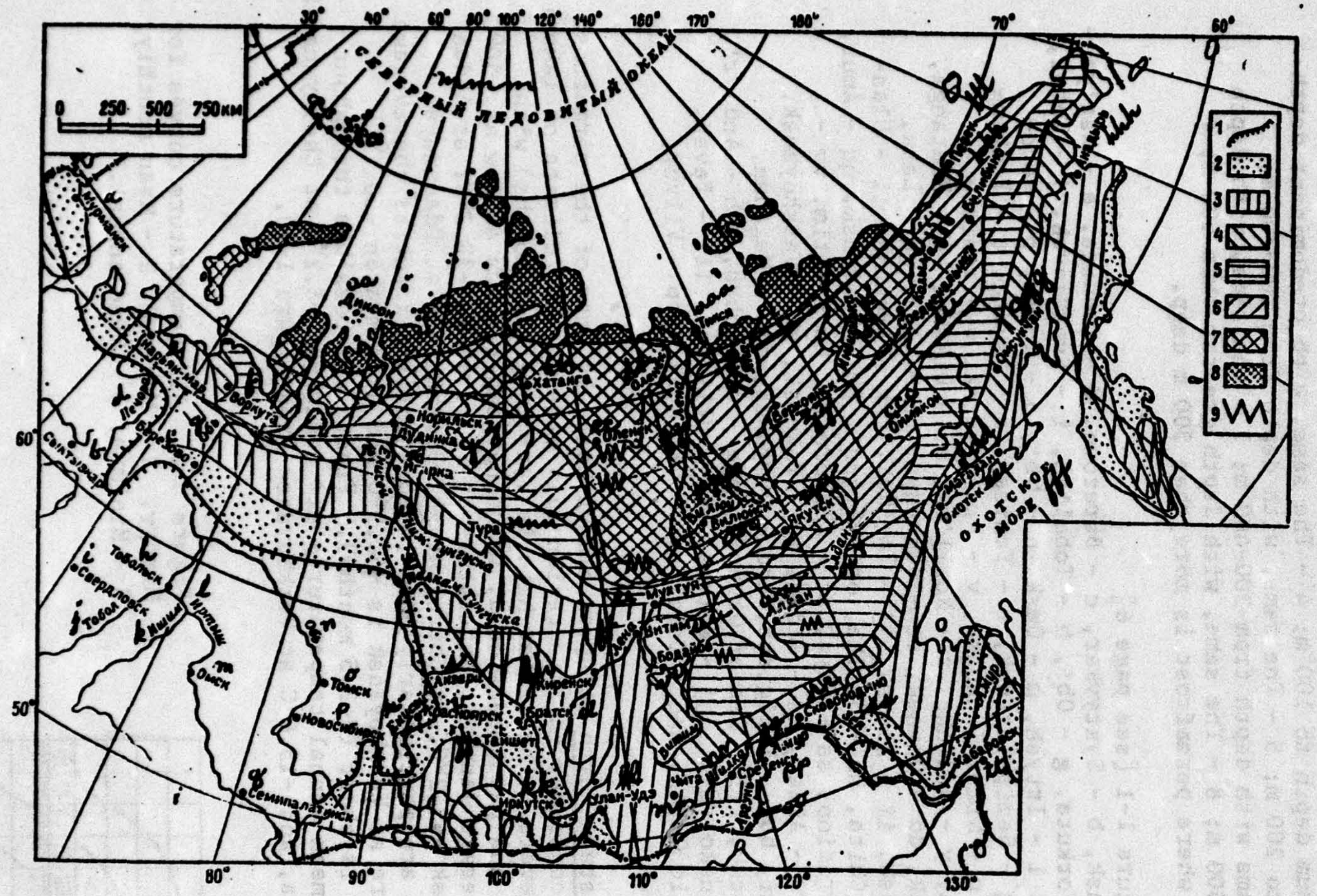

Figure 1-1. Diagrammatic map of permafrost distribution in the USSR. [Legend and key on page 7].

[Legend for Figure 1-1 on page 6]
1 - Southern boundary or permafrost; 2 - Zone of separate islands of permafrost with maximum depth to 25 m; 3 - Zone of permafrost with maximum depth to 100 m; 4 - The same, with predominant depth from 100 to 200 m; 5 - The same, with depth from 200-300 m; 6 - The same with depth from 300-400 m; 7 - The same, with depth from 400-500 m; 8 - The same, with depth greater than 500 m; 9 - Areas where permafrost is more than 500 m deep.

Key to Figure 1-1 [see page 6]
a - Murmansk, b - Syktyvkar, c - Berezovo, d - Pechora, e - Nar'yan-Mar, f - Vorkuta, g - Ob', h - Tobol'sk, i - Sverdlovsk, j - Tobol, k - Ishim, l - Irtysh, m - Omsk, n - Ob', o - Tomsk, p - Novosibirsk, q - Semipalatinsk, r - Yenisey, s - Angara, t - Podkam-Tunguska, u - Nizh. Tunguska, v - Krasnoyarsk, w - Yenisey, x - Igarka, y - Dudinka, z - Noril'sk, aa - Dikson, bb - Khatanga, cc - Olenek, dd - Olenek, ee - Mukhtuya, ff - Lena, gg - Lena, hh - Kirensk, ii - Bratsk, jj - Tayshet, kk - Irkutsk, ll - Ulan-Ude, mm - Chita, nn - Shilka, oo - Argun', pp - Sretensk, qq - Amur, rr - Skovorodino, ss - Zeya, tt - Khabarovsk, uu - Vitim, vv - Bodaybo, ww - Aldan, xx - Aldan, yy - Yakutsk, zz - Verkhoyansk, aaa - Tiksi, bbb - Indigirka, ccc - Oymyakon, ddd - Magadan, eee - Okhotsk, fff - Sea of Okhotsk, ggg - Omsukchan, hhh - Andayr', iii - Srednekolymsk, jjj - Kolyma, kkk - Bilibino, lll - Pevek, mmm - Arctic Ocean, nnn - Tura, ooo - Vilyuysk, ppp - Vilyuy, qqq - Yana.

The Far Eastern North occupies almost half the area of the permafrost zone. Climatic conditions in this region vary widely. The climate of Yakutia is particularly severe. The winter here is very long (7-8 months) with cloudless weather and persistent cold, reaching temperatures as low as -60°C or -70°C. The average January temperature at Vilyuysk is -38.2°C, at Aldan -27.6°C, at Yakutsk -43.2°C, at Chernyshevsk -33.2°C, and at Udachnaya -42.0°C. Winters are particularly cold in northeastern Yakutia; the average January temperature at Verkhoyansk is -48.9°C and at Oymyakon -50.1°C. Summer in Yakutia is short (2-2.5 months). Frosts are possible throughout the summer. The mean annual temperatures are negative: -8.2°C at Chernyshevsk, -12°C at Udachnaya, and -15.6°C at Verkhoyansk (See Figure 1-2).

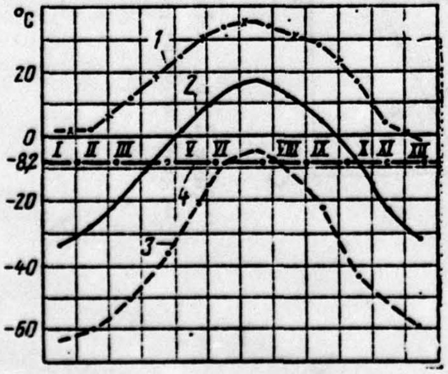

Figure 1-2. Air temperature curves for Mirnyy. 1 - Maximum; 2 - Mean monthly; 3 - Minimum; 4 - Mean annual.

The Sea of Okhostsk and the Bering Sea have a significant influence on the climate of Kamchatka and Magadan. Winter temperatures drop with distance from the coast toward the interior, while summer temperatures increase as we move from the coast to the center of the region. The average temperature in January at Okhostsk is -24.5°C and that at Nikolayevsk-on-Amur is -24°C. On the Chukotskiy Peninsula temperatures decline from -11°C along the coast to -34°C in the center.

Air temperatures in the various permafrost regions of the USSR are shown in Table 1-2.

Table 1-2

Air Temperatures in the Permafrost Regions of the USSR

a - Районы	b - Температура, °C		
	c - Среднегодовая	d - Абсолютный минимум	e - Абсолютный максимум
h - Европейский Север:	v w	v w	v w
i - Архангельская обл.	От +0,8 до −7,6	От −32 до −54	От 24 до 34
j - Коми АССР	От +0,3 до −6,3	От −45 до −55	От 31 до 34
k - Север Западной Сибири:			
l - Тюменская обл.	От −0,4 до −9,9	От −47 до −57	От 22 до 36
m - Енисейский Север	От −2,2 до −15,6	От −52 до −69	От 18 до 37
n - Север Восточной Сибири:			
o - Иркутская обл.	От −2,6 до −8,6	От −56 до −60	От 33 до 37
p - Читинская обл.	От −7,2 до −7,3	От −58 до −60	От 35 до 36
q - Дальневосточный Север:			
r - Якутская АССР	От −5,5 до −15,6	От −49 до −71	От 21 до 38
s - Магаданская обл.	От −3,7 до −13,7	От −36 до −67	От 22 до 35
t - Сахалинская обл.	От −2,1 до −2,3	От −43 до −49	33
u - Камчатская обл.	От −0,3 до −10,3	От −34 до −52	От 24 до 33

Key: a - Regions, b - Temperature, °C, c - Mean annual, d - bsolute minimum, e - bsolute maximum, h - European North, i - Arkhangel'skaya Oblast, j - Komi ASSR, k - Western Siberian North:, l - Tyumen Oblast, m - Yenisey North, n - Eastern Siberian North:, o - Irkutsk Oblast, p - Chita Oblast, q - Far Eastern North:, r - Yakut ASSR, s - Magadan Oblast, t - Sakhalin Oblast, u - Kamchatka Oblast, v - From, w - to

1-2. ENGINEERING GEOLOGY

Conditions related to engineering geology in the permafrost regions vary considerably, but certain common characteristics can be detected in different zones.

Conditions related to engineering geology are described below for the permafrost areas within the USSR where man has been most active.

The <u>European North</u> has deep Quaternary deposits more than 120-130 m thick. In some places, however, the bedrock is exposed or is covered with a thin layer of Quaternary deposits. Mantle deposits, mainly diluvial, are everywhere, consisting mainly of silty loams with occasional inclusions shingle and gravel. The mantle diluvial deposits are 0.5-5.0 m thick on slopes. The mantle loams have a high content of silt particles (up to 90%) and a high moisture content (30% or more). It is very difficult to excavate the silty materials with their high moisture content without first drying them. In addition they are subject to frost heaving.

Under the mantle loams are Postglacial lake-bog deposits, moraine deposits of the Second Glaciation, etc.

The Western Siberian North is a lowland, with Quaternary deposits almost everywhere. These deposits reach a thickness of 100-200 m. Floodplains have developed in the valleys of all rivers. The deposits there are quite varied, consisting of sands of mixed grain size, loamy sands, and silty loams. These deposits are ordinarily 7-8 m thick.

In the Yenisey North thick strata of bedrock predominate, consisting mainly of limestones, dolomites, and marls. In the plain near the Yenisey they are covered with a layer of Quaternary deposits up to 200 m thick (town of Dudinka). These consist of glacial (morainic and glacial-lacustrine), marine, lake-bog, and other deposits.

In the Far Eastern North geological conditions are extremely varied and complex. The western region of Yakutia lies geographically within the Siberian platform, while the central part is in the Lena-Vilyuy depression, which is filled with ancient sedimentary deposits. East of the Lena River is the Verkhoyano-Kolyma region of mountains and lowlands with a wide variety of geomorphological regions, with areas ranging from mountain systems to depressions and a broad maritime lowland along the coast of the Arctic Ocean consisting of alluvial deposits with a total thickness up to 100 m.

1-3. FROZEN GROUND (GEOCRYOLOGICAL) CONDITIONS

Frozen ground is defined as that with a negative or zero temperature and which contains ice.

From the viewpoint of age, frozen ground is subdivided into seasonally frozen ground and permanently frozen ground (frozen for many years). Permanently frozen ground is that which remains in the frozen state for a long time (from several years to tens of thousands of years). According to Construction Standard and Specification II-B.6-66, this period is defined as exceeding three years.

When its temperature regime changes, permanently frozen ground obviously becomes structurally unstable ground. Construction on such ground without taking special precautions will most certainly result in intolerable deformation of the structures. At the same time, if the properties of such ground are taken into account correctly, a wide variety of structures can be built on it successfully.

Above the permanently frozen layer there is a layer of material which freezes and thaws every year with the seasons; this is called the active layer. Seasonal freezing is defined as the freezing of thawed ground which has a mean annual temperature above 0°C. The seasonally frozen layer is underlain by unfrozen ground. Seasonal thawing of the ground is understood to mean the thawing of frozen ground having a mean annual temperature of 0°C. The layer of seasonal thawing is underlain by a layer of permanently frozen ground.

There may be intermediate forms between permanently frozen ground and seasonally frozen ground. Thus, seasonally frozen ground may not thaw during the summer and thus persist several years. Such layers of frozen ground are called <u>pereletki</u>.

The development of seasonally freezing and seasonally thawing layers and their depth depend on a variety of natural factors, but most particularly on the lithological composition and origin of the surface deposits, the characteristics of the surface soil, the ground water, the nature of the topography and vegetation, the microclimate of individual areas, etc.

Frozen ground is subdivided on the basis of distribution into continuous and discontinuous frozen ground.

Islands of frozen ground are found along the southern boundary of the permafrost zone as islands within a general expanse of ground which thaws.

Discontinuous frozen ground with islands and tracts of ground subject to thawing within a general expanse of permafrost is found much farther north than permafrost islands. Ground subject to thawing is found mainly in river valleys, on water divides, and on their slopes.

Continuously distributed frozen ground predominates in the central and northern parts of the permafrost zone. But zones subject to thawing exist here also because of the influence of warming factors (such as surface and underground water) and are found mainly as perforating or enclosed thawed zones (<u>taliki</u>) under rivers and lakes.

The thickness of the permafrost layer varies depending on numerous factors, chief among which are the duration of the cold season and the depth of cooling of the earth's crust, as well as on the properties and state of the soil materials. The latter in turn depend on latitude and elevation.

Fig. 1-1 is a diagrammatic map of the distribution of permafrost in the USSR. It must be pointed out that the sourthern boundary of the permafrost zone shown on this map is not accurate but rather diagrammatic, since frozen and thawed areas "interpenetrate" along a broad band in the region of this boundary. Thawed and frozen areas, both at the surface and at some depth, can be found along the southern boundary of the permafrost zone.

A number of authors have now compiled map-diagrams of the temperature and frozen-ground geological regions of the USSR (V. A. Kudryavtsev, A. I. Popov, I. Ya. Baranov, and P. F. Shvetsov).

The distribution of frozen ground is defined on the basis of freezing conditions in a given area, the latter in turn being based on equivalent climatic geological, and geographic situations.

The geocryological zoning of a territory means its subdivision into zones wherein the characteristics on which the zoning is based, both qualitatively and quantitatively, exist in strictly defined combinations which differ from the characteristics of other zones [3].

The southern boundary of the permafrost zone has been drawn to a large extent on the basis of previously established requirements for the existence of permafrost. Actual data which could be used to establish its location accurately are very few, even within the Soviet Union.

In locating the southern permafrost boundary on the map some authors adopt an isothermal line corresponding to the mean long-term depth of seasonal freezing of the ground as observed under typical geographical conditions. Other authors, and V. A. Kudryavtsev in particular, suggest that this boundary be the average line corresponding to zero ground temperature over a long period of time at the base of the layer with annual temperature fluctuations [42]. Such a boundary can also be run along the southernmost islands of permafrost. The latter boundary, of course, will fall significantly farther south than the preceding one, since it will include the region of insular permafrost distribution and permafrost islands are found within a zone where mean annula temperatures are positive.

In our opinion the latter approach should be adopted as most useful from the practical point of view. At the same time it should be stated that whatever basis is used for drawing the southern boundary of permafrost, maps containing such boundaries can be considered only as roughly indicative.

Regardless of the availability of such maps showing zonal boundaries, successful construction and use of structures on permafrost require a detailed study of the engineering geology, the hydrogeology, and freezing conditions at the construction site.

Fig. 1-3 shows the vertical temperature field in permafrost based on data developed by N. S. Bogdanov [24], and N. A. Tystovich [100].

The ground temperature varies from positive to negative, depending on the time of year, down to the depth of seasonal thawing (h_c). Below h_c the ground temperature remains always negative. Starting at a certain depth, the ground temperature rises to 0^oC or above. The distance from the upper to the lower boundary of the frozen layer is defined as its thickness H_C.

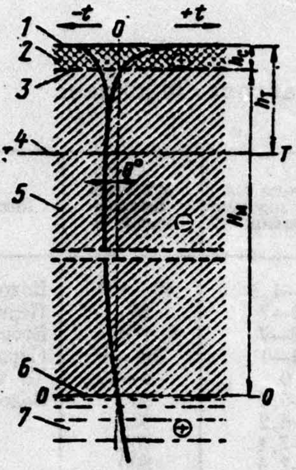

Figure 1-3. Temperature field in permafrost.
1 - Surface of the earth; 2 - Seasonally thawing layer; 3 - Lower surface of seasonally thawing layer; 4 - Level at which annual temperature fluctuations do not appear; 5 - Permafrost layer; 6 - Lower boundary of permafrost; 7 - Unfrozen zone of the earth's crust; 0° - Ground temperature at the zero amplitude level.

Some data on permafrost thickness at various locations in the USSR are given in Table 1-3 [70].

In Fig. 1-3 the plane TT corresponds to the depth h_T at which the amplitude of annual temperature fluctuations is practically zero.

The depth of seasonal freezing and thawing of the ground varies in different regions, and data on this are given in Table I-4 for certain characteristic locations.

As was mentioned earlier, frozen ground is subdivided according to age into seasonally frozen ground (measured in months) and permanently frozen ground (measured in years, hundreds of years, and thousands of years). Between these basic types there are intermediate forms of frozen ground, mutual transitions, etc.

Study of seasonal freezing and thawing of the ground is very important from the viewpoint of engineering geology. Assessment of the engineering geology of an area is impossible without prediction of change in the freezing situation following construction. The prediction involves above all a description of seasonal thawing and freezing under the altered conditions, a study of the possibility of the development of pereletki and the formation of new frozen areas in unfrozen zones, the thawing of permafrost, the development of thermokarst, etc.

Table 1-3

Permafrost Thickness

a – Пункты	b – Температура у подошвы с годовыми колебаниями, °C	c – Мощность мерзлой толщи, м	d – Место наблюдения
e – Воркута	—1—1,5	40—130	Водораздел – n
f – Якутск	—3—7	230	Первая терраса – o
g – Верхоянск	—6—7	180	Вторая терраса – p
h – Игарка	0—1	20—50	Первая терраса – o
i – Чита	0	8—20	То же – o
j – Мирный	—6,0	300	Водораздел – n
k – Чернышевский	—6,2	350	•
l – Айхал	—7,4	450	•
m – Удачная	—7,6	500	•

Key: a – Location, b – Temperature at the bottom of the layer with annual fluctuations, °C, c – Thickness of the frozen layer, m, d – Observation site, e – Vorkuta, f – Yakutsk, g – Verkhoyansk, h – Igarka, i – Chita, j – Mirnyy, k – Chernyshevskiy, l – Aykhal, m – Udachnaya, n – Water divide, o – First terrace, p – Second terrace.

Table 1-4

Depth of seasonal freezing

a – Пункты	b – Среднегодовая температура воздуха, °C	c Глубина сезонного промерзания и оттаивания, м
d – Москва	7	0,4
e – Омск	3,6	2,2
f – Свердловск	5,5	1,1
g – Иркутск	2,5	2,2
h – Чита	1,3	3,8
i – Якутск	—8,6	1,9
j – Мирный	—8,2	1,8
k – Верхоянск	—9	1,6
l – Анадырь	—5	0,7

Key: a – Locations, b – Mean annual air temperature, °C, c – Depth of seasonal freezing and thawing, m, d – Moscow, e – Omsk, f – Sverdlovsk, g – Irkutsk, h – Chita, i – Yakutsk, j – Mirnyy, k – Verkhoyansk, l – Anadyr'.

The formation of seasonally freezing and seasonally thawing layers is influenced by a variety of natural factors: the lithology of the ground, the characteristics of the soil cover, the presence of ground water and surface water, and the nature of the topography (exposure, vegetation, the microclimate of individual areas, etc.).

The depth of seasonal freezing and thawing changes when there is a change in the overall temperature conditions.

A snow cover protects the ground from freezing by altering the mean annual temperature of the ground and the annual amplitude of temperature changes at the ground surface. When the snow cover is eliminated, seasonal freezing is deeper.

Vegetation influences the seasonal freezing and thawing of the ground by changing heat-exchange conditions at the surface.

The influence of slope exposure on the depth of seasonal freezing and thawing results from the difference in average annual temperatures of the ground on slopes of southern and northern exposure. This difference is due chiefly to the difference in maximum temperatures during the summer at the surface. The redistribution of the snow cover where the snow is continuously blown by the wind also has an effect.

Waterlogging affects seasonal freezing and thawing of the ground mainly because in waterlogged areas the moisture content of the ground is equal to or close to the absolute moisture capacity. When water-saturated ground freezes, the phase transition of the water evolves more heat.

The productive activity of man has a very large effect on temperature conditions in the ground and thus on its seasonal freezing and thawing. Removal of vegetation, cutting of trees, clearing snow from an area, the compaction and storage of snow, the building of artificial reservoirs and drainage systems, the darkening of the snow cover in settled areas and areas close to industrial facilities, paving with asphalt and concrete, cutting and piling up the earth, building and shading the ground, etc., all alter significantly the conditions for heat exchange at the surface and result in a change of the overall temperature regime, and particularly the depth of seasonal freezing and thawing of the ground.

1-4. HYDROLOGY OF THE RIVERS

The presence of permafrost and severe climatic conditions influence the geographic factors of runoff, the supply and distribution of river water during the year, as well as winter conditions in the rivers before and after dams are built.

Because we usually do not know enough about the rivers in the regions in question, in water-related construction special attention should be given particularly to studying the character of the runoff, to determining the rates of runoff, i. e. the annual volume of runoff and its variations with time, the amount of water discharged and the levels during low-water and high-water years. In doing this, of course, particular attention should be given to determining maximum flood discharge, which will determine the size of sluiceways.

There have been cases where water-related structures in the Far North have been destroyed as a result of improper determination of flood discharge.

The sluiceway of a temporary dam on the Irelyakh River was designed to pass, with a frequency of 10%, a flood discharge of 167 m^3/sec, but during the first year of its use the flood discharge reached 193 m^3/sec. Since the sluiceway could not handle this discharge and water spilling over the crest threatened to erode the dam, the sluiceway at the right end was enlarged with explosives.

Because of the presence of trees and a layer of moss and peat, rivers in the permafrost zone do not carry much sediment. Therefore little attention is ordinarily given to siltation of the impoundment when dams are built in the Far North.

The behavior of rivers in this zone during winter also has a significant influence on the construction and use of dams and reservoirs. Ice conditions, the formation of frazil ice, and the movement of ice through a channel narrowed by cofferdams are very important factors. Cases of ice gorges threatening construction sites have been observed.

Hydrological conditions of rivers in the permafrost zone differ significantly because of variation in natural conditions. But all rivers in this zone have spring floods, high water in the summer and after rains when the spring floods have subsided, and a long winter period of low water, when most small rivers freeze up. The seasonal distribution of river flow is shown in Table 1-5 as percents of the annual flow.

Hydrological conditions in the rivers of the permafrost zone are extremely variable during the year because of the special nature of their water supply. River flow is very uneven during the year. For instance, maximum and minimum flow in the Vilyuy River observed during the construction of the Vilyuy Hydroelectric Power Station was 12,700 m^3/sec and 1.5 m^3/sec, respectively. Maximum discharge was 8500 times greater than minimum discharge within a year! By way of comparison maximum/minimum discharge ratios for other rivers of the Temperate Zone are: Volga, 40-60; Svir' and Niva, 2-6; and Angara, 3.

The uneven distribution of flow in the Vilyuy River, in particular, is seen in the following figures: 84% of the runoff occurs in spring-summer, 14% in summer-fall, and only 2% of the annual total during the winter (see Figure 1-4).

Table 1-5

Seasonal distribution of river flow

a - Река	b - Пункт наблюдений	c - Распределение стока, %		
		d- Весна (май—июнь)	Лето — осень (июль—октябрь) e	Зима (ноябрь—апрель) f -
g- Оленек	с. Оленек p	60,5	39,2	0,3
h- Вилюй	с. Сунтары q	66,4	31,9	1,7
i- Яна	г. Верхоянск. r	33,9	65,8	0,3
j- Колыма	п. Среднекан s	48,2	50,0	1,8
k- Даддын	п. Новый t	70,3	29,7	0
l- Чона	с. Туой-Хая u	81,1	16,1	2,8
m- Витим	г. Бодайбо v	33,1	62,9	4,0
n- Алдан	г. Томмот w	52,9	37,7	9,4
o- Адыча	с. Устье x	43,1	56,7	0,2

Key: a - River, b - Observation site, c - Distribution of flow, percent, d - Spring (May-June), e - Summer-fall (July-October), f - Winter (November-April), g - Olenek, h - Vilyuy, i - Yana, j - Kolyma, k - Daldyn, l - Chona, m - Vitim, n - Aldan, O - Adycha, p - Olenek, q - Suntary, r - Verkhoyansk, s - Srednekan, t - Novyy, u - Tuoy-Khaya, v - Bodaybo, w - Tommot, x - Ust'ye.

The discharge of all rivers of Yakutia and Magadan Oblast drops sharply in the winter. Most of them freeze up completely or individual disconnected pools remain in deep stretches of the river.

The rivers are fed in winter by ground water in taliki (unfrozen areas), but these often freeze up and the supply of water to the river ceases.

During the summer there are several peaks due to rains as well as to the so-called "black waters", which enter the rivers when rain causes significant thawing of the permafrost.

Spring flooding is heavy in years when there has been abundant snowfall and snow melting coincides with spring rains. In the case of the Vilyuy River, for instance, spring flood discharge may vary from 2400 to 13,000 m^3/sec, i. e. 5.5 times.

Observational data on discharge and water level are not always adequate for the rivers of the permafrost zone. In these cases the mean long-term flow is usually estimated on the basis of maps showing mean rates of runoff. But this method cannot always be accurate. The analogy method is difficult to use because of the difficulty in choosing a river analogue. Here it is necessary to make a thorough analysis of conditions of physical geography in the basins of both rivers.

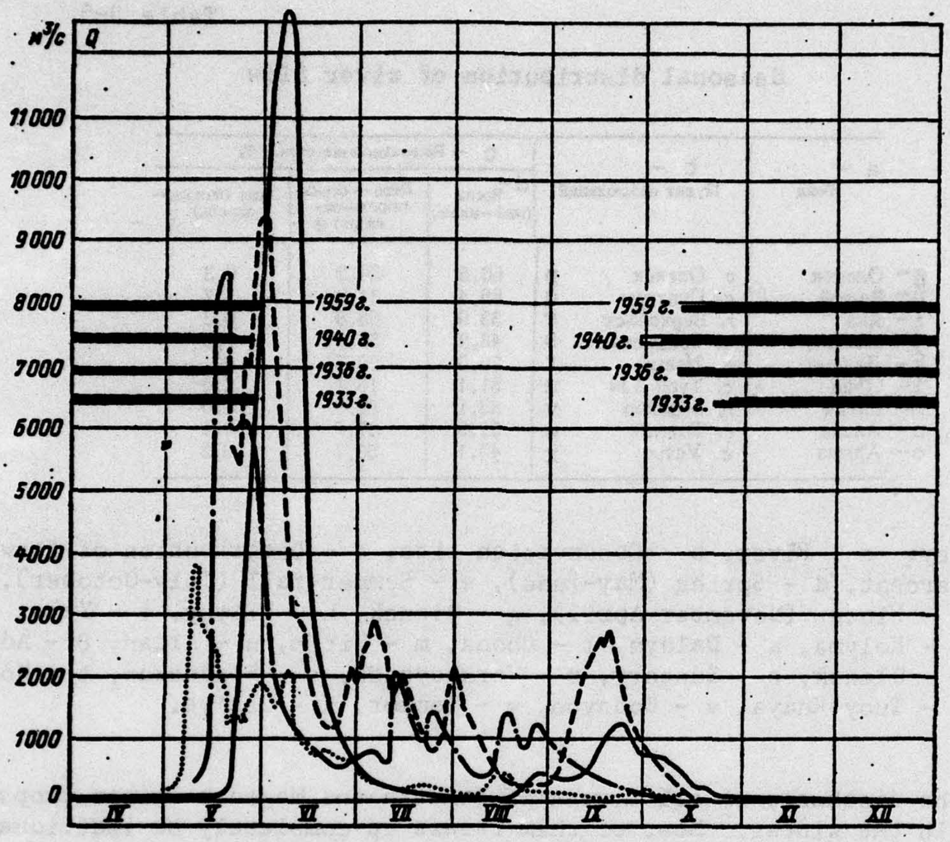

Fig. 1-4. Hydrograph of the Vilyuy River.
... - a low-water year (1936); ___.___ - a year close to average (1940); _____ - a high-water year (1959); - - - - the same (1933).

The thermal regime of reservoirs and rivers in the Far North is particularly important. It depends on heat exchange conditions between the water and the environment, particularly exchange with the atmosphere and with the bed of the reservoir, where permafrost is a critical factor.

The vertical distribution of temperature in the water is essential to a proper understanding of phenomena taking place during the winter in rivers and other water bodies with impeded flow. In a reservoir, where the rate of flow is very low, the basic factor governing the vertical distribution of temperatures is density.

Water, of course, has maximum density at 4°C, and it approaches this temperature at great depths. But the water temperature is lower near the bottom due to the influence of permafrost there.

The ice cover on particular rivers reaches a thickness of 200 cm. River opening varies depending on the rate of melting of the snow and ice. Conditions are especially unfavorable for ice movement on rivers which flow from south to north or east and west.

In the first case, ice moving from the south is blocked by the edge of ice which is still strong, and this, of course, causes ice gorges. The same thing occurs where river channels are locally narrow. Very high water levels accompany such ice gorges. For instance in 1966 the Lena River rose 17 m at the town of Lensk. During construction of the Mamakan Hydroelectric Power Station, the Mamakan River was clear of ice earlier than the Vitim River, into which it empties. The result was an ice gorge and the water level in the Mamakan River rose above the top of the protective cofferdams, flooding the construction site.

In the case of rivers which flow east and west, simultaneously heavy melting of snow in the drainage basin causes heavy movement of ice along the entire length of the river and a violent rise in water level. On the Vilyuy River, for instance, water levels on particular days rise 4-5 m, and the overall flood level reaches 16 m.

Problems of ice passage are particularly important during periods of construction because of the need to build cofferdams and confine the river. Hence water velocity in parts of the river bed confined by cofferdams is significantly greater than under natural conditions for in the approaches to the cofferdams, so that there is no special problem in providing for smooth entry. An interesting example of ice passage through a construction bypass is seen in the construction of the dam for the Vilyuy Hydroelectric Power Station. A large waterfall was built at the entry to the channel. Even thick ice floes were broken up here and passed easily through the channel (see Fig. 1-5). When a reservoir has been completely filled, experience has demonstrated that the ice ordinarily melts in the reservoir.

1-5. GROUND WATER CONDITIONS

Ground water in permafrost regions is of three principal types: suprapermafrost, intrapermafrost, and subpermafrost.

Ground water occurs mainly in alluvial, diluvial, glacial, and other Quaternary deposits, and its presence depends on the depth of seasonal thawing. The permafrost in this case represents the impermeable substrate. The principal source of the suprapermafrost water is atmospheric precipitation. This water tends to cause bog conditions in flat areas because of the impeded runoff and the presence of the impermeable stratum below.

On steep slopes where the soil materials are sufficiently permeable, the suprapermafrost water drains into low areas. Such water sometimes comes out on the surface.

Figure 1-5. Ice on the Vilyuy River passing through a bypass in 1965.

During the winter the suprapermafrost ground water usually freezes up completely. But in certain areas, when the surface layer freezes from above and the permafrost encroaches from below, the cross-section of the talik is reduced. This causes the pressure of the ground water to build up on slopes, and in the valleys of rivers and creeks, to be released as naleds. River waters may also sometimes be the source of supply for naleds. Naleds are subdivided into surface and subsurface types, and the latter may occur in the form of fossil ice. Naleds are also subdivided into seasonal and long-term types.

Other types of ground water are found in the floodplains of river valleys. These are fed by the suprapermafrost water draining from the slopes and by atmospheric precipitation. The suprapermafrost water in river valleys occurs mainly in recent alluvial deposits.

Permanent taliki develop on floodplains and the terraces above as a result of heat transfer by river and suprapermafrost waters.

The size of the talik aquifers in the valleys of rivers which freeze up depends on the season of the year, the geographical location, geological structure and area of the basin, the shape of the valley bottom, the thickness and composition of the permeable deposits, water-supply conditions, and the duration of circulation of the suprapermafrost flow [68].

Intrapermafrost water occurs mainly in river valleys. The intrapermafrost water of penetrating taliki occurs both in Quaternary deposits and in fissures in bedrock of various composition and age. Intrapermafrost talik quifers are important because they provide a link between the suprapermafrost and the subpermafrost water.

The subpermafrost ground water includes all the underground water that occurs at various depths below the permafrost. The depth of the subpermafrost water usually depends on topography and the thickness of the permafrost. The subpermafrost water is under pressure almost everywhere, and the impermeable layer above is usually permafrost.

In many cases subpermafrost water is a reliable source of water supply. The possibility of using subpermafrost water for water supply depends on the rate of discharge and the chemical composition.

Chapter 2

PERMAFROST AS A FOUNDATION FOR DAMS[1]

2.1 THE ENGINEERING PHYSICS OF FROZEN GROUND

The presence of ice in frozen ground. The critically important characteristic of permafrost is that it contains ice. Hence the composition, structure, and properties of permanently frozen ground depend, on the one hand, on the composition, and on the other, on the quantity and nature of the distribution of ice in it.

Permanently frozen ground is subdivided into epigenetic and syngenetic types on the basis of freezing history [42]. Epigenetic permafrost includes those materials which froze after they were developed. These are mainly pre-Quaternary deposits. Syngenetic permafrost includes those materials which froze simultaneously with the formation of Quaternary sediments. Thus only Quaternary sediments can be syngenetic permafrost, and these sediments are most widespread on the low plains of the North.

Syngenetic frozen strata are very important in engineering permafrostology, since they are located in the upper part of the earth's crust, which is the site of most engineering activity, and it is here that we find such cryogenic processes of engineering importance as frost fissuring, the formation of polygonal veins of ice, solifluction, thermokarst, etc.

Texture of frozen ground. The cryogenic texture of permafrost depends on the distribution of the texture-forming ice, which can be present as relatively small lenses, sheets, layers, grains, and other forms of inclusions.

Several basic cryogenic textures are distinguished in unconsolidated deposits, including: massive, reticulated, stratified, etc. The massive texture is typical of sandy materials, where the ice serves as a sort of cement, but without visible inclusions. Both reticulated and stratified textures are characteristic of dispersed materials and peat. In clayey soil materials (clays, loams, and loamy sands), the ice content depends on the concentration of ice inclusions in the form of individual lenses and layers. In the reticulated texture, systems of cross-oriented and intersecting ice inclusions

---

[1] The properties of permafrost are discussed briefly in this chapter. For more information see, for example, References 42, 73, and 102.

create an ice network. In the layered texture the ice is present in the form of separate layers and sheets. In this case the layers may be horizontal, slanted, or wavy.

A crusty texture is characteristic of gravelly-pebbly materials with sand and loamy sand fillers and of loams with a gravelly-pebbly filler. In this case the ice forms crusts around the larger fragmentary material, while in the filler the ice is a cement. Sometimes the ice in such materials may be present in the form of small diffuse accumulations.

Frozen ground with massive texture shows greater strength, other conditions being equal. The strength characteristics of frozen ground with stratified texture depend on the size and number of the ice inclusions and on the nature of their interstratification with the frozen mineral layers. Here the larger the ice inclusions and the greater their thickness, the smaller the continuous load strength of the ground. Ground with a reticulated texture occupies an intermediate position as far as strength is concerned between ground with massive and stratified textures.

In consolidated and semiconsolidated ground, ice inclusions are present as filler in fissures and cavities which might have existed before freezing started and were filled with water. Such ice is of epigenetic origin.

The upper zone of bedrock to a depth of several tens of meters is usually more broken due to weathering processes that took place in geological epochs prior to freezing. This zone also may have been subject to so-called frost weathering as a result of multiple freezing and thawing. The ice content of this zone is usually highest at the surface, gradually decreasing with depth.

In solving engineering problems one must take into account not only the quantitative ratio of ice to mineral particles in the frozen ground but also the form and size of the ice inclusions (crystals, lenses, layers, and veins). They may include ice naleds buried by river alluvium and lake-bog deposits, frozen water bodies, river and lake ice, ice formed as a result of the freezing of aquifers, etc. The literature contains descriptions of syngenetic and epigenetic "refrozen" vein ice or polygonal vein ice, the formation of which results from the frost-fissuring of massive material [42, 70].

Fossil ice may vary in form and size, reaching thicknesses ranging from several tens of centimeters to several meters. During construction of a dam on the Irelyakh River, when a tooth trench was being extended into alluvial deposits of the floodplain on the left bank of the river, a lens of fossil ice was discovered which was up to 1.5 m thick and extended 12 meters along the axis of the dam.

An even larger lens of fossil ice (8 m) was found at the construction site of a dam on Byystakh Creek, which is a tributary of Daldyn River in the Yakut ASSR.

Permafrost is subdivided according to ice content into very icy, icy, and slightly icy. Very icy ground is that which contains more than 50% ice by volume; slightly icy ground contains less than 25% ice by volume, and icy ground contains from 25% to 50% ice by volume. When it thaws, very icy ground has a very low bearing capacity and is highly compressible [102].

Permafrost is subdivided according to physical state into hard permafrost and plastic permafrost. Hard permafrost is that which is tightly cemented by ice, shows relatively brittle fracturing properties, and is practically incompressible under load.

Plastic permafrost is that which is cemented by ice but, because of its high content of unfrozen water possesses viscous properties and is capable of compression under load. Plastic frozen ground includes all high-temperature finely dispersed frozen ground where the negative temperature does not drop below -0.3°C in the case of silty sands, -0.6°C in the case of loamy sands, -1.0°C in the case of loams, and -1.5°C in the case of clays.

The existence and development of frozen ground must be considered, not as constant and unchanging, but as in motion, interacting with all the phenomena of nature.

A change in temperature conditions in frozen ground leads to qualitative changes in it, to changes in its physical and mechanical properties. Hence, it is important to predict the change in frozen ground with the passage of time depending on environmental conditions, and to study these conditions, the pattern and dynamics of their change. The composition and properties of freezing and thawing monolithic consolidated materials are less variable than in the case of dispersed materials.

Dispersed frozen ground is a multiphase and multicomponent system in which the following constituents are distinguished: solid, consisting of the mineral skeleton and ice; liquid, consisting of unfrozen water with the salts dissolved in it; and gaseous (gases and water vapor), which are present in the pores of the ground in the free state [98].

<u>Migration</u> of water contained in the ground during the processes of freezing and thawing leads under certain conditions to the formation of ice layers in finely dispersed materials or to the formation of frost structures and textures, to changes in volume and deformation of the ground, and to heaving.

Migration of moisture associated with a change in texture and structure, in ice content and in moisture content sharply alters the physical and mechanical properties of the ground.

The moisture content of thawing ground depends to a large extent on its actual bedding conditions. If free water cannot drain away from the thawing ground, all the water present when it froze is retained. If free drainage is possible, then the amount of moisture in the ground is determined by its water-holding capacity. In this case the final moisture content of the ground when it thaws may be less than its original moisture content prior to freezing.

Heaving of the ground is the uneven increase in volume associated with freezing. This occurs both as a result of the increased volume of water present in the ground and as a result of freezing of new water entering the ground in question from the outside, and water moving to the freezing front, i.e. with seepage of moisture from elsewhere [42].

Heaving of dispersed soil materials when frozen occurs throughout the frozen ground region, but it varies due to different freezing conditions, composition of the ground, and moisture of the ground.

Frost-fissuring. The distribution of temperature with depth in the permafrost region is shown in Fig. 2-1. We see there that the surface layers of the ground are subject to the influence of temperature fluctuations ranging from a minimum value in the winter (t_{min}) to maximum in summer (t_{max}).

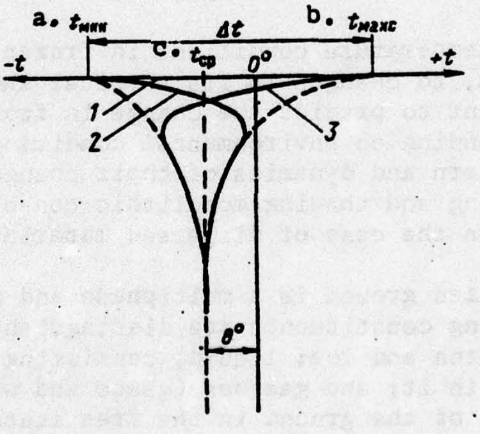

Fig. 2-1. Curves showing temperature variation in the ground. 1 - Surface of the ground; 2 - Distribution of temperature in the upper layer of frozen ground in the winter; 3 - Distribution of temperatures in the upper layer of frozen ground in the summer; 0° - temperature of the ground at the level of annual fluctuations; t_{min} - minimum winter temperatures at the surface of the ground; t_{max} - summer temperatures at the surface of the ground.
Key: a - t_{min}; b - t_{max}; c - t_{av}.

Volume changes occur in unfrozen and frozen ground with cooling and warming, with freezing and thawing. These changes usually are uneven. Thus, during cooling or freezing the surface layers of the ground are subject to tensile stresses while the interior layers experience compressive stresses.

In the winter the tensile stresses may exceed the instantaneous tensile strength, which results in cracking of the ground surface, i.e. the appearance of frost fissuring (see Figure 2-2).

Figure 2-2. Frost fissuring of rocks.

Frost weathering. Physical weathering of the ground is more intense in permafrost regions than in southern regions. This is promoted by the greater amplitude of temperature fluctuations which cause internal stresses in the ground and a mechanical wedging action by films of water in tiny fissures when they freeze. In spring and fall temperatures in the surface layer of the ground move back and forth across zero frequently.

Solifluction. This is the slow flow usually of dispersed ground down a slope caused by excessive wetness and a reduction in the angle of internal friction. As a result the ground flows down the slope under its own weight in the form of "tongues."

2-2. PHYSICAL AND MECHANICAL PROPERTIES OF FROZEN GROUND UPON THAWING

The physical and mechanical properties of frozen ground depend on temperature conditions. Frozen ground with low negative temperatures has a high mechanical strength, reaching 10 kg/cm^2 or more. When it thaws, the physical properties of this ground change sharply. It assumes a fluid consistency and loses its bearing capacity entirely. Other physical and mechanical properties of the ground also change. Hence the physical and mechanical properties of the ground must not be considered without taking into account temperature conditions, loading, and time.

One of the principal factors involved in the change of physical and mechanical properties of frozen ground with thawing is the sharp change in structure as a result of breakdown of the ice-cement bonds accompanying the transition from ice to water [98]. Thawing of pore ice in frozen dispersed material occurs not just at 0°C but, in accordance with the theory of incomplete freezing of water, in dispersed frozen ground with any rise in temperature, even in the negative temperature range.

According to this theory as the temperature of the ground approaches 0°C, its strength properties deteriorate, but the frozen ground still will possess a certain strength because cementation has not yet completely disappeared and also many voids will be filled with ice, which joins the individual particles mechanically into aggregates which are quite dense. But when all the pore ice thaws, the structure of the ground is sharply altered [98].

After freezing the ground assumes special physical and mechanical properties. Friable materials assume the properties of solid rock after freezing, and they usually can be worked only after they have been broken up. At the same time the thawing of frozen ground can lead to a loss of bearing capacity as a foundation for construction.

Freezing changes the thermophysical and other properties of the ground in a fundamental way. Thus its percolation properties are completely changed; convective heat exchange is practically stopped in all frozen ground; the electrical conductivity of the ground is altered, and it practically becomes dielectric. When the ground freezes, moisture is redistributed and a cryogenic texture develops together with frost fissuring, the formation of ice veins, etc. At the same time thawing of such ground is often associated with subsidence, solifluction phenomena, thermokarst, etc.

The change in the physical and mechanical properties of frozen ground accompanying thawing varies in ground of different texture. In the case of stratified and reticulated texture there are sharp changes both in the cohesion of the ground and in its angle of internal friction. The angle of internal friction either approximates that for the unfrozen ground or becomes less than it. As far as cohesion is concerned, this will be significantly less than that of the same ground not subject to freezing and thawing, or it will be close to zero.

The bearing capacity of ice-saturated loamy sand and loamy ground is small after thawing, and special steps must be taken when structures are to be placed on such ground.

In the case of sandy and gravelly ground the reduction in bearing capacity following thawing is insignificant in comparison with the unfrozen state. Thawing sandy ground differs from unfrozen sandy ground in the fact that subsidence is appreciably greater and continues during the entire foundation thawing period, while the subsidence of unfrozen sand is practically finished during the construction period.

One of the main problems in placing structures on thawing ground is that of its deformation with thawing and related problems of settling of foundations and structures. Foundations and structures on such ground settle due to compaction of the ground under its own weight, the weight of the structures, and the weight of other external loads.

Another phenomenon related to the severe deformation of foundations is also typical of the thawing of frozen ground. This is subsidence, which is the rapid irreversible deformation of the foundation with a sharp change in its structure and even with material being forced out from under the construction. Subsidence of frozen ground following thawing is one of the principal causes of deformation and damage to structures built on frozen ground.

The susceptibility to subsidence with thawing is the relative compression of the ground as we go from the frozen to the thawed state under a load of 1 kg/cm^2. Ground is considered susceptible to subsidence when this subsidence index falls between 0.03 and 0.1, while it is considered not subject to subsidence or highly susceptible when the index is less than or higher than these values.

Structural characteristics of frozen ground determined by its cryogenic texture affect the nature of deformation of the ground when it thaws and the total settling of structures placed on such ground. Frozen loamy ground, which has a high total moisture content and a well-defined cryogenic texture, can be expected to produce strong and uneven settling of a foundation. Deformation of thawing ground occurs mainly as the result of melting of ice inclusions and change in the structure of the ground due to its own weight and that of external loads.

In order to estimate the susceptibility of thawing ground to compaction and to calculate the settlement of structures, various formulas have been proposed by a number of authors. We give below a formula suggested by N. A. Tsytovich, where the total subsidence of the foundation when the ground thaws (s, cm) is considered to consist of subsidence due to thawing itself and subsidence due to compaction:

$$S = A_o h + ahp \tag{2-1}$$

where A_o is the thawing factor introduced: $A_o = A_M/(1 = \varepsilon_1)$; a is the compaction coefficient, cm^2/kg; p is the pressure, kg/cm^2; h is the thickness of the compressed layer, cm; A_M is the thawing coefficient; and ε_1 is the initial poresity coefficient.

It follows from formula (2-1) that subsidence of frozen ground with thawing consists of two parts: thawing or thermal subsidence ($A_o h$), which is not dependent on external pressure, and compaction subsidence (ahp), which is directly proportional to extermal pressure.

2-3. RHEOLOGICAL PROPERTIES OF FROZEN GROUND

Frozen ground and permafrost, as Tsytovich states [102], are materials in which stresses and strains caused by loading do not remain constant but change with the course of time.

The presence of constituents such as ice and water in frozen ground gives it new rheological properties; i.e. creep occurs, stresses are relaxed, and strength is reduced.

The strength of frozen ground depends not only on its temperature conditions but also on the duration of loading. In contrast to unfrozen ground, frozen ground typically shows well-defined rheological properties; i.e. it creeps under constant loads and stresses are relaxed when deformation remains unchanged. By relaxation here we mean the effect expressed in terms of the change with time of stresses in a material accompanying constant overall deformation. These rheological processes in frozen ground determine its strength. The strength of frozen ground is quite high under brief loading, but the strength declines significantly as loading is prolonged.

According to S. S. Vyalov [36] the strength of frozen ground is a logarithmic function of duration of loading, and the parameters of the relationship depend in turn on the type of ground and its temperature.

2-4. PERCOLATION PROPERTIES OF FROZEN GROUND UPON THAWING

The physical, mechanical, and thermophysical properties of frozen ground vary widely. This diversity means that the physical nature and percolation mechanism of water in such ground are complex.

The thawing of frozen dispersed and fissured ground makes it permeable to water. At the same time the moving water brings ever-increasing amounts of heat and promotes further thawing.

The thawing of unfissured permanently frozen rock does not alter its percolation properties, although heat transport in such materials at positive and negative temperatures is by conduction. Percolation phenomena, movement of water, diffusion, etc. are typical of fissured and unconsolidated coarsely skeletal rocky and dispersed materials, and heat exchange in them typically involves a range of different forms and mechanisms of heat transfer.

Water seeps through frozen ground when communicating pores and cracks are present. As percolation takes place, ice thaws rapidly, and this in turn tends to increase total porosity and thus the rate of percolation.

The percolation properties of thawing frozen ground are not constant with time, reaching maximum values under given conditions when the entire mass of ground reaches temperatures in the positive range.

Ground that has thawed basically retains the large voids resulting from its cryogenic texture. This is an important factor in its percolation characteristics. G. V. Porkhayev [74] found that when a loam with a stratified cryogenic texture, a moisture content of 42-49%, and a density of 1400-1500 kg/m^3 was thawed, the percolation coefficient in the direction parallel to the ice layers was 0.0155-0.0270 m/hr, while in the perpendicular direction the coefficients were 0.0089-0.0096 m/hr, and after the structure was disrupted -- 0.0050-0.0060 m/hr.

According to the data of L. N. Khrustalev [96] the percolation coefficient of thawed loams with structure undisturbed may be several hundreds of times larger than after the structure is disrupted.

The nature and extent of change in the percolation and other physical and mechanical properties of permafrost following thawing depend on various factors, including above all the cryogenic texture, the shape and size of the ice inclusions, their direction, the distances between them, irregularities in the materials, etc.

Chapter 3

SHORT HISTORICAL REVIEW OF DAM CONSTRUCTION IN THE FAR NORTH OF THE USSR

The economic exploitation of permafrost regions lags significantly behind the development of the other regions of this country. Hence water-related construction in these regions has not been developed as it should have been.

The problem of building dams on permafrost arose in connection with the startup of industrial and agricultural activity as well as construction related to transport in the eastern and northeastern regions of the country.

The first information on dam construction on permafrost dates from the end of the Eighteenth Century. The project was an earth dam on the Mykyrt River to create a reservoir at the town of Petrovsk-Zabaykal'skiy [97]. Construction of this dam, which was 910 m long and 9.5 m high, was completed in 1792. The dam was constructed of coarse loamy sand and had a wooden spillway. The dam was built in the wintertime in order to freeze the body of the dam and maintain it in the frozen state. This dam was used successfully for about 140 years. In 1929, during repair operations on the spillway, the temperature regime at the base of the spillway was disturbed; the ground thawed and seepage started. Subsequent use was possible only after the dam had been completely rebuilt.

During the construction of the Transbaykal and Amur Railroads (1912-1916) several low-rise earth dams were built to create small reservoirs for domestic drinking water [58]. This was the first time that serious difficulties were encountered due to the presence of permafrost.

A series of failures, in the opinion of M. Ya. Chernyshev [103] "in most cases were due to our technological ignorance."

In the winter of 1913/14 reservoirs were built on the Amazar River to supply water to the railroad stations of Mogocha and Amazar. The building of temporary dams provided a means for determining whether structures of this sort could be built and operate reliably.

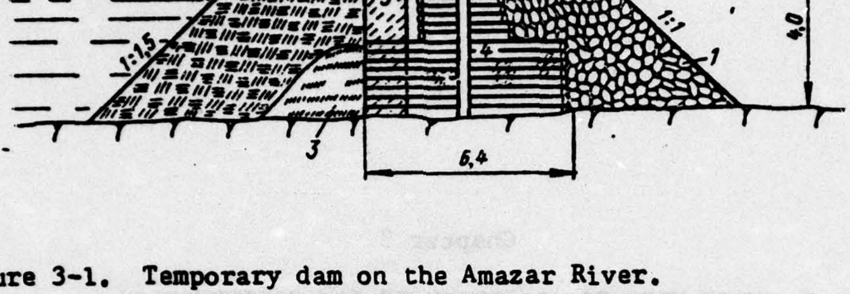

Figure 3-1. Temporary dam on the Amazar River.
1 - Rockfill; 2 - Loam; 3 - Bags filled with earth; 4 - Cribwork loaded with rock; 5 - Pit for freezing the earth.

Construction on the temporary dam on the Amazar River to provide water for the Mogocha railroad station was started in the middle of October and completed in the second half of November. After construction was completed and the reservoir was being filled, washout holes began to form in the body of the earth dam, and efforts to control these lasted almost the entire month of November. Water leaks could be reduced only after an open trench was cut along the entire length of the dam on the upstream side at the cribwork in the body of the dam in order to freeze the earth (see Figure 3-1). This temporary dam was considered to be a failure since water gradually was lost from the reservoir.

In December 1913 work was begun on a permanent fixed dam on the same river. This dam was finished at the end of March 1914, i.e. at the beginning of the spring flood. The dam was built on rock covered with a layer of alluvial material. Loose sand and gravel 1-1.5 m thick were removed and the foundation was stripped down to the rock.

The height of the dam was 4.0 m and the head 2.5 m. The dam had a cribwork cutoff filled with rock. The downstream wedge was built of rockfill, the upstream side of clay. This design was used with the idea that flood waters would spill over the crest of the dam (see Figure 3-2).

Figure 3-2. Dam on the Amazar River.
1 - Loam; 2 - Cribwork; 3 - Abutment; 4 - Rock.

In order to avoid seepage along the axis of the dam, a trench was dug. But adequate freezing of the body and foundation of the dam were not achieved, since this work was done after the structure was already in use and there was seepage under pressure [58].

First a temporary and then a permanent dam 4.5 m high were built on the Amazar River to provide water for the Amazar railroad station. Construction conditions and design of the dam were the same as for the dams at the Mogocha station. Wooden cribwork was sunk into loose alluvial deposits to a depth of 2.9 m and rested directly on the rock. The crest of the dam was 1.6 m above the bottom of the river.

Temporary and permanent dams impounding 2-3 m of water were built on the Urka and Chichatka Rivers. These dams usually were without a tooth. The cribwork was set on the foundations after the gravel had been cleared away. Seepage losses were very high.

The long severe winter, freezing of the water in the river, and the formation of a thick ice cover made it necessary to build a reservoir that was quite large. At the same time the violent summer floods bringing large amounts of sediment and driftwood made it necessary to use other dam designs -- sectional dams and dams with provision for overflow throughout their length.

Figure 3-3 shows the cross-section of a dam built on the Amazar River. These dams contain masonry functioning as a weir over the entire length and girders of the Poiret type installed on the crest. At first the girders, which were mounted on hinges, fitted together on the crest, but since during the passage of the flood waters the girders were torn apart by floating ice and driftwood, it became the custom to remove them during the flood period. A deficiency of such dams is the absence of a lateral bypass, making it impossible to flush out the reservoirs, and the latter were rapidly silted up.

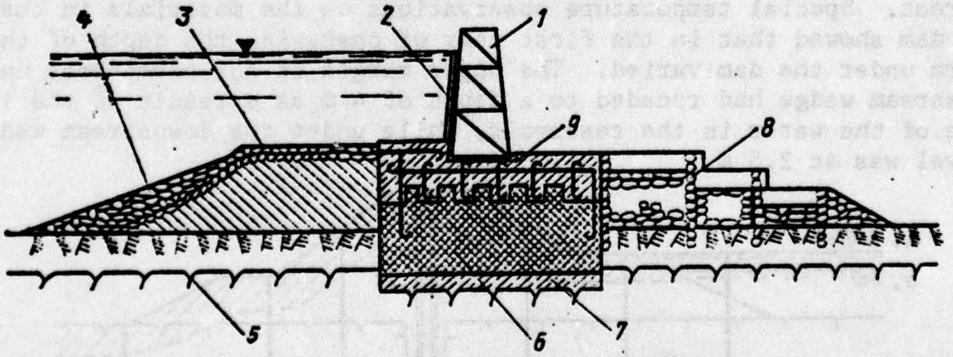

Figure 3-3. Dam on the Mazar River.
1 - Poiret girders; 2 - Shields; 3 - Wood facing; 4 - Stone facing; 5 - Rock; 6 - Masonry; 7 - Concrete; 8 - Cribwork loaded with stones; 9 - Reinforced concrete.

The theoretical basis on which the dams were built was not of a high order. For this reason certain specialists felt that dams made of local materials would be shifted on the thawed frozen layer by hydrostatic pressure, and they suggested using cribwork supports filled with stones at the base of such dams [43].

The examples of dam construction given above demonstrate a definite interest in finding a solution for the problem of building reservoirs under unusual conditions. These efforts were very intense and bold. Many problems which arise even today in hydraulic construction on permafrost had already arisen at that time. It should, unfortunately, also be mentioned that some of the errors of our predecessors are often repeated these days.

M. Ya. Chernishev stated as early as 1933, "The opinion has long been maintained among many structural engineers that permafrost should be a desirable base on which to build a reservoir regardless of the geological structure of the underlying rock." And further, "subsequently it has become clear from a number of observations that permafrost under a reservoir begins to degrade rapidly under the influence of the warm water of the reservoir and its surface horizon drops from year to year. Thus underlying friable deposits and bedrock thaw out, and if the structure of these materials includes fissures, and if a waterproof curtain is not put in the way, bottom seepage begins to occur" [103].

As an example of the development of "bottom seepage," as Chernishev calls it, i.e. seepage through thawed foundation materials, we might mention the dam on the Right Magdagacha River, which was built in 1932 (see Figure 3-4). The dam was built on tuffs and fragmented porphyrites, which were in a permanently frozen state. The permafrost under the dam was no more than 25-30 cm thick. The concrete cutoff in these rocks was not sunk deep enough, and steps were not taken to preserve the foundation materials in the frozen state. The dam operated satisfactorily during the first year even though base seepage was observed, and it was not considered to pose any threat. Special temperature observations on the materials in the body of the dam showed that in the first year of operation the depth of the zero isotherm under the dam varied. The upper margin of the permafrost under the upstream wedge had receded to a depth of 4 m as a result of the influence of the water in the reservoir, while under the downstream wedge the level was at 2.5 m.

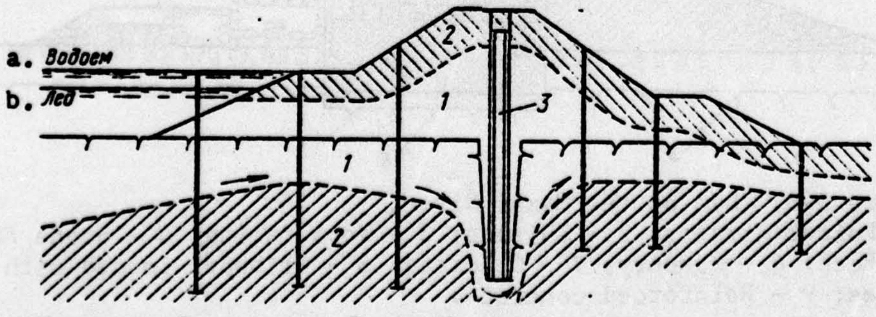

Figure 3-4. Dam on the Right Magdagacha River.
1 - Thawed ground; 2 - Frozen ground; 3 - Concrete cutoff.
Key: a - Reservoir; b - Ice.

As a result of thawing of the permafrost in the base of the dam, after two years seepage had reached such a volume that it threatened to empty the reservoir completely. The rate of seepage reached 8,000 m^3/day, of which about 5,000 m^3/day was seepage through the foundation materials. During the winter the seepage water was captured and pumped back into the reservoir. Subsequently the fissured materials under the dam settled due to melting of the ice; the concrete cutoff broke in several places; a crack formed in the concrete abutment joining the concrete spillway to the earth dam; and the dam was destroyed in 1935.

A similar thing happened to another dam. A dam 530 m long at the crest, 9.6 m high, and with a head of 7 m was built in 1932 on the Bol'shoy Never River to provide a water supply for the Skovorodino railroad station. The clay core of this dam was "in a semiliquid state." Special boring into the bed of the reservoir showed degradation of the permafrost, which under natural conditions was about 90 m thick. But the bedrock here (Jurassic deposits) was less disintegrated and fissured than in the earlier example. It was also covered with a layer of silty loam and there was no bottom seepage. But in the dam the body, which was made of silty loam, and the clay core were not sufficiently reliable. Considerable seepage developed through the body of the dam, reaching 2,000 m^3/day. The seepage water was collected and pumped back into the reservoir. The condition of the dam became uncertain, and prompt steps were taken to assure its stability. In 1934 the upstream and downstream slopes were covered with gravelly earth. A sheet-pile cutoff was sunk into the core to a depth of 6-8 m, and the fluid earth behind the trough was replaced with loam, which was placed dry and compacted. Seepage was reduced but ground thawing at the base continued. In 1936 the zero isotherm had dropped 6-18 m, although there was no marked deterioration of seepage conditions in the foundation nor deformation of those materials [97, 103].

Experience in building and operating these dams showed that fissured rock permafrost, when it thaws, is subject not only to seepage but also to subsidence, and rigid structural elements built on such base materials are destroyed. Beginning in the period from 1940-1950, when heavy exploitation of the natural resources in the Yenisey North, in Magadan Oblast, and in the Yakut ASSR started, there has been significant development in the construction of dams on permafrost. Several low-pressure dams were built in the Noril'sk region (on Dolgiy Creek, Kvadratnyy Creek, Nalednyy Creek, and elsewhere).

A temporary dam impounding 1-2 m of water was built on the creek flowing from Lake Dolgoye. Later on it was flooded by the reservoir formed by a permanent dam with an ice-earth frozen curtain. Use of this dam was possible only after a number of important steps were taken to stabilize its temperature regime, its seepage resistance, and its overall stability. In addition to a brine system, an air cooling system was installed, an ice refrigerator was built on the downstream slope, and other steps were taken.

Figure 3-5 shows the dam built on Kvadratnyy Creek. The dam, which was built of thawed ground, failed a month after the reservoir was filled. It was built on loamy sand and loams with a high content of ice. The failure was due to seepage at the juncture with the right bank and in the foundation. The seepage resulted from thawing of the ice contained in the foundation materials and also evidently from inadequate workmanship where the dam joined the banks.

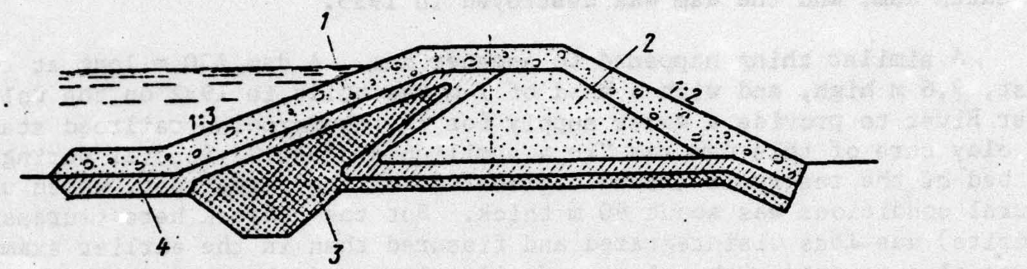

Figure 3-5. Dam on Kvadratnyy Creek.
1 - Sandy-gravelly earth; 2 - Loamy sands; 3 - Clay; 4 - Loam.

In reviewing experience in dam construction during this period it should be stated that this stage in the development of hydraulic construction in the Far North, a period of massive effort and vigorous growth, unfortunately repeated errors typical of the early period of dam construction on permafrost and emphasized the unsuitability of ordinary construction methods for building on permafrost.

But there are examples also of successful construction and operation of dams during this period. Here we should mention the dam with a head of 1.5-2.0 m built in 1942 on Razvedochnyy Creek (Figure 3-6). The body of the dam was built of sandy materials and covered with peat and a stone facing. Seepage was blocked by a sheet-pile cutoff.

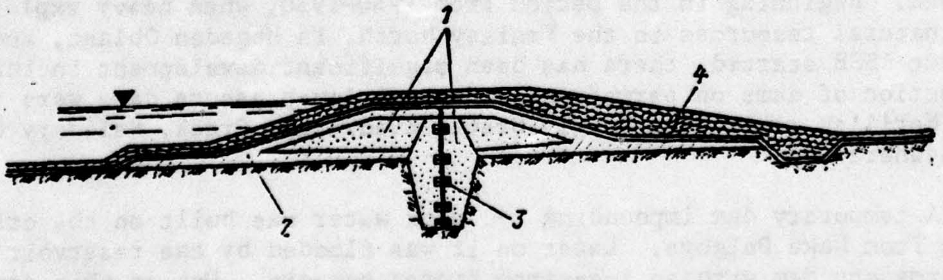

Figure 3-6. Dam on Razvedochnyy Creek.
1 - Fine sand; 2 - Peat; 3 - Cutoff wall; 4 - Stone facing.

Another successful example is the dam on the Nalednaya River, which is about 65 m long, 10 m high, and was built in 1950-1951 to divert the river into a new channel (see Figure 3-7). Its planning took into account experience gained in building and operating the dam with the frozen seepage curtain on Dolgiy Creek. The air cooling system consisted of 30 columns. Since the downstream wedge of the dam will always be in the frozen state, it was built by placing blocks of frozen ground and pouring water over them. The downstream slope was protected from the warming effect of snow and direct solar radiation by a wooden roof, the space under which was ventilated with a special fan. After two years of operation the thawed zone under the channel was frozen and stable negative temperatures were established in the foundation and the downstream wedge.

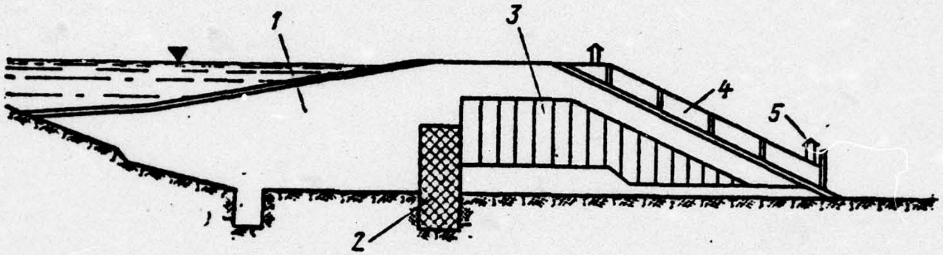

Figure 3-7. Dam on the Nalednaya River.
1 - Loam placed in the unfrozen state; 2 - Clay cement core;
3 - Loam placed as frozen chunks; 4 - Roof; 5 - Ventilating pipe.

There was a large amount of dam construction on permafrost in Magadan Oblast. Dams were constructed there on the Sol'veyg (1944), the Middle El'gen (1945), Lake Kedrovoye (1943-1945), and the Utinaya, Myaundzha (1954), Kamenushke (1957), Zharkaya, Kadykchan, Pevek, and Kazachke Rivers, and others. Most of them were built on thawed ground except for the dams on the Myaundzha and Pevek Rivers.

Two dams with head pressures of 6-7 m were built in Verkhoyansk Rayon (Yakut ASSR) near the village of Ege-Khaya on the Kumakh River in 1940-42. They were built of loamy sand and loam materials with wood cutoff walls (see Figure 3-8). Seepage through the body of the dam occurred during the first years of operation, but subsequently the dam froze and seepage stopped.

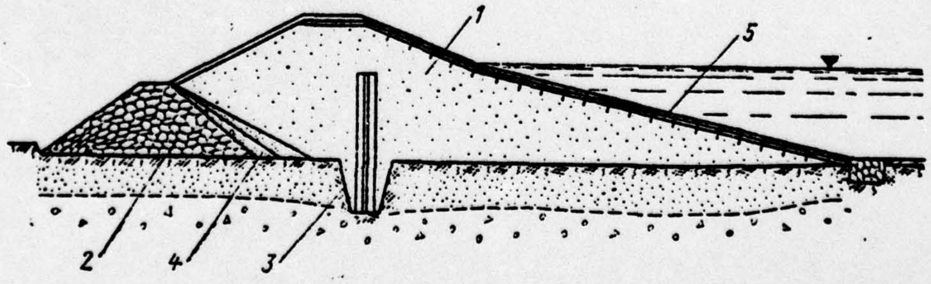

Figure 3-8. Upper Kumakh Dam.
1 - Loamy sand; 2 - Rock full; 3 - Wall of wood sheeting (cutoff);
4 - Inverted filter; 5 - Stone facing in grid spaces.

Since 1959 dam building has grown particularly fast in western Yakutia. During that period about 10 dams have been installed for various purposes, of both the unfrozen and frozen types. These include such unusual dams with frozen curtains as the dams on the Irelyakh and Sytykan Rivers. During this period there was a better selection of construction methods and design solutions, and the workmanship was of higher quality.

Chapter 4

CLASSIFICATION OF DAMS ACCORDING TO ECONOMIC PURPOSE AND DESIGN

4-1. ECONOMIC PURPOSE OF DAMS

Dams may be built under conditions of the Far North to create reservoirs for hydroelectric power stations, to create tailing ponds for ore-dressing operations, to create cooling ponds for thermal electric power stations, to provide water supply for settlements and industrial enterprises, to flood dredging works, and to meet agricultural needs.

In addition to dams for impounding water, hydroelectric stations usually include water escape structures, intakes, structures to control ice, and structures for other purposes.

Hydroelectric power station dams. In view of the hydrological behavior of rivers under permafrost conditions, where the river discharge varies by factors of hundreds or thousands during the year, a reservoir of adequate capacity must be provided at hydroelectric stations.

Hence the planning and building of dams for hydroelectric power stations must meet very high requirements. Hydroelectric power station dams already built and under construction are discussed in detail later (Section 5-3).

Dams for thermal electric power stations. Operations at thermal electric power stations involve the consumption of considerable quantities of water.

Only rivers with lots of water can supply the needs of electric power stations during the low-water season by "direct flow." Power stations mainly operate with a circulating water-supply system using cooling ponds. This system is better from the engineering and economic point of view than the use of cooling towers and spray tanks.

The use of cooling ponds provides a power station with circulating water at consistently low temperatures and also permits multiple use of the reservoir for other water-supply purposes. The dams for such reservoirs are classed as major Class I structures on the basis of capital investment and importance.

The water of cooling reservoirs may reach a temperature that is quite high, and temperature conditions in the dams and their foundations may become very stressed. Under these conditions dams with a frozen curtain are most applicable, since they provide stable temperature conditions in the dam itself and in the foundation.

Dams of this sort were built for the cooling reservoirs of the Arkagalinskaya Thermal Electric Power Station on the Myaundzha River, for the Noril'sk Thermal Electric Power Station (on Lake Dolgoye), etc.

Tailing pond dams for ore-dressing of plants. The wet method of concentration is ordinarily used in processing ores at ore-dressing plants, so that the tails come out as slurries and are conveyed to the storage site (tailing pond) through pipes. Tailing ponds are usually built in river valleys by cutting them off with dams or surrounding natural depressions with dikes. The solid particles settle out of the slurry in tailing ponds, and the clarified water is discarded or used again.

The construction of tailing facilities involves substantial investment, especially to build the hydraulic structures. Special techniques are used in permafrost regions for building and operating dams for tailing ponds.

In building dams for tailing ponds in the valleys of watercourses one must deal with the need to accommodate flood waters, since the catchment area involved is frequently large and the flood discharges greatly exceed the amount of clarified water discharged.

Drain structures associated with tailing pond dams usually are built as open drainage canals and trenches, siphon or shaft spillways with collector drains.

The requirements to be met by tailing ponds as engineered structures are quite high. Damage to these facilities, especially when the water is reused, would interfere in a major way with the operation of ore-dressing plants, and therefore very careful attention should be given to the engineering geology and frozen ground conditions of the dam foundation. Sites should be selected where the basement materials when thawed are relatively impermeable and not much subject to deformation.

Tailing ponds usually are built in two stages: the first stage consists of the pioneer or starter dam built of fill or alluvial materials, and the second stage, the alluvial buildup of the dam from tailings as the pond is used (see Figure 4-1).

The height of the starter dam is determined by the need to create a pond with the volume necessary for clarifying the slurry during the early period of operation, taking into account the quantity of tailings deposited.

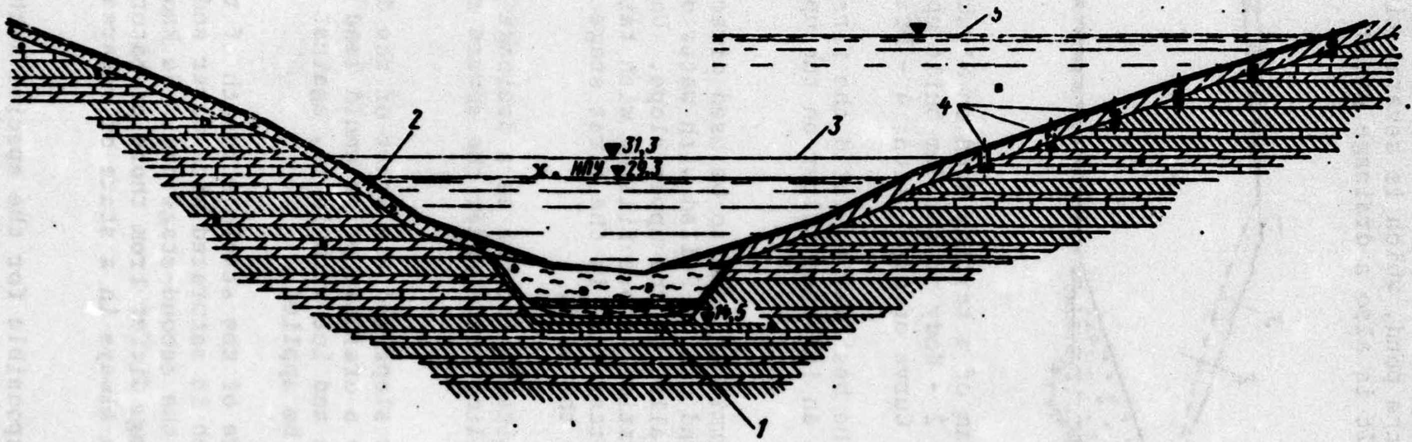

Figure 4-1. First-stage dam of a tailing pond on N'yukka Creek. 1 - Outline of tooth; 2 - Outline of zone where topsoil was removed; 3 - Crest of starter dam of tailing pond; 4 - Bank drains arranged to correspond to stages of deposition of tailings; 5 - Crest of dam built up by alluvial fill.
Key: x - NHL.

A special characteristic of the function of the starter dams is that when the pond is filled with tailings they act as a supporting prism for the second-stage siltation dam of the pond, which is several times as high as the starter dam (Figure 4-2). It is also a drainage prism.

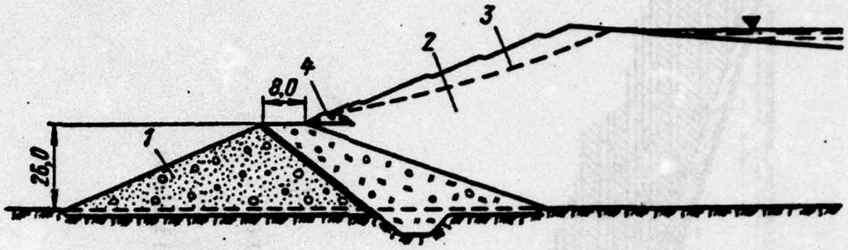

Figure 4-2. Second-stage dam of a tailing pond on N'yukka Creek. 1 - Starter dam, built dry; 2 - Body of the dam built up by settlement of tailings; 3 - Curve of depression; 4 - Drainage.

Under ordinary conditions the best way to build the first-stage dam is with rock fill and to build an inverted filter on the upper slope to prevent piping.

But this design does not permit the dam to be used directly as a water-retaining structure. The pond can be filled with water only after a blanket seal of tailings is installed on the upper slope. Unfortunately the pond must sometimes begin operation before filling with tailings starts. It often happens that starter dams in the first stage are used as impounding structures to create the pond.

But a starter dam is not suitable for use as a drainage prism in the second stage when building tailing ponds under the severe climatic conditions of the Far North.

During the winter the lower slope and the crest of the dam freeze to a depth of 4-5 m and its use as a drainage prism would lead to the formation of a stratified zone of earth and ice with in it, against which the entire hydrostatic pressure would be applied.

Moreover, one cannot be sure of the static strength of the body of a drainage (supporting) prism which is saturated with water and ice and is loaded by the alluvial part of the second-stage dam. We know that the mechanical properties of the tailings differ from those of natural earth, and in a tailing pond this material is always in a state of supersaturation, and sometimes suspension.

These circumstances are responsible for the special methods used to build starter dams for tailing ponds in the Far North.

Starter dams are usually built as unfrozen dams where the foundation is allowed to thaw during operation but special steps are taken to prevent loss of bearing capacity. The impermeable component of a starter dam may be a seal with or without an apron, while the supporting prism is built of permeable earth.

The drainage structures of the starter dam should be buried in the body of the dam, and, in the freezing zone of the lower slope, in heated ditch galleries.

When starter dams are built as no-seepage dams, a curved depression naturally appears with alluvial buildup of the second stage of the pond above the crest of the starter dam. In these cases the builder should provide for buried pipe drains at the level of the starter dam crest. Such drainage may consist of inverted filter layers and rock fill or perforated pipe installed in the drainage prism.

Dams for placer dredging. The dredging method of working placer deposits of minerals is very widespread. Earth dams with wooden by-wash structures are mainly used to flood the area being worked, but one can also build rock fill dams with appropriate elements to reduce seepage.

In the Far North flooding placer works extends the mining season, prevents the deposits from freezing in the winter, promotes thawing of frozen materials containing the placer minerals, and makes them easier to work.

Building dams for dredging operations differs little from building dams for other purposes. The dams are built 4-6 m high, depending on the level of flooding required, and 5 m wide at the top, with slopes of 1:3 and 1:2.

The period of service of dams for dredging operations is usually short. The demands placed on them are not great, and therefore they can be grouped with those at the bottom in terms of capital investment.

We might cite as an example a cascade of five earth dams built to work deposits on the Irelyakh River (see Figure 4-3). Some of the dams were used only a few months, while others have been in operation several years. These dams impound 2.2-4.5 m of water. The dams were built of diluvial loams. They were spaced 1.5-4 km apart. The lower dam had a wooden crib-work spillway with panel gates which were removed to pass flood waters reaching 35 m^3/sec (Figure 4-4), while the other dams had lateral spillways to carry off the spring flood and siphon spillways for use during dredging operations.

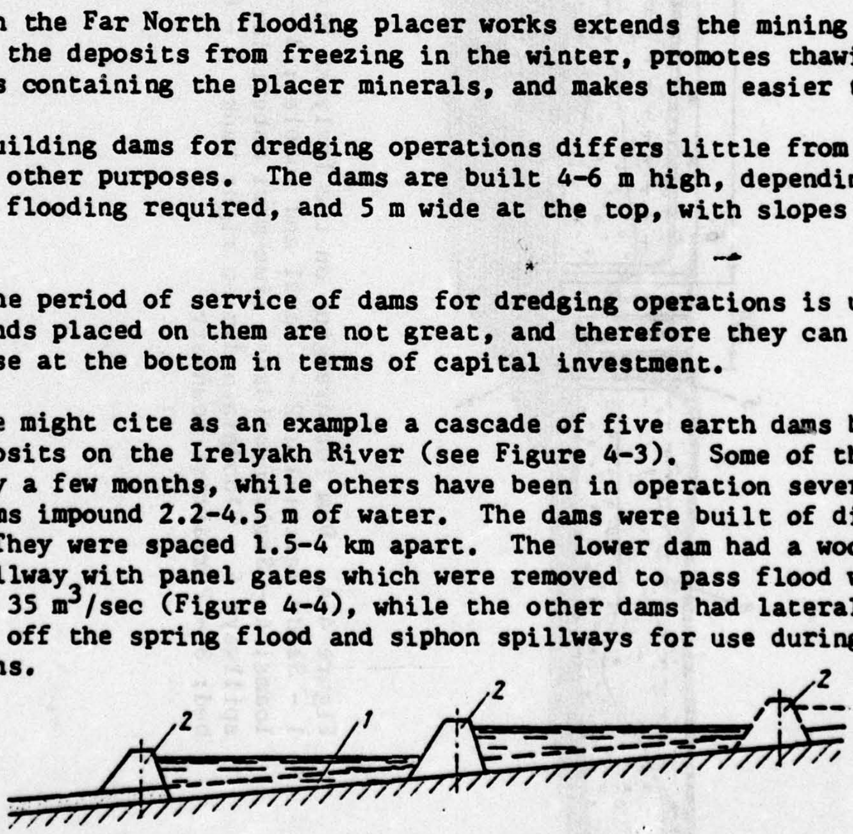

Figure 4-3. Cascade of dams for dredging operations on the Irelyakh River. 1 -- Layer containing placer minerals worked by flooding; 2 - Cascade dams.

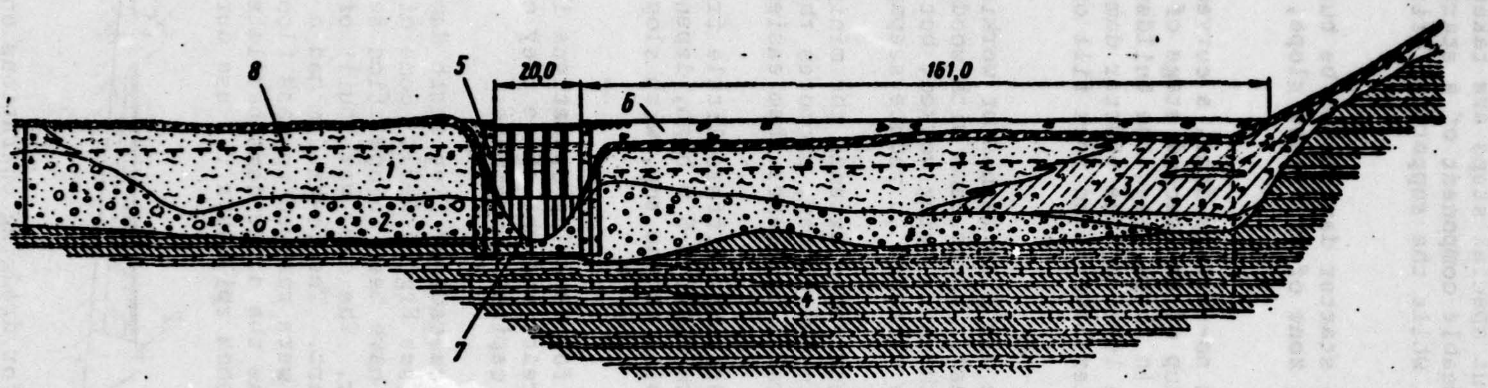

Figure 4-4. Dam for dredging on the Irelyakh River.
1 - Sandy materials; 2 - Gravel and cobbles; 3 -- Diluvial loams; 4 - Semiconsolidated lime-marl materials; 5 - Cribwork spillway; 6 - Floodplain dam on right bank; 7 - Bottom of river bed; 8 - Permafrost boundary.

Dams for reservoirs to supply water for drinking and industrial purposes. In permafrost regions water can be impounded from natural sources for water supply only from large rivers which do not freeze up in the winter and have a sufficient year-round discharge or from deep lakes which do not freeze up.

When most rivers and streams cannot be used for water supply because they freeze, the problem of industrial and drinking water supply usually is solved by building dams and creating large reservoirs, where spring flood waters are accumulated for subsequent use in fall and winter. The dams may be either of the frozen or unfrozen type.

Dams for agricultural purposes. Some areas in the permafrost zone have very continental and dry climates. Little precipitation falls; the rivers freeze almost completely in the winter, and most small rivers dry up in summer, since practically none of their flow comes from the ground water. With the arrival of spring thaws, most of the snow water runs off on the frozen soil. Hence water for agriculture and other purposes must be drawn from the surface meltwater runoff in the spring.

In 1959 there were about 300 dams in Yakutia: mainly fixed earth or wood dams [47].

Since until recently construction of such dams had not received adequate attention, they were frequently destroyed by floods because they were built without appropriate water escape structures and the workmanship was poor.

An example of proper design and construction is the dam on the Suola River in the Yakut ASSR (see Figure 4-5) [84].

4-2. TYPES OF DAMS

Dams built of local materials -- earth dams, rock-earth dams, rock fill dams, and wood dams -- are most widely used in the North, a practice which is based on their advantages over other types of dams.

In recent years it has been shown that dams can be built of natural materials under practically any climatic, geological, and frozen-ground conditions [4]. Any materials available at the construction site are actually suitable for building such dams.

The body of the dam is built using standard methods, and mechanized techniques are used to dig the materials, transport them, and place them. Hence the construction of dams from local materials is usually less costly than building dams of other types of materials.

In addition to the advantages mentioned for dams made of local materials (except wood dams) there are others, namely: the material used in the body of such dams does not lose its characteristics with time, the strength of the structure even increasing; and such dams rarely require repair during use. Foundation quality requirements for dams built of local materials are less strict than for concrete dams. Also, dams made of local materials can be made higher when necessary without draining the reservoir.

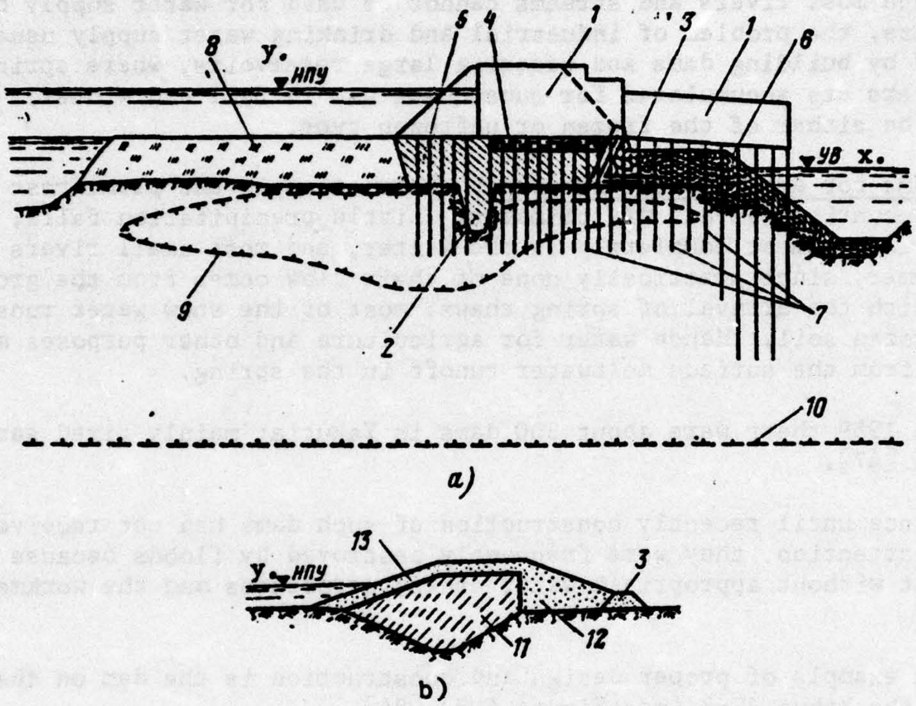

Figure 4-5. Dam for agricultural purposes on the Suola River. a - Cross-section through spillway; b - Shore dam; 1 - Spillway; 2 - Tooth; 3 - Drainage; 4 - Apron; 5 - Loam; 6 - Stone facing; 7 - Pilings; 8 - Earth fill; 9 - Lens of frozen ground; 10 - Permafrost boundary; 11 - Loamy sand; 12 - Sand; 13 - Gravel fill stabilization.
Key: x - WL; y - NHL.

A drawback of dams built of local materials is the impossibility of discharging flood waters directly through the dam. This must be done with water escape structures usually outside the body of the dam. Some experience has been gained recently in passing floodwaters across the crest of the dams. Thus, during construction of the Khantayka Hydroelectric Power Station in 1968 and 1969, spring floods were successfully allowed to pass across the incomplete rock fill dam after extra steps were taken to stabilize the lower slope. Rates of discharge reached 4200 and 6700 m^3/sec [76, 77]. Spring floodwaters and ice were also spilled across a temporary dam 7.5 m high on the Sytykan River in Yakutia in 1971 and 1972.

a) Earth Dams

Earth dams, of course, are classified both according to height and according to method of construction, cross-sectional design, method of seepage prevention, and capital class.

Dams are classified according to height into high dams (with maximum head of more than 50 m), medium dams (with heads of 15-50 m), and low dams (with a head of less than 15 m).

Dams are subdivided according to method of placement, into fill dams, which in turn may be built by placing the earth dry or tipping it into the water, and hydraulic fill dams, which are built up using hydraulic techniques.

The type and design of a dam are chosen on the basis of engineering and economic comparison of various alternatives, which differ from one another in the method of using material, frozen-ground conditions, the method of construction, etc.

Earth fill dams built dry. Earth dams built in the Far North are subdivided into the following types according to cross-sectional design (Figure 4-6):

1) dams built of homogeneous material (Figure 4-6, a);

2) dams of more than one material, the materials placed in gradually increasing order of permeability to water from the upper pool to the lower (Figure 4-6, b);

3) dams with a blanket seal of relatively impermeable materials. The supporting prism of such dams is usually built of homogeneous material with a high percolation coefficient (Figure 4-6, c);

4) dams with a core of relatively impermeable materials (Figure 4-6, d). This type of dam is susally used when the basic dam construction material is quite permeable -- rock, gravel, etc.;

5) dams with a solid cutoff wall (Figure 4-6, e).

It should be noted, however, that this breakdown of dams according to type of construction is valid, when built on permafrost, only for the so-called unfrozen dams, which are built practically without regard for temperature conditions in the foundation and the body of the dam. Only two types of dam design are distinguished in the building of frozen dams, namely types a and d in Figure 4-6, since in these dams the antiseepage element is the frozen ice-earth curtain created by artificial freezing of the ground (see Section 5-6).

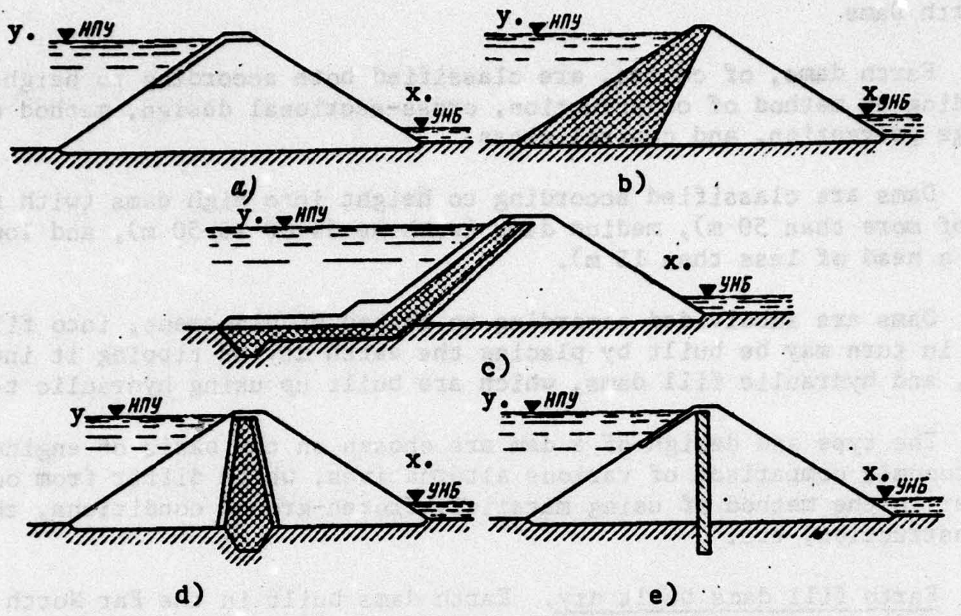

Figure 4-6. Types of earth dams.
a - Made of homogeneous material; b - Made of more than one material; c - With a blanket seal; d - With a core; e - With a cutoff wall.
Key: x - LPL; y - NHL.

Dams with a rigid blanket seal are not found in the Far North, but there are examples of temporary dams with a seal made of polyethylene film.

Dams with a solid cutoff wall are built quite frequently under the conditions of the Far North [8].

One of the types of antiseepage structures in the body of earth and rock fill dams recommended recently [1] is the injection core, which is made by injecting a hydraulic cement solution under pressure through holes (see Figure 4-7). The supporting prisms of such dams can be built up of any material, including rock, but the central portion intended for injection should be built of sandy-gravelly or gravelly materials. The injection core of a dam is joined to a rock base or alluvial deposits in the foundation by cement injection also. The injection core can be built up layer by layer as the lateral prisms are built.

There is no question concerning the use of such a core in the construction of unfrozen dams on permafrost. If there is not enough cohesive soil (loam and loamy sand) in the construction area for use in the core of dams built by the frozen method, we feel it is also possible to build a frozen curtain in the body of the dam by freezing an injection core made of sandy-gravelly materials. It stands to reason that a reliable frozen curtain can be created in this case if care is taken to see that the injection core is properly built. This can be achieved by building the injection core in layers as the dam is constructed and by assuring very careful grouting of the contact zone between the sand-gravel fill and the foundation material. In spite of the apparent complexity of such designs, they can be more economical than freezing cores of traditional materials (loam and loamy sand). It should be kept in mind, however, that in the Far North materials suitable for use in the core of a dam are found in a thin surface layer, are permanently frozen, and they sometimes contain as much as 85% ice. After they have thawed during the short summer season to a shallow depth, the materials in the borrow pits are in a fluid state, and it is impossible to work them and place them into the dam without first drying them somewhat. At the same time, sandy-gravelly materials are quite widespread.

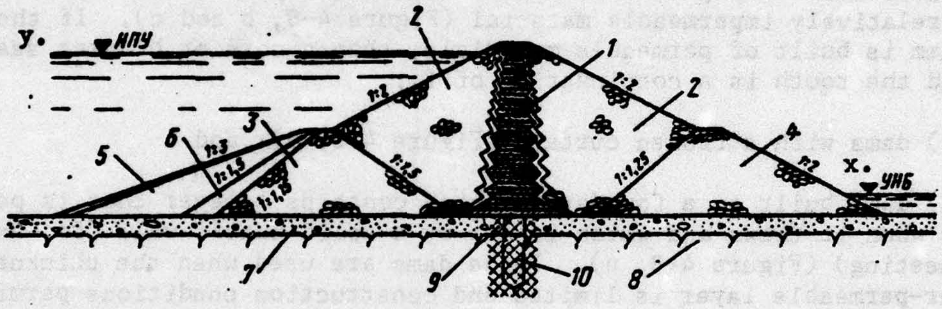

Figure 4-7. Dam with an injection core.
1 - Injection core; 2 - Lateral prisms; 3, 4 - Toe; 5 - Sand;
6 - Sand-gravel mixture; 7 - Alluvium; 8 - Rock; 9 - Area grouting;
10 - Grout curtain in the rock.
Key: x - LPL; y - NHL.

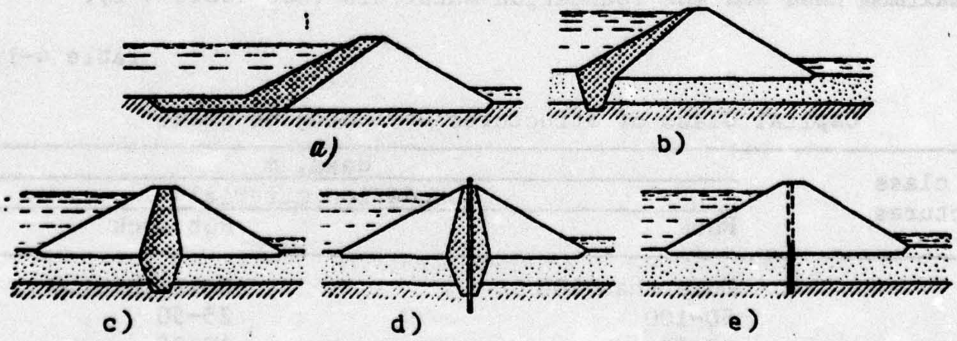

Figure 4-8. Types of attachment of earth dams to the foundation.
a - Dam with apron and blanket seal; b - Dam with tooth and blanket seal; c - Dam with tooth and core; d - Dam with frozen curtain; e - Dam with cutoff wall.

Dams are divided into the following types according to the design of antiseepage structures in the foundation, which depend on the particular engineering geology and frozen-ground conditions (see Figure 4-8):

1) dams with a blanket seal and apron made of relatively impermeable materials (Figure 4-8, a). The water-permeable layer in the foundation (usually quite thick) is not cut by the antiseepage elements, so that seepage is permitted in the foundation of the dam. With this subgrade outline the depression curve in the body of the dam is reduced and percolation losses are not as large. Such dams are used when the body is built of permeable materials and the foundation consists of sandy or gravelly materials which are relatively thick;

2) dams built on a foundation consisting of a relatively thin layer of materials which are permeable when thawed but which are cut by a tooth made of relatively impermeable material (Figure 4-8, b and c). If the body of the dam is built of permeable materials, then a core or blanket seal is built and the tooth is a continuation of it;

3) dams with a frozen curtain (Figure 4-8, d); and

4) dams built on a foundation which contains a layer that is permeable to water when it thaws and which is cut by a solid cutoff wall (for example, metal sheeting) (Figure 4-8, e). These dams are used when the thickness of the water-permeable layer is limited and construction conditions permit it to be cut. In this case the body of the dam may be built of uniform material with a core, apron, or cutoff wall.

According to SNiP II-A.3-62 all water-engineering structures are divided into four capital classes depending on the conditions under which they are used. The capital class of dams built of local materials depends on the maximum head and the foundation materials (see Table 4-1).

Table 4-1.

Capital Class of Structures Depending on Head.

Capital class of structures	Head, m	
	Foundation material	
	Rock	Not rock
I	More than 100	More than 50
II	50-100	25-50
III	20-50	15-25
IV	20 or less	15 or less

Requirements as to strength, stability, useful life, etc. depend on the capital class of the structure.

Hydraulic fill earth dams. Both hydraulic fill and semihydraulic fill methods are used to build dams in the Far North. The hydraulic fill method is widely used to build dams for mine tailing ponds. In this case dams up to 50 m high or more are built by hydraulic filling from one side where there is a starter dam, which is usually built dry.

There are no hydraulic fill dams in the Far North of the USSR, but such dams exist abroad: for example, the Hess Creek Dam in Alaska 64 km from the Arctic Circle built in 1940-46 to provide flooding for dredging of placer deposits. The dam is 25.5 m high with a volume of about 370,000 m^3 and impounds a reservoir of 9,240,000 m^3. The dam is 508 m long at the crest, 4.8 m wide at the top, and about 103 m wide at the base. The slopes are 2.5:1 at the base and 1.3:1 at the crest. The upper part of the dam for a height of 8 m was built dry in rolled layers. The core of the dam, built by hydraulic fill with fine material, constitutes from 1/6 to 1/10 of the width of the dam. This core is connected to the base by metal sheeting 6 m high, which was buried 3.9 m deep into the foundation of the dam. The damsite was covered with alluvial deposits averaging 6 m in thickness and with a maximum thickness of 12 m. These were heavily permeated with ice and contained large ice lenses. In preparing the foundation these materials were completely removed to bedrock. Construction operations for this dam are of great interest and are discussed below (see Section 6-2).

b) Rock Fill Dams

In the Far North, where rocky ground is widespread, there is every reason, including an economic reason, for building rock fill dams. Examples of such dams are the completed Vilyuy and Khantayka Hydroelectric Power Station dams, the dams on the Oyuur-Yurege and Sytykan Rivers, the dam for the Kolyma Hydroelectric Power Station, which is still under construction, etc.

Rock fill dams are usually built as fixed dams without provision for spilling water over the crest. Bypass structures are built to carry spring floods around the dam. But cases exist in which low-head dams have been built to spill water over the crest. In these cases the downstream slope and crest are appropriately stabilized. But in the Far North, where one must always be concerned with temperature conditions in the body and the foundation of the dam, dams permitting water to spill over the crest when they are in use can be built only in particular cases, for example, when there is a firm rock foundation, the dam is of the unfrozen type, and the specific discharge of water is low.

The principal advantages of building rock fill dams are that they use local materials, they are of simple design, they can be built throughout the year with highly mechanized methods, the height of the dam can be increased, and foundation requirements for these dams are less strict than for concrete dams.

Among the drawbacks of this type of dam we can mention that settlement is significant and this must be taken into account when antiseepage elements are built.

The body of the dam or its supporting prism is built of rock fill containing broken stone, massive rock, gravel, shingle, or boulder material, etc. Rock fill dams are subdivided into two basic types depending on the design of the antiseepage structures -- dams with a core or with a blanket seal (see Figure 4-6, c and d).

c) Earth-Rock Dams

Earth-rock dams are built when there is an insufficient supply of either material locally to build the entire dam of one material. There may be several designs used in building earth-rock dams.

In these dams the downstream wedge is built of rock fill, and the upstream wedge of loamy sand, loam, and other cohesive soils, to act as an antiseepage element (see Figure 4-6, b).

Earth-rock dams can be built so that the external prisms consist of rocky materials and the central part of the dam body, which is an expanded core, is built of less permeable cohesive or noncohesive materials. This construction may also be desirable when there are adequate amounts of cohesive and rocky materials available from excavations for other parts of the installation. The upstream and downstream wedges can be built of several types of materials. For example, the Summersville Dam (Figure 4-9) was built of various types of materials. Zone 1-2 is practically an expanded core.

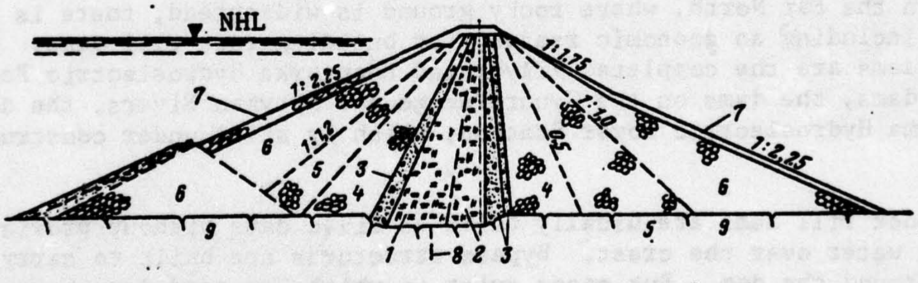

Figure 4-9. The Summersville earth-rock dam (USA).
1 - Clay; 2 - Loamy sand; 3 - Transition zones; 4 - Rock fill up to 25 cm; 5 - Rock fill up to 45 cm; 6 - Rock fill up to 60 cm; 7 - Stabilization with large stones; 8 - Grout curtain; 9 - Rock.

An advantage of this type of dam, as far as construction methods are concerned, is that it permits work to proceed on individual components of the dam independently of one another. Specifically, the rock fill part of the dam can be built at any time of the year, even in winter, while the upstream wedge is built of soft materials during the favorable warm season. The dam on the Oyuur-Yurege River can be cited as an example of this type.

d) Wooden Dams

The building of wooden dams is favored by their relative cheapness, especially in regions where wood is a local construction material, their low sensitivity to temperature fluctuations, and the fact that they can be built rapidly at any time of the year. The disadvantages of wood dams such as susceptibility to rotting is less important in the Far North than in a warm climate, since the annual recurrence of long periods of cold temperatures is not a favorable situation for the growth of fungi which attack wood. The service life of wood can be prolonged also by treatment with preservatives.

Spillways and abutments for dams built on permafrost are usually made of cribwork. Box compartments are loaded with earth, which stabilizes them and makes it possible for them to withstand horizontal forces. The foundation for cribwork spillways may be either rocky or nonrocky permafrost or thawed ground.

Wooden dams may be either fixed or contain spillways, but usually they are built with spillway openings.

The floor of the downstream apron may be built of timbers and concrete (see Figure 4-10). The space below the floor of a spillway is packed with loam laid in layers and carefully compacted to keep out water. But no matter how carefully the packing is done, some space always remains between the floor and the earth; it is not watertight. Water seeps into this space, the foundation materials thaw, temperature conditions are altered, and the structure is deformed. The loam used for loading may also be washed out of the spillway compartments.

By way of example, a wooden dam built near the mouth of the Irelyakh River to flood dredging works is shown in Figure 4-10. Quaternary deposits up to 5 m thick, consisting of loams containing 30-35% gravel, form the surface layer of the foundation. Below this layer there are alluvial fine-grained muddy sands containing plant residues. The bedrock consists of partly consolidated marls interstratified with thin sheets of limestone.

The dam was built with a cribwork sluiceway and abutments. The sluiceway is 20 m wide. A gating structure was built in the sluiceway consisting of braced panels and movable support frames impounding a head of 2.0 m of water. The movable frames are folded against the sluiceway to pass ice and spring floods. The cells of the cribwork are filled with loam up to the level of the floor of the apron, and with a sand-gravel mixture above that level. The floor of the sluiceway in the area of the apron is made of concrete and then covered with squared timbers. The cribwork was built of logs and the grooves were packed with fiber. To assure watertightness and prevent seepage through the sbutment, as the compartments were filled with the sand-gravel material the openings were also faced with planks.

The dam was built in the winter. Because there was no unfrozen loam available, the tooth of the apron and the floor of the sluiceway were made of "cold" concrete.

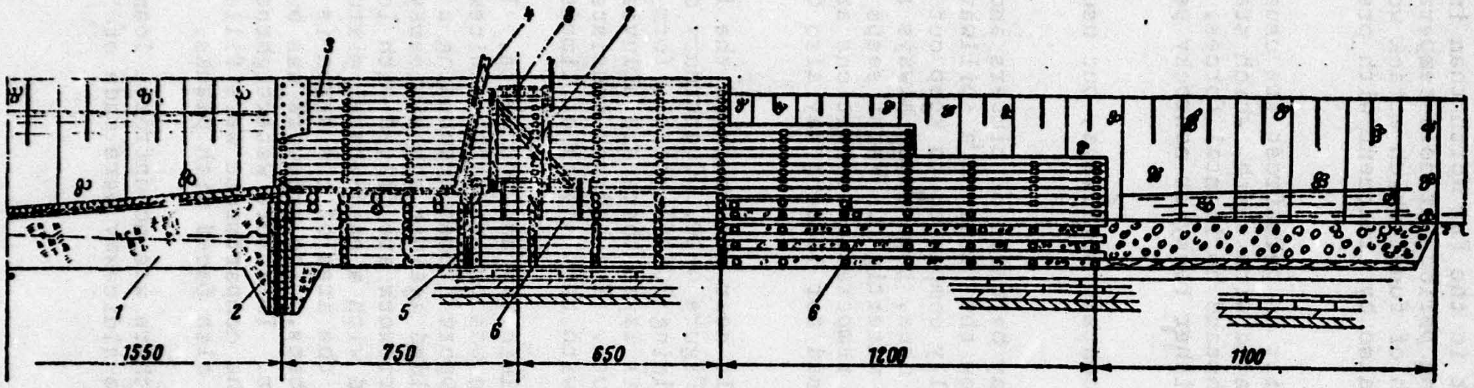

Figure 4-10. Wooden dam near the mouth of the Irelyakh River.
1- Apron; 2 - Apron sheeting; 3 - Abutment; 4 - Gate panel;
5 - Main sheeting; 6 - Cribwork filled with rock; 7 - Buttresses;
8 - Pedestrian bridge.

Chapter 5

BASIC PRINCIPLES OF DAM CONSTRUCTION ACCORDING TO TEMPERATURE REGIME

5-1. CLASSIFICATION OF DAMS ACCORDING TO TEMPERATURE REGIME

Experience with the construction and operation of dams on permafrost both in our country and abroad has shown that the basic principles for building them have now been clearly delineated. These principles are based on temperature conditions in the structures during construction and operation. We have primarily the frozen and unfrozen *methods of construction*, which are governed by the processes of heat and mass exchange that take place between the dam, its foundation, and the environment.

Dams built on an unfrozen basis, which are called unfrozen dams for brevity, allow thawing of the permafrost in the foundation and thawing of the dam body during use, whether they are built of unfrozen materials or of materials which were thawed but freeze during the course of the work. Such dams are made impermeable to water by appropriate antiseepage components (core, blanket seal, or cutoff wall), seepage being allowed within design limits.

According to VSN-08-65 unfrozen dams are recommended for rocky ground or nonrocky ground where the degree of subsidence is less than 0.05 and there are no layers of ice.

When dams are built on a frozen basis (dams which are called frozen dams for short), they are made impermeable to water by freezing the foundation ground and the body of the dam. During operation of these dams thawing of the frozen ground within the core, thawing of the downstream wedge of the dam and of the foundation beneath them as well as seepage are not permitted.

Unfrozen and frozen dams can be built from both unfrozen and frozen materials.

According to VSN-08-65, frozen dams are recommended for construction on nonrocky ground where the degree of subsidence is more than 0.05.

5-2. UNFROZEN DAMS

Construction of unfrozen dams on a permafrost foundation where thawing of the foundation is permitted during operation is quite widespread.

Dams allowing thawing of the permafrost foundation can be built when deformation of the foundation due to thawing does not result in a loss of structural stability. This is true where the ground in the foundation does not contain much ice and its subsidence after thawing does not threaten the stability of the dam, as well as where frozen ground containing ice remaining in the foundation is thin and relatively incompressible or where rock lies close to the surface in the foundation.

In assessing the possibility of building a dam of this type, one should take into account water-management problems, i.e. possible losses of water from the reservoir and through seepage. In the Far North, where water-management problems are solved mainly through regulation of spring floods and surface runoff, even slight losses of water from a reservoir through seepage frequently cannot be tolerated.

In the case of unfrozen dams (where thawing of the foundation is permitted during operation), seepage losses usually are unavoidable and are quite large. Therefore steps are taken to capture and return the seepage flow to the reservoir or to deliver it directly to the consumers.

When the foundations of a dam contain materials which lose their bearing capacity when thawed, dams of this type must not be built. If other construction methods cannot be used where there are foundation materials of this sort, then an unfrozen dam can be built only if certain steps are taken, namely, if the hazardous materials in the foundation of the dam are removed down to rock about which there is no doubt as to its bearing capacity after thawing. Examples of this solution are the Hess Creek Dam in Alaska and the dam on the Novyy River in Yakutia.

Sometimes hazardous materials are removed only under the central part of the dam, where they are removed entirely and antiseepage elements of the dam are attached to better consolidated foundation materials. One can also build thick drainage pads with inverted filters which protect the hazardous foundation materials from mechanical piping.

Materials used in the body of unfrozen dams should be capable of adapting to the deformation of the foundation without forming cracks and otherwise disrupting the integrity of the body of the dam.

When unfrozen dams are built in Canada, if complete removal of ice-saturated material in the foundation is not feasible and bedrock lies relatively close to the surface, weak materials are left in the foundation but special engineering techniques are used to improve their bearing capacity after thawing.

1. A temporary earth dam on the Irelyakh River creates a temporary reservoir for domestic water supply and satisfies the water needs of industrial activities.

In addition to an earth dam the installation includes a water escape with intake and tail race and an ice-retaining structure. The dam is 13 m high and 300 m long at the crest. It was built of local gravelly marly loams. A cribwork cutoff wall was built in the body of the dam. Its compartments were filled with sandy material and loam containing gravel (see Figure 5-1). The upstream slope was stabilized with a layer of stones 1 m thick, and covered drainage was provided at the foot of the downstream slope, running into a toe of stones in the stream-bed area. During construction the height of the dam was raised 2 m to increase the capacity of the reservoir and the crest was widened from 6 to 10 m.

Construction was started in 1957. Up until the spring flood of 1958 a major part of the work on the floodplain areas of the dam was completed: the trench for the cribwork cutoff was dug, the cutoff cribwork was built and the compartments of the cribwork filled with soil, part of the earth was placed for the body of the dam, and a trench was dug for the sluiceway with tail race and intake channel.

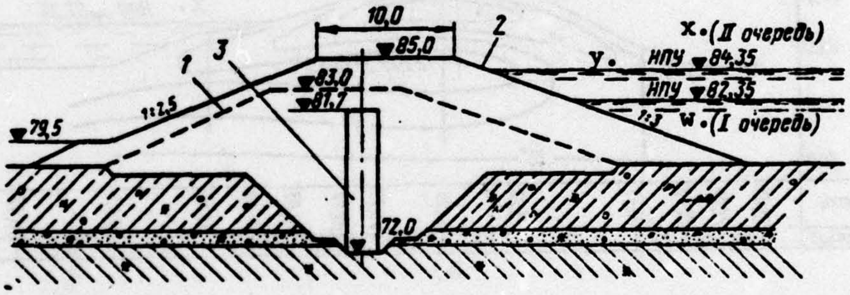

Figure 5-1. Dam for a temporary reservoir on the Irelyakh River. 1 - Contour of the dam as first designed; 2 - Contour of the dam as built; 3 - Wooden cribwork cutoff wall.
Key: w - Stage I; x - Stage II; y - NHL.

After the 1958 spring flood work continued to raise the cutoff wall to the design level and fill the compartments of the cribwork with loam from the sluiceway trench. At the end of August 1958 the river bed was closed and the reservoir began to fill. After the reservoir had filled to the design level, seepage was observed in the foundation of the sluiceway so that it was decided to build a clay tooth and apron.

In 1958 the Institute of Permafrostology of the Siberian Division of The USSR Academy of Sciences undertook observations on the effect of seepage flow on temperature conditions in the body and the foundation of the dam along three imaginary lines.

The observations showed (see Figure 5-2) that filling the body of the dam with unfrozen earth, which accumulates heat, affects the temperature regime of the dam body only during the first year of operation. During operation the main factors affecting the temperature regime of the dam are the temperature of the outside air and the temperature of the frozen ground in the foundation, as well as the warming influence of the reservoir and the seepage flow. The warming effect of the seepage flow on the earth in the body of the dam and the foundation is determined not only by its intensity but also by its size and depth. After two years of operation with very little seepage in the body of the dam located on the floodplain, the unfrozen zones froze up.

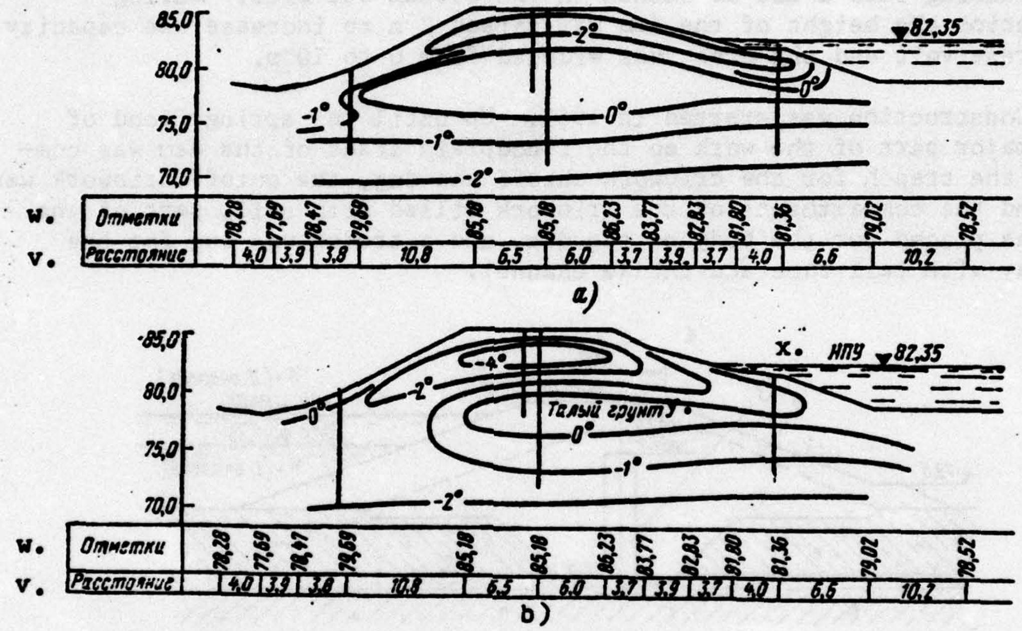

Figure 5-2. The temperature field in the floodplain portion of the dam for a temporary reservoir on the Irelyakh River.
a - Isotherms on June 1, 1960; b - Isotherms on June 1, 1961.
Key: v - Distance; w - Levels; x - NHL; y - Unfrozen ground.

The characteristics of the seepage flow and the temperature regime in the channel portion of the dam evidently were affected by the presence of naturally thawed ground under the channel, the boundaries of which expanded after the reservoir was filled (permafrost degraded at a rate of 1.3 m/year in the channel area).

At the end of the first year of operation of the installation, stronger seepage was observed in the foundation of the sluiceway. The presence of tectonic folds running with the direction of flow evidently promoted this. During the construction period the ground in the sluiceway foundation thawed, fissures which had previously been filled with ice opened and natural seepage paths were formed. As a result of the strong seepage flow the permafrost boundary was lowered 12 m (see Figure 5-3).

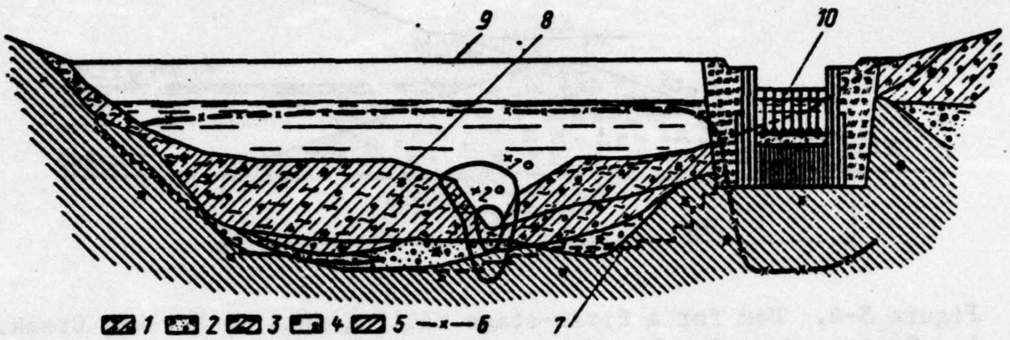

Figure 5-3. Longitudinal section through the dam for a temporary reservoir on the Irelyakh River.
1 - Loams with gravel; 2 - Sandy-gravelly earth; 3 - Muddy loams; 4 - Ice inclusion; 5 - Bedrock; 6 - Position of zero isotherm; 7 - Toe of cutoff wall; 8 - Ground surface; 9 - Crest of dam; 10 - Sluiceway.

The following statements should be made on the basis of experience in planning, building, and operating the dam. The presence of tectonic folds with a high degree of fissuring in the sluiceway foundation, which was not detected during exploration, resulted in major seepage losses from the reservoir. The plan did not include measures to prevent thawing of the foundation materials. The ice-retaining apparatus, which consisted of separate upright poles and ties running lengthwise in the bottom of the sluiceway was designed without taking into account the complex ice conditions in rivers of the Far North.

The decision to build a cribwork cutoff in the body of the dam must also be considered wrong, since it is practically impossible to mechanize the placement and compaction of the ground along the cribwork and in its compartments. Compaction by hand is not very efficient, and, more important, quality control is difficult. In addition, because of differential settlement of the relatively rigid cribwork structure with its vertical walls and that of the earth body of the dam along the interface between the earth and the cribwork, formation of voids, which become seepage paths, was unavoidable.

2. <u>The dam for a first-stage tailing pond on N'yukka Creek</u> (Figure 5-4) was built to create a separation pond for an ore-dressing plant, but during the first three years it was operated only as a reservoir. Excess runoff was bypassed through a lateral self-regulating wood sheet-pile by-wash built on the right bank, with the floor 2 m below the level of the dam crest. The dam is 332 m long with a maximum height (including the tooth) of 17.8 m. The width at the crest is 5.5 m and at the base, 73 m. The slopes are 1:3 on the upstream side and 1:2.5 on the downstream side.

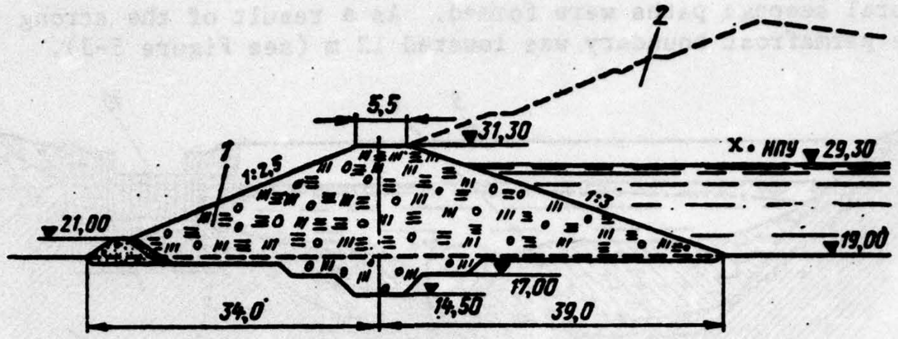

Figure 5-4. Dam for a first-stage tailing pond on N'yukka Creek.
1 - Starter dam; 2 - Dam built by hydraulic fill with tailings.
Key: x - NHL.

The foundation contains diluvial loams with 20-25% calcareous rock gravel under the topsoil to depths of 1.2-1.5 m. Below this there is gravelly-fragmented material from calcareous rocks of Ordovician age with frequent alternation of clay, marl, and fissured argillaceous limestones.

The stream bed contains, at a depth of 3 m, heavily ice-saturated muddy loams and muds, which assume a fluid consistency when thawed. These give way below to sandy-gravelly deposits with a clay filler, which in turn are underlain by muddy materials. The total thickness of the alluvial deposits on the bedrock is about 4.5 m.

In preparing the foundation for the dam the topsoil was removed over the entire site. The heavily ice-saturated muddy earth and the gravelly deposits in the channel area under the tooth of the dam were removed, and the tooth was cut into semiconsolidated marly material. The homogeneous dam body with a volume of 95,500,000 m^3 was built of diluvial loams containing 20-40% sharply angular and flat fragments of calcareous rocks of varying size bedded in a borrow layer up to 2.5 m thick.

During operation of the dam pressure seepage was observed under the foundation. The reservoir was filled by the spring flood of 1965. In September 1965 a gryphon was discovered on the left bank 10 m below the dam. The flow was 5 l/sec and carried no solid particles. The flow subsequently increased to 30 l/sec. In the summer of 1966, when the reservoir was completely full and the head was 14 m, bypass seepage along the left side of the valley intensified, and seepage also began on the right side. In order to reduce the water losses, a covering of earth consisting of tailings from the ore-dressing plant was applied to the juncture of the dam with the left bank at locations where leaks were thought to occur. This appeared to be enough to stop the seepage entirely. But a leak under the right-bank junction intensified and became a threat to the integrity of the structure. A special wood-concrete curtain was built to trap the seepage flow. Holes 500 m in diameter were bored from the dam crest to a depth 3 m below the toe of the dam. Burlap bags filled with wet concrete were packed into the holes. The holes were bored one after the other in a checkerboard pattern. Creation of this concrete cutoff wall 15 m long made it possible to stop the leaking of water from the reservoir entirely.

The reason for the seepage under the dam was a change in the thermal regime of the foundation material. This is unavoidable when unfrozen dams are built. During the construction period the foundation of the dam (after removal of the topsoil and prior to covering it with loam) remained exposed for almost two summer seasons. During this period the fissured rock of the foundation thawed and the cracks opened. In addition, in preparing the foundation the topsoil was removed from a large area around the dam site. Because of this areas above and below the dam remained without a loam covering, and this promoted thawing of the foundation. Also, when the topsoil was removed, fissured marly deposits were uncovered in places.

3. Dam for the second-stage tailing pond on N'yukka Creek. This dam is 26 m high and 320 m long at the crest. It was built in 1971-1972. The dam, with a volume of 295,000 m^3, is the starter dam for the reservoir to make subsequent hydraulic fill possible as tailings from an ore-dressing plant are stored (see Figure 5-5). The foundation contains a layer of diluvial loams 0.4-1.8 m thick with a 30-40% content of gravel and rock fragments. The diluvial formations are gradually transitional to the bedrock, which consists of fissured limestones. Cracks 1-5 cm wide are filled with clay and ice. The limestones alternate with layers of marly clay. Alluvial deposits are found in the stream bed area: muddy and gravelly materials.

The supporting prism of the dam was built of consolidated and semiconsolidated marly limestones and dolomites, which were packed down by the traffic delivering them. A blanket seal was made of gravelly diluvial loams with a moisture content of 12-16%.

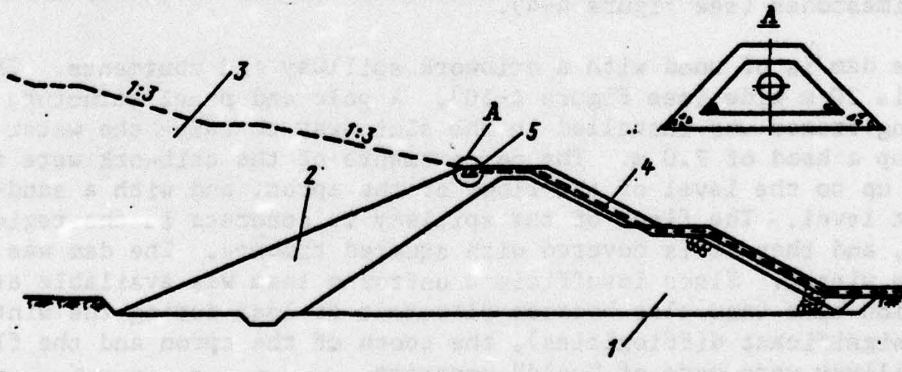

Figure 5-5. Dam for the second-stage tailing pond on N'yukka Creek. 1 - Marly semiconsolidated material; 2 - Gravelly loam; 3 - Hydraulic-fill portion of the dam; 4 - Pipes to carry off drainage water; 5 - Drainage.

During the summer the loam was dug from the borrow pit by cutting layers away with bulldozers. Because it was necessary to hold the spring flood at the upper level of the dam, it was put in place in the spring when the outside air temperatures were negative. The marly earth in the borrow pit was successfully mined by first breaking it up to a depth of 1.4 m with a "Caterpillar" tractor ripper. With positive daytime air temperatures (+2 to +6°C) the broken ground became somewhat plastic as it was mined, transported, and spread, and it was well compacted with MAZ-525 trucks. About 24,000 m^3 of loam were placed in this fashion. Seventy-two samples showed density values from 1.40 to 1.75 tons/m^3 with an average value of 1.58 tons/m^3.

4. Dam for a tailing pond on a tributary of the Irelyakh River. This was designed in a similar manner to the dam for the first-stage tailing pond on N'yukka Creek described above. The dam was 10 m high and 60 m long at the crest. The dam was built entirely during the summer. Earth was dug as it thawed in a borrow pit located alongside the dam. It was then transferred to the construction site, smoothed out and packed by bulldozers. The dam functioned as a reservoir impoundment for three years.

The wooden water escape of this dam was built during the winter. Because of low-quality workmanship considerable seepage was found where the water escape structure joined the foundation during the first year of operation, especially under the spillway. Repair work was done during the first summer, completely eliminating the seepage.

5. A dam for flooding dredging works was built near the mouth of the Irelyakh River in 1960. The Quaternary deposits in the foundation (up to 5 m thick) consisted of a top layer of loams containing 30-35% gravel, and a lower layer of alluvial fine-grained muddy sand containing plant residues. The bedrock consisted of marls and interstratified thinly laminar limestones (see Figure 4-4).

The dam is of wood with a cribwork spillway and abutments. The spillway is 20 m wide (see Figure 4-10). A pole and panel structure carried on swinging frames was installed in the sluiceway to raise the water level. It backs up a head of 2.0 m. The compartments of the cribwork were filled with loam up to the level of the floor of the apron, and with a sand-gravel above that level. The floor of the spillway is concrete in the region of the apron, and then it is covered with squared timbers. The dam was built during the winter. Since insufficient unfrozen loam was available at the construction site (and also because placement of loam during the winter involves significant difficulties), the tooth of the apron and the floor of the spillway were made of "cold" concrete.

6. The dam on the Kadykchan River is 593 m long at the crest, impounding a 6.7 m head of water with a maximum height of 9 m. The dam was placed in operation in September 1968. The slope of the upstream face is 1:2.5, and that on the downstream side is 1:2. A bypass was built at the left bank face of the dam to carry floodwaters away. It consists of an

open canal dug into the bedrock, which is sandy-argillaceous shale. A weir equipped with a 8x4 m gate was installed in the canal (in line with the axis of the dam) (see Figure 5-6).

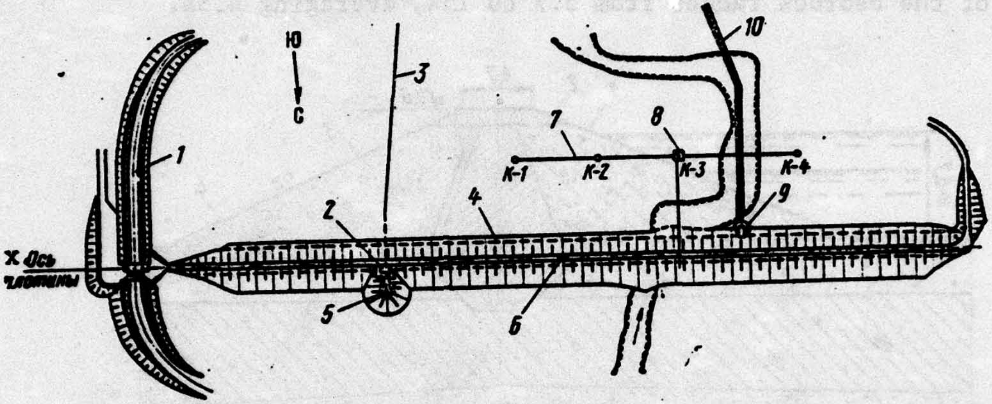

Figure 5-6. Installation on the Kadykchan River.
1 - Bypass; 2 - Water intake; 3 - Water line; 4 - Drain; 5 - Intake head; 6 - Dam; 7 - Captation gallery; 8 - Return pumping station; 9 - Drainage shaft; 10 - Gravity-flow pipe.
Key: x - Axis of dam.

Gravelly materials from 3.5-5.0 m thick with a percolation coefficient from 400-4000 m/day occupy the dam site. Below this material are loams underlain by clay shales, interrupted in the central part of the alignment by andesites. The loams vary in thickness from 2.5-7.0 m; the percolation coefficient of the loams and clay soils is no more than 0.02 m/day. A thawed area no thicker than 5.0 m remains under the channel. The loams and underlying material are in the permanently frozen state. The core of the dam was made of loamy materials; the upstream wedge, of sandy-gravelly materials, and the downstream wedge of gravelly material (see Figure 5-7). The tooth cuts completely through the gravelly material into the foundation of the dam and is buried in the bedrock to a depth of 0.75 m. At the juncture with the right bank it was planned to cut a trench for the tooth into the bedrock a distance of 2.5 m.

Dam on Ponneuregen Creek. This dam, which is 680 m long and impounds a head of 16 m was built on the Chukchi Peninsula to provide water for the Bilibino Nuclear Power Station. The climate in the region is very severe: the mean annual temperature is -12°C and winter lasts 7-8 months. The average monthly temperature in January is 36.4°C, and in July +13.3°C. The average absolute minimum temperature is -52°C and the absolute maximum, +28°C.

The bedrock in the foundation of the dam (at depths of 1.5-8 m) consists of mixed layers of clay and sandy-clay shales and sandstones with percolation coefficients from 0.5-2 m/day. The upper part of the bedrock at depths of 0.4-2 m has been weathered to large and medium gravel. The ground is permanently frozen (with temperatures of -3 to -5.8°C). The dam foundation contains materials with lenses, pockets, and layers of ice 2-4 m thick as well as wedge-shaped veins of ice down to 15 m. The ice content of the bedrock ranges from 3.7 to 13%, averaging 4.5%.

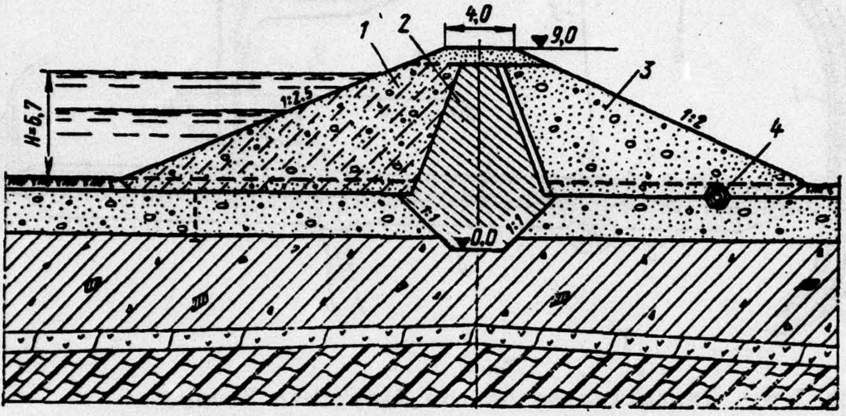

Figure 5-7. Cross-section of dam on the Kadykchan River.
1 - Upstream wedge of sandy-gravelly material; 2 - Core of loam;
3 - Downstream wedge of gravelly material; 4 - Drain.

Ponneuregen Creek is a left tributary of the Kaperveyem River. It is 22.4 km long with a catchment area of 29 km^2. The creek is 4.7 m wide and up to 1.5 m deep. Stream flow is as follows:

Frequency (%)	0.1	0.5	1.0	3	5	10
Discharge (m^3/sec)	111	89	80.3	72.6	47.8	47.4

The installation includes an earth dam, a bottom intake-discharge, a sluiceway and a pumping station to return seepage water to the reservoir.

The bottom water outlet is combined with the water intake and consists of a supply canal and a concrete control tower (horizontal dimensions 5.6x6 m and height with superstructure 23 m). The tower is located in the upstream prism of the dam. Gravity-flow water-intake pipes 1 m in diameter and 59 m long, unitized with concrete, are installed in the body of the dam. The escape pipes terminate in an overhang.

The body of the dam is built of sandy-gravelly material; the tooth and core of loam (see Figure 5-8). The dam is 6 m wide at the crest. A concrete plate 0.7 m thick was installed at the juncture between the tooth and the foundation. The tooth of the core was cut into the bedrock to a depth of 1.0 m. An inverted filter layer of sandy material 1.0 m thick was installed along the downstream slope of the core. The upstream slope was stabilized with prefabricated reinforced concrete plates 20 cm thick placed on a bed of gravel.

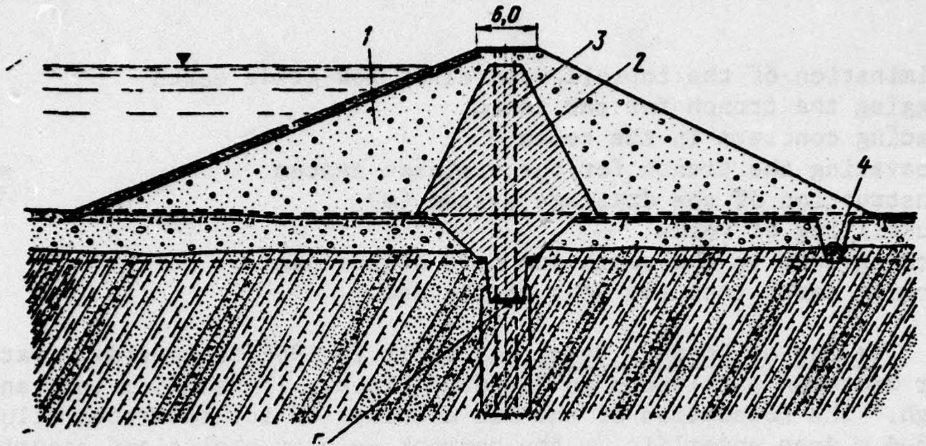

Figure 5-8. Dam on Ponneuregen Creek.
1 - Sandy-gravelly material; 2 - Core of loam; 3 - Stage II grouting holes; 4 - Drain; 5 - Reinforced concrete plate.

In order to prevent seepage flow from emerging from under the downstream slope, a drain system was installed in the form of reinforced concrete perforated pipes sunk in the bedrock of the foundation a distance of 3-6 m.

The bedrock in the foundation of the dam has percolation coefficients of 0.5-2.0 m/day after thawing. Since water losses through seepage in this case would be too high, it was decided to grout the foundation of the dam. The grouting work was planned in two stages. In the first stage (before construction of the dam) cement would be injected for purposes of adhesion to a depth of 5.0 m. The second stage of grouting -- deep grouting to 15 m -- was planned for later during the operation of the dam when seepage discharge reached 40 l/sec.

According to the plan the control value for the density of the loam used in the body of the dam was set at 1.6 tons/m^3, and that for the sandy materials at 2 tons/m^3. To achieve this their moisture content was not to exceed 18-20% for the loam and 12-15% for the sandy material. The loams in the borrow pits had a moisture content of up to 50% and were in a fluid state. In working the borrow pits special engineering operations were done for purposes of melioration, and then the earth was predried in mounds.

The following work was done to build the dam (thousands of cubic meters):

Elimination of the topsoil, removal of low-grade earth	101.8
Digging the trench for the tooth	34.3
Placing concrete in the tooth	1.23
Excavating the trench for the drainage system	7.5
Construction of the drainage system (m)	624.0
Backfilling of voids	26.8
Packing loam in the tooth and core of the dam	168.0
Packing earth into the body of the dam	420.0

8. The dam on Zharkiy Creek was built in 1967 to provide a water supply for Orotukan settlement in Magadan Oblast. It is 315 m long and 16.6 m high. The foundation of the dam consists of ice-saturated alluvial deposits 3-4 m deep underlain by the bedrock -- clay shales and sandstones. The body of the dam was built of sandy-gravelly materials with a blanket seal of loam. The blanket seal was attached to the bedrock with a tooth, which cuts completely through the alluvial deposits.

A by-wash with a concrete header was built around the dam along the left-bank slope. A drainage system was built under the downstream slope of the dam. Plans called for capturing all seepage flow and pumping it back into the reservoir, and a special pumping station was built for this purpose.

9. A dam was built on the Kazachka River in 1969 to create a water-supply reservoir for the town of Anadyr' on the Chukchi Peninsula. A bypass was built around the dam at the right bank.

The surface layers at the dam site consisted of peat and silty-muddy sandy loams to a depth of 2 m. Beneath this layer were gravelly materials underlain by dense loams. The deposits covering the loams were from 3-6.5 m thick. The loamy sands and gravelly materials were heavily impregnated with ice and contained lenses of pure ice. At the junction between the dam and the bank, lenses of buried ice up to 5 m thick were found as well as wedge-shaped, polygonal vein ice in a network pattern measuring approximately 15 x 15 m. In preparing the foundation for the dam the peat and loamy sand were removed entirely with bulldozers as they thawed.

The central part or expanded core of the dam was built of fine-grained sand and loamy sand, and the lateral prisms of sandy-gravelly material and stones. The dam core was attached to the foundation by a tooth, which was cut into the compact loams. The plan called for the tooth to be made of loam, but as the work progressed the slopes of the tooth trench began to thaw. To prevent everything from turning to mud, the ditch was backfilled hydraulically with fine-grained sand.

5-3. EXAMPLES OF CONSTRUCTION OF LARGE UNFROZEN DAMS

a) The Dam of the Vilyuy Hydroelectric Power Station on the Vilyuy River

Experience in building the Vilyuy Hydroelectric Power Station is of great interest: this large plant (first-stage power 308 MW and full power about 650 MW) is the first in the world to be built under extremely severe natural climatic conditions. The construction site is located in the zone of continuous permafrost, which thaws no deeper than 0.5-0.8 m during the short summer season. The cold winter with a long period of low temperatures, which reach -60°C or below, creates conditions which are difficult for construction activity. Until construction started on the Vilyuy Hydroelectric Power Station there had been no experience in planning and building such a large facility under such conditions either in this country or abroad.

Abroad, particularly in Sweden and Canada, building the body of dams with rock fill ordinarily is limited to times when air temperatures are not below -30°C, and it is recommended that antiseepage elements be built only during the warm season of the year or, in exceptional cases, when temperatures are not below -5° to -10°C.

The facilities at the Vilyuy Hydroelectric Power Station include (Figure 5-9): a fixed rock fill dam, an intake canal, a sluiceway equipped with a segmented gate, a bypass, a water receiver, a power station building with pressure-head conduits and tail race, and an enclosed distributor.

The rock fill dam (Figure 5-10) is 74.5 m high and about 600 m long at the crest. The slope ratio varies with height. Berms are built on the downstream slope every 10 or 15 m vertically. These are used as temporary access lanes and permanent roads.

The dam impounds a reservoir of 36 km^3, about 450 km long with a maximum depth 70 m.

The antiseepage element of the dam is a blanket seal which turns into a core at the upper levels. The blanket seal is made of loamy gravelly materials on a two-layer inverted filter of 0-40 mm and 0-150 mm crushed stone. The side of the blanket seal facing the reservoir is covered with a filter of mixed sand and gravel. In the region of changing water level this filter is made of 0-40 mm crushed stone.

The foundation of the dam contains strong diabases, which on the lateral slopes are covered with unconsolidated deposits 1.5-2 m thick. The ground is permanently frozen and the permafrost thickness is more than 350 m.

The blanket seal was attached to the rock foundation with a lightly reinforced concrete plate and a grouting tunnel. Before the reservoir was filled, the foundation in the unfrozen area under the stream bed was grouted from the tunnel. The dam foundation in the area of the valley slopes was grouted as the permafrost degraded and rock thawed.

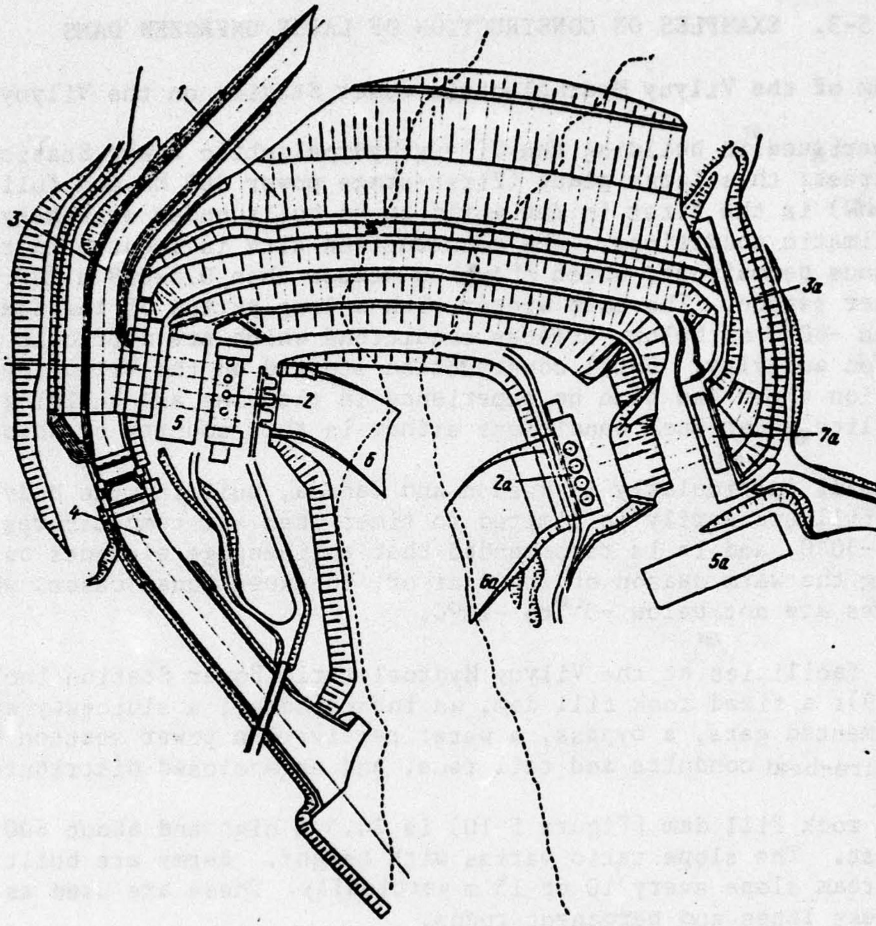

Figure 5-9. Vilyuy Hydroelectric Power Station on the Vilyuy River. 1 - Dam; 2 - Power station building; 3 - Intake channel; 4 - Sluiceway; 5 - Covered distributor; 6 - Tail race; 7 - Water receiver (the items marked with "a" belong to the second stage).

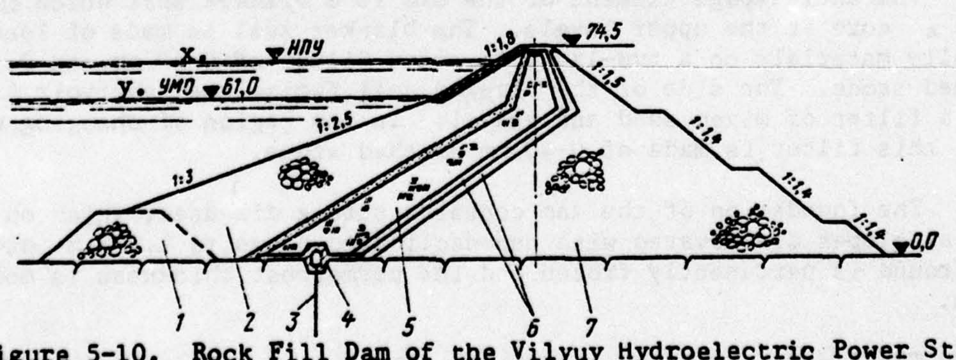

Figure 5-10. Rock Fill Dam of the Vilyuy Hydroelectric Power Station. 1 - Toe; 2 - Fill; 3 - Grout screen; 4 - Grouting tunnel; 5 - Blanket seal; 6 - Inverted filters; 7 - Supporting prism. Key: x - NHL; y - LWL.

A bypass was dug into the rock, and the first section ahead of the intake serves as a supply channel for the hydroelectric power station. The bypass was designed to handle a specific discharge 160 m^3/sec.

Work involved in building the dam can be judged from the following figures:

Fill for the body of the dam	3955 thousand m^3
Loam placed	630
Filters	420
Facing with concrete and reinforced concrete	49

It was difficult to plan and organize the work. The main reason was the extremely uneven flow of the Vilyuy River during the year. Most of the annual discharge (84%) occurs during the spring and summer, when maximum rates of discharge reach 13,000 m^3/sec. High water also occurs at the end of summer and in early fall, with peak flows reaching as high as 2700 m^3/sec. These are due to rains and thawing of the permafrost. During the winter river flow drops to 4-2 m^3/sec. These factors made it necessary to do the bulk of the work of raising the dam during the winter.

A diversion trench was built in the left bank to handle flows of as much as 8400 m^3/sec which occurred with a 5% frequency during the construction season. Work could begin on the dam in the area of the diversion trench after it was closed, but because of hydrological conditions this became possible only in December (taking into account the accumulation of winter flow in the newly formed reservoir). Under these conditions in the winter of 1964/65 the channel portion of the rock fill dam was built to a height of 40 m and the loam blanket seal to a height of 25 m. The spring flood in 1965 was passed through the diversion trench, and between the time it was closed (on December 8, 1966) and the spring flood of 1967 (i.e., five months later) the dam was raised to a height of 55 m in the trench area.

In the winter of 1964-65 300,000 cubic meters of loam were placed for the blanket seal and in the winter of 1966/67, 220,000 cubic meters. This was possible because facilities were developed and built at the site for packing cohesive earth into high-quality fill at low air temperatures (-35 to -40^{o}C). The technology was based on the accumulation of solar heat by the loam during the summer sufficient to maintain the loam in a thawed state until placement in the blanket seal had been finished. In spite of the fact that such a large amount of ungraded rock fill was placed during the winter without special compaction, settlement of the dam after four years was no more than 4-6.5%, and about half this settlement occurred during the first 2-3 months after filling operations.

After the particularly severe winter of 1965/1966 an attempt was made to place loam in the blanket seal using the technology developed at the site at air temperatures as low as -50 to -55^{o}C. But this attempt was not successful. The problem was not with the winter earth-placement technology which had been developed, but with other causes: in particular, the frequent breakdown of machinery at the low temperatures which prevailed, the numerous days when all were excused from work, power failures, etc. All this caused interruptions which lasted 3-5 or even 10 days at a time. With such long interruptions the layers of loam placed earlier were thoroughly chilled and froze completely. When the next layer was laid it was not possible to return the layer laid earlier to the plastic state, and the new layer froze at the interface with the lower layer before it could be compacted.

An air temperature of -40°C appears to be the lower limit for proper construction of water-retaining components from cohesive earth during winter. This limit is based, however, not on construction methods per se, but on the lack of machines (cranes, shovels, etc.) capable of working at temperatures lower than that.

In planning the dam it was assumed that the temperature regime of the dam would be determined by the temperature regime of the waterproof blanket seal, and that the temperature at the surface of the blanket seal would always be positive because of the enormous amount of heat accumulated in the reservoir. The rock fill supporting prism in turn would be cooled by the outside air, since the mean annual air temperature was negative. Since the air within the body of the dam was warmer and lighter than the outside air, currents would be generated within the rock fill.

Such a model obviously is very simplified. Even during the operational period (when conditions have stabilized) the upper range of temperatures in the blanket seal will vary significantly during the year. When the reservoir drops in winter, the upper portion of the blanket seal (about 25% of the total height) is exposed and freezes. In addition, one cannot ignore the effect of the snow cover on temperatures in the rock fill prism during the winter, and the effect of precipitation during summer.

Another important factor governing temperature conditions in the dam is the blanket seal. When the loam is placed during the winter, the blanket seal usually remains unfrozen until compaction is complete. Then it freezes and assumes a temperature close to the temperature of the outside air. In areas of the blanket seal placed during the summer, of course, the temperature is positive, but during the winter before the reservoir fills, it remains exposed and also freezes.

In our opinion the temperature regime of the foundation and body of the dam differs somewhat from that assumed in the planning hypothesis.

The temperature field in the body of the dam is shown in Figure 5-11, where the uneven temperature distribution is apparent. The blanket seal is located in a positive temperature zone, even though during the construction period it was partially frozen. At the same time, temperature conditions at the upper levels of the blanket seal are complex. A zone of deep cooling is seen under the downstream slope of the supporting prism. There temperatures in the rock fill drop to -25°C. Intense thawing of the left-bank slope occurs in the region of the diversion trench. The explanation for this lies in the fact that as the spring floodwaters and summer discharge passed through the trench, the rock was thoroughly warmed, cracks were opened, and seepage water began to pass through them bringing additional heat. Thawing was slower on the right bank (see Figure 5-12).

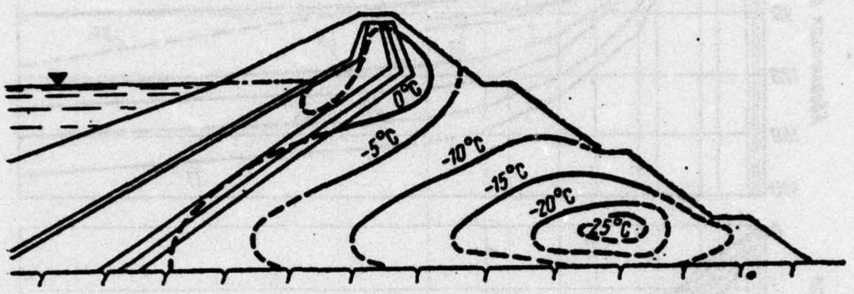

Figure 5-11. The temperature field in the Vilyuy Dam.

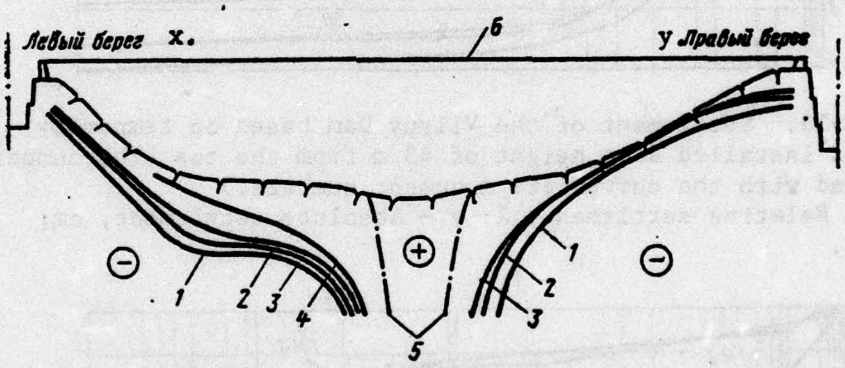

Figure 5-12. Thawing of the foundation of the Vilyuy rock fill dam. 1-4 - Zero isotherms based on the situation in May 1971, April 1969, November 1968, and June 1968, respectively; 5 - Outline of the thawed area below the channel prior to construction of the dam; 6 - Crest of the dam.
Key: x - Left bank; y - Right bank.

Curves showing settlement of the dam are presented in Figures 5-13 and 5-14. During the first four years the total settlement of the rock fill dam was 6.5% of the height of the dam.

b) The Dam of the Khantayka Hydroelectric Power Station on the Khantayka River.

The installation at the Khantayka Hydroelectric Power Station includes a rock fill dam in the river channel, earth dams on the left and right banks, water escape structures, and the hydroelectric station building with an underground machine room [65, 76].

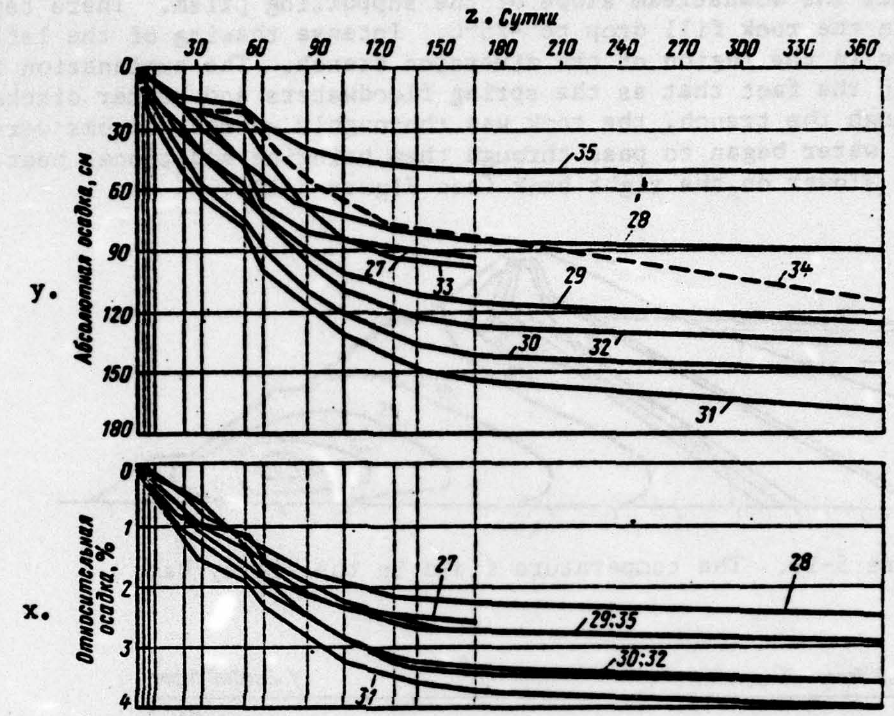

Figure 5-13. Settlement of the Vilyuy Dam based on temporary monuments installed at a height of 43 m from the toe (The numbers associated with the curves are monument numbers.)
Key: x - Relative settlement, %; y - Absolute settlement, cm; z - Days.

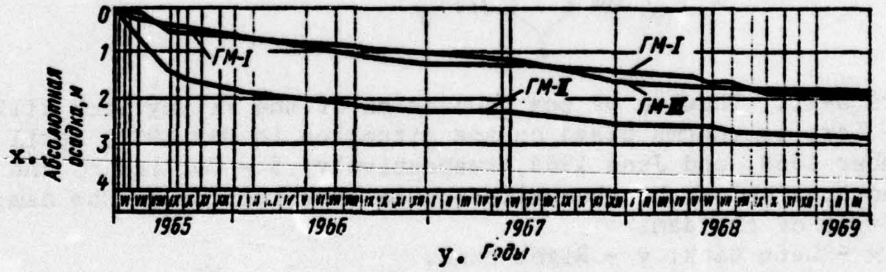

Figure 5-14. Absolute settlement of the Vilyuy Dam based on depth markers. (The numbers associated with the curves are marker numbers.)
Key: x - Absolute settlement, m; y - Years.

The Khantayka River Basin is located above the Arctic Circle in the forest tundra zone with large numbers of lakes and bogs. Permafrost is not continuous in the vicinity of the construction site. The depth of seasonal freezing and thawing is no more than 2.5 m.

The climate in the region of the construction site is subarctic, with a long severe winter and a short summer. The mean annual air temperature is -8°C.

The rock fill channel dam (Figure 5-15) 65 m high creates a reservoir of 23.5 km^3 of water. The antiseepage element is the core made of moraine materials. The upstream and downstream supporting prisms are built of rock from excavations made for the other facilities. The core of the dam is linked to the foundation by a tooth, which contains a gallery for grouting. The grout curtain in the channel area is up to 20 m deep.

There are depressions on the right bank and left bank floodplains, which have been shielded by protective dikes. The left-bank earth dam is 1800 m long and 11 m high, and is a direct continuation of the channel dam. The foundation of the left-bank dam contains dolerites, as well as thawed and permanently frozen moraine materials. The core of the dam consists of moraine materials, and the supporting prism of gravelly materials.

The right-bank earth dam (Figure 5-16), which is 2.5 km long with a maximum height of 33 m, is located a distance of 4 km from the channel dam. It also contains a core of moraine materials and supporting prisms of gravelly materials with a loamy sand filler. Its foundation consists of solid and semiconsolidated bedrock covered by unfrozen clay, loam, and loamy sand of lacustrine-glacial and lacustrine-bog origin. The foundation contains unfrozen and water-bearing materials, but in particular areas there are lenses of permafrost of massive texture which are relatively thin (no more than 4-8 m) as well as a lens containing stratified ice up to 15 m thick with its long dimension paralleling the axis of the dam for a distance of about 200 m.

In preparing the dam foundation permafrost with massive texture was left in place in accordance with the plan regardless of its ice content and the lithology of the rock. Only a portion of the silty loams and varved clays within permafrost lenses containing stratified ice were removed. The maximum thickness of earth removed was 12 m. Part of a lens of clay permafrost containing stratified ice from 5.5 to 7.5 m thick was left in place under the dam. The dam in this area is 26 m high, and it is about 11-13 m high in the area of the lens with massive texture.

The materials intended for use in the antiseepage elements were moraine materials and loams of glacial origin, which were thinly bedded. The materials were also very waterlogged in their naturally bedded state. These circumstances required special digging techniques. As in the case of the construction of the Vilyuy Hydroelectric Power Station, the earth in the borrow pits was dug with bulldozers a layer at a time as it thawed, carried away and piled up to lose moisture. Before being stripped away, the surface layers of the mounds in which the earth had been stacked and treated with salt were warmed with an electric heater.

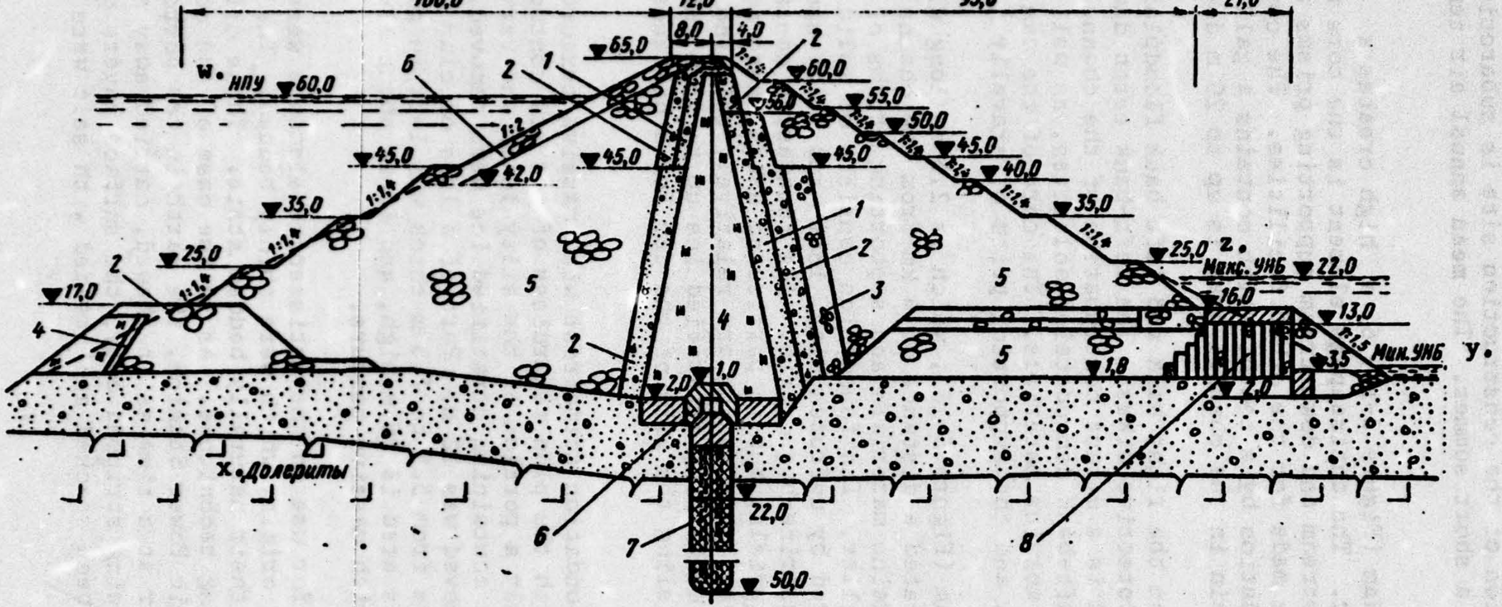

Figure 5-15. Channel dam of the Khantayka Hydroelectric Power Station on the Khantayka River.
1, 2, 3 - Filters; 4 - Core of Moraine material; 5 - Rock mass; 6 - Grouting gallery; 7 - Grout curtain; 8 - Dam facing for passage of floodwaters.
Key: x - Dolerites; y - Minimum lower pool elevation; z - Maximum lower pool elevation; w - NHL.

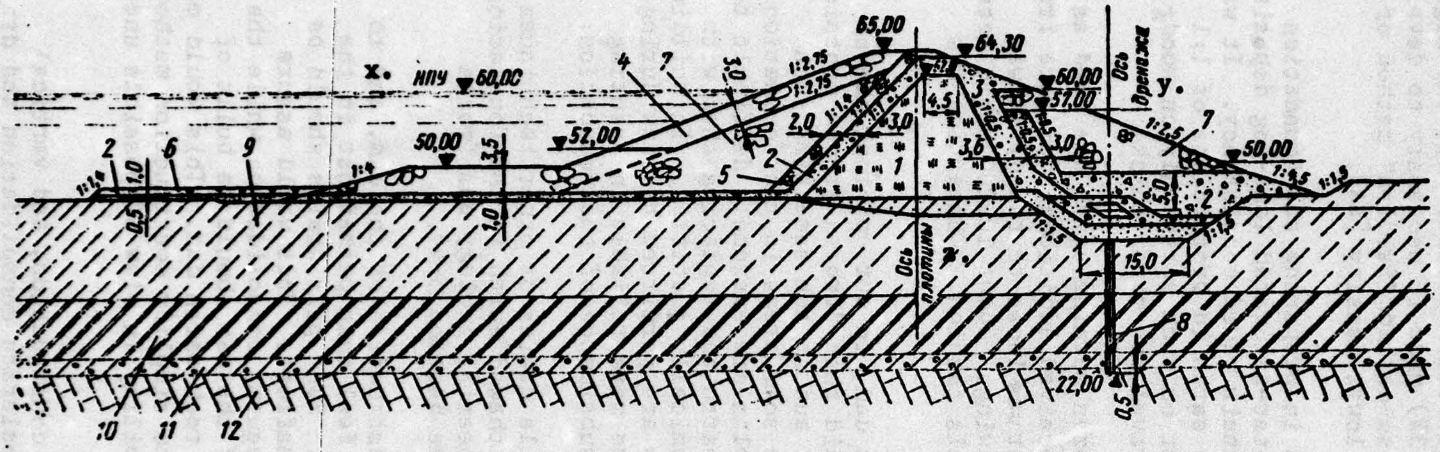

Figure 5-16. The right-bank dam of the Khantayka Hydroelectric Power Station on the Khantayka River.
1 - Core of moraine material; 2, 3 - Gravelly-sandy material; 4, 7 - Rock mass; 5 - Gravelly material; 6 - Sand; 8 - Drainage holes; 9 - Loams of lacustrine glacial origin; 10 - Varved clays; 11 - Moraine; 12 - Limestone.
Key: x -NHL ; y - Drainage axis; z - Dam axis.

The rock fill channel dam was built mainly using techniques for building water-retaining structures of cohesive earth, which were developed and used in building the Vilyuy Hydroelectric Power Station. But the high moisture content of the borrow material (15.3%) made it necessary to develop a new technique for placing it based on the so-called "Swedish" method of packing excessively wet cohesive earth with lower density [44].

Serious difficulties were encountered in preparing the foundation for the left-bank dam, since the unfrozen water-saturated lake-bog deposits had practically no bearing capacity due to their fluid consistency. It was possible to work the ground after adding dry earth in proportions of 1:1 and 1:2. Excavating equipment cound maneuver on such earth only on wood-metal planks placed on a fill of sand and gravel one meter thick.

In preparing the core foundation, frozen moraine earth was used as a cover layer under timbers. This material received additional moisture from the ground water and froze. Since this layer was loose-textured, it was necessary to compact it after electrical thawing. About 10,000 cubic meters of frozen moraine material were thawed in this fashion. Compaction was done with caterpillar tractors.

The planning steps for the right-bank dam, in our opinion, are no different in their sequence. It has been said that "frozen ground containing stratified ice should be removed completely" and at the same time such materials up to 7.5 m thick have remained in some areas in the foundation of the dam. The techniques adopted in the plan are extremely difficult to carry out. Removal of large quantities of earth and replacing them with better-quality materials is not always economically justified. In building unfrozen dams where the foundation materials are permitted to thaw during use, other solutions may also exist which are more reliable in assuring stability of the dam in the face of uneven subsidence of the foundation.

Special drains in the form of sand pile drains or other techniques are used to improve the bearing capacity of thawed earth in the foundation of a dam in Canada where conditions of engineering geology and frozen ground are similar to those in this case (see Section 5-4).

In order to simplify and speed up construction of the dam, and to reduce its cost and improve its quality, we feel that permafrost in the foundation of the dam should not be removed. Sand pile drains should be installed in areas containing lenses of permafrost. This would assure drainage of the ground as it thaws and subsequent consolidation while the bearing capacity is maintained. The body of the dam should be built of sandy and gravelly materials, possibly as a rock-earth dam. This would make it possible to build it during the winter, to simplify construction methods, and to assure necessary stability and reliability in view of possible uneven subsidence of the foundation materials.

The plan was revised somewhat during construction, and vertical drains were built in particular areas to accelerate consolidation and drainage of the foundation materials below the downstream prism. This significantly increased the reliability of the structure [65].

c) Dam of the Serebryanskaya Hydroelectric Power Station on the Voron'ya River

This dam, which is 78 m high and 182 m long at the crest, creates a reservoir of 4.3 km^3. The installation includes a dam, a hydroelectric station, and a sluiceway structure. The core of the dam (maximum width 70 m) was built by filling sandy moraine material into the water; the lateral prisms were built of broken rock, and the transitional zones of a sand-gravel mixture (see Figure 5-17). The dam was built north of the Arctic Circle but outside the permafrost zone, even though the air temperatures in winter dropped to -48°C, rising in summer to +38°C.

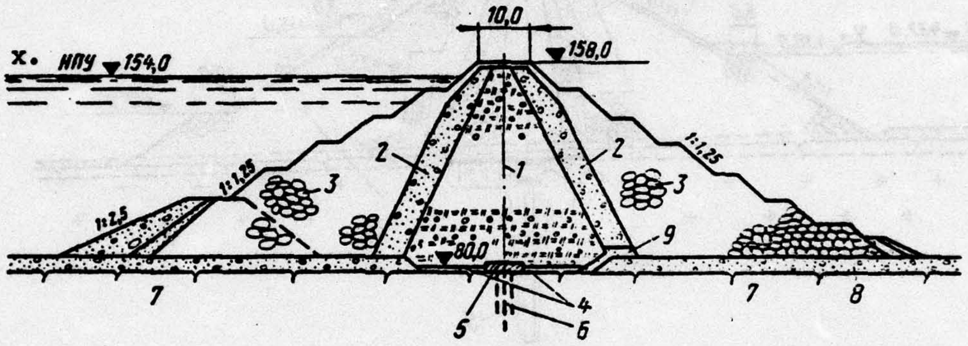

Figure 5-17. Rock-earth dam of the Serebryanskaya Hydroelectric Power Station on the Voron'ya River.
1 - Core of loamy sand moraine material; 2 - Sand-gravel material;
3 - Rock; 4 - Clay; 5 - Concrete plate; 6 - Grout curtain;
7 - Granite; 8 - Moraine-gravel material.
Key: x - NHL.

The particle-size distribution of the materials used in the core of the dam was as follows:

Particle size, mm	Content, %
>100	13
100 - 10	25
10 - 2	14
2 - 0.5	11
0.5 - 0.25	8
0.25 - 0.05	17
0.05 - 0.002	8
<0.002	4

During the winter the water temperature in the pool dropped to +6°C, and fill work continued at air temperatures down to -25°C. The surface of the pool was protected from the extreme cold with polystyrene foam [95]. A 650 kw electric boiler plant was used to heat the water in the pools, and shore ice was thawed by blasting with hot gases of a turbojet device. In order to lower the freezing temperature of the earth, the surface was flooded with a calcium chloride solution.

d) Dam of the Kolyma Hydroelectric Power Station on the Kolyma River

This dam, which is about 125 m high and 750 m long at the crest, impounds a reservoir of 14.6 km^3. A sluiceway 950 m long and 100 m wide is planned for an excavation in the rock on the left bank of the river. The sluiceway in use is designed to pass a flow of 17,500 m^3/sec occurring with a frequency of 0.01%.

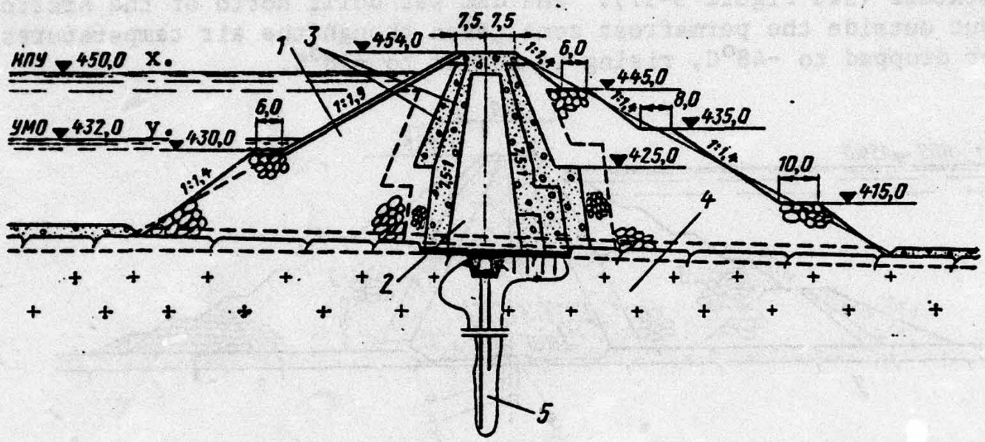

Figure 5-18. Rock fill dam of the Kolyma Hydroelectric Power Station on the Kolyma River.
1 - Rock fill; 2 - Core; 3 - Sand-gravel filter; 4 - Granite; 5 - Grout curtain.
Key: x - NHL; y - LWL.

The dam is being built under severe continental climatic conditions with a moderate warm summer and a very cold winter. The mean annual air temperature is -12°C. Minimum temperatures in winter may reach -60°C, with maximum temperatures in summer of +36°C.

As far as flow is concerned, the Kolyma has a very pronounced period of high water in the spring, rain floods in summer and fall, and a long period of low water in the winter. Rain floods during summer and fall number from 1 to 6. From 95-97% of the annual runoff occurs during the five warm months, with only 3-5% in the 7-8 winter months. Flow is extremely low (0.27-0.59 m^3/sec) at the end of winter (March-April).

The foundation materials of the structures are permanently frozen and thaw to depths of 1.0-1.1 m. The thickness of the frozen ground in the slopes of the river banks varies: it is thickest on the left bank (up to 200-300 m), and not as thick on the right bank (from 25-150 m). The thawed area under the river bed extends below the permafrost.

The structures of the installation are built on rock. Granites, which underly the dam alignment, are covered almost everywhere with unconsolidated deposits and crop out only here and there on the bottom of the valley. The overlying unconsolidated deposits are thin. Cohesive earth borrow pits for the antiseepage elements are located 5.5 km from the dam alignment. The mean composition by weight of the borrow material used in the dam core is as follows: clay 16%, silt 21%, sand 23%, small gravel 15%, and large gravel 25%.

The physical properties of these materials are shown in Table 5-1. The borrow materials can be used in the watertight components of the dam if previously dried somewhat.

Table 5-1.

Principal physical properties of earth used in the core of the dam for the Kolyma Hydroelectric Power Station

Property	Naturally bedded earth	Disturbed (mixed) earth	
		Coarse material (particles 2 mm), 40%	Fine earth (particles 2 mm), 60%
Specific gravity, tons/m^3	2.72	2.72	2.72
Bulk density of skeletal material, tons/m^3	1.65	2.0	1.6
Total moisture content, %	20.0	--	--
Absolute moisture capacity, %	24.0	15.0	24.0
Coefficient of friction		0.4	
Cohesion, kg/cm^2		0.1	
Relative compression when loaded to 10 kg/cm^2; %		7	
Percolation coefficient, cm/sec.		$2\cdot10^{-6}$	

It is planned to build the supporting prisms of rock from excavations dug for other purposes and special quarries, without sorting and without special compaction. The core of the dam will be made of gravelly loams and loamy sands, and the transitional zones of natural sandy-gravelly materials and crushed stone. The core of the dam will be joined to the foundation with a tooth cut into the rock. A gallery is planned within the tooth for curtain grouting (see Figure 5-18).

The volume of the loam core will be 1,200,000 m^3; that of the sand-gravel filters, 700,000 m^3, that of the crushed stone filters, 450,000 m^3, and that of the rock fill body of the dam more than 9,000,000 m^3.

The material will be placed in the core of the dam using techniques employed at the Vilyuy Hydroelectric Power Station.

* * *

Experience in building unfrozen dams shows that they can function successfully only if proper steps are taken in planning to assure seepage resistance and overall stability of the structure.

These steps include, in our opinion, careful preparation of the foundation for the dam based on a detailed study of frozen ground conditions and of the physical and mechanical properties of the ground, taking into account the change in those properties with time. The attachment of the body of the dam to the foundation, the subgrade contour, and the drainage facilities should be designed in such a way that temperature conditions in the foundation are properly controlled. Special steps should be taken to see that the foundation materials are resistant to seepage and not subject to piping.

The plan should include provisions for the possibility of the occurrence of seepage and subsidence of the foundation as well as measures to control heat flow in the foundation. Unfortunately, as we have seen from previous examples, these matters are not always taken into account in the design stage. As a result, major effort is devoted, during the construction and operation of the facilities, to design and implement steps to reduce the damaging effects of heat and mass exchange on the integrity of the structure.

5-4. UNFROZEN DAMS IN THE CANADIAN NORTH

Several hydroelectric power stations of different sizes having reservoir dams built of natural materials are now operating in the Northwest Territories and the Yukon Territory of Canada, which are entirely located within the far northern region of that country. These include a hydroelectric station on the Taltson River near Fort Smith, two stations on the Snare River in the vicinity of Yellowknife, and on the Yukon and Mayo Rivers in the region of Whitehorse. A hydroelectric power station with a dam of sand-gravel and clay materials was built on the Yukon at Whitehorse Falls to provide electricity for gold mining operations and copper mines in the Whitehorse region. Every year when temperatures drop below freezing longitudinal cracks at the crest form in the core of the dam, which is built of clay and silt materials (see Figure 5-19).

In building dams and similar structures in permafrost regions, efforts are made to select sites where the foundation material for the structures will be rock. Permanently frozen cohesive and friable materials are used as the foundation for a dam if they do not lose their bearing capacity when they thaw. In this case sand pile drains are installed to speed up the process of consolidation. Materials which represent a potential hazard when they thaw are frequently replaced.

Figure 5-19. Crack along the crest of a dam on the Yukon River.

Where possible, supporting prisms are built of rocky material and antiseepage elements of sandy-clay, sandy-silt, or other materials of glacial origin. In particular cases, such as the Whitehorse Dam on the Yukon River, materials were used in the cutoff core of the dam which are subject to frost heaving and fissuring when they are wet and undergo periodic freezing and thawing.

During the operation of this dam substantial additional capital investments have been necessary to repair it every year. The dam is repaired by pumping "lignosite," a paper mill waste, into boreholes.

The well-planned program for monitoring the condition of the structures should be mentioned. Scientific research institutions and individual specialists are involved in this work, so that undesirable changes in the dam can be detected early and necessary preventive steps can be taken.

Experience gained in building the dams and other structures in the Nelson River basin is of very great interest [14, 17, 108].

1. <u>The Kelsey Hydroelectric Power Station on the Nelson River</u> was built in 1957-1960 in the province of Manitoba. The climate near the site is cold. The mean annual temperatures is -3.9°C. The frost-free period lasts an average of 140 days each year. About 400 mm of precipitation fall annually.

The surface of the ground is covered with clay in a layer 0.3-7.5 m thick. Beneath this clay there is either rock or glacial moraine material up to 6 m thick, which consists of sand, gravel, and granular till (boulder clay). The total thickness of the overburden varies; it may reach 15 m or more.

The construction site is located not far from the southern permafrost boundary. The permafrost is discontinuous in nature: frozen ground is found sporadically both in the vertical and in the horizontal directions. Ice is found in the permafrost in the form of uniformly distributed small crystals and lenses up to 17.8 cm thick. The temperature of the permafrost ranges from 0 to -1.9°C.

The installation consists of the hydroelectric power station, a sluiceway, a central earth dam, and seven smaller marginal dikes along both sides of the reservoir. The river bed is walled off by a rock fill dam 219 m long with a maximum height of 36.6 m and a total volume of 220,000 m^3. The foundation of the dam consists of sound rock (see Figure 5-20).

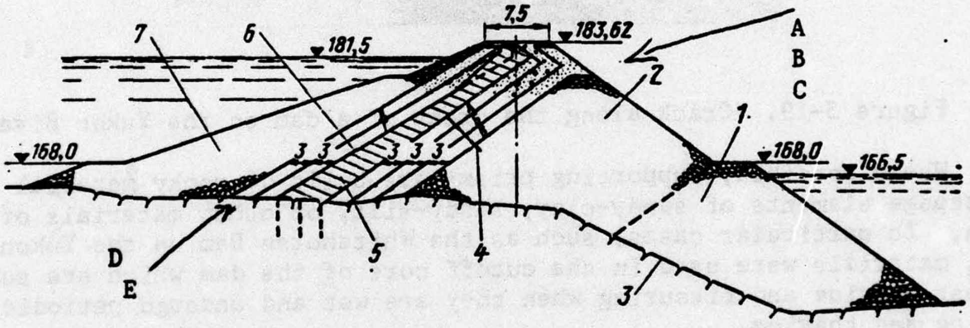

Figure 5-20. The Kelsey rock fill dam.
1 - Downstream cofferdam; 2 - Stage II fill of supporting prism; 3 - Rock surface; 4 - Transition zones A, B, and C; 5 - Impervious core; 6 - Transition zones D and E; 7 - Upstream supporting zone.

The downstream cofferdam, which was built in the river bed to wall off the dam site, became a component part of the body of the dam as its downstream wedge. The rest of the rock fill for the body of the dam was built up in a single lift to the full height.

The main prism of the rock fill dam was built of rock from excavations for other purposes and from special quarries. The maximum size of the stones was determined by the capabilities of the loading and transporting equipment. No more than 15% soft and weak stone and overburden was permitted in the main prism.

The watertight earth sloping core was protected below with three and above with two layers of transitional filter materials. Natural sand was used in transition zones A and B, while different grades of crushed rock from excavations for other purposes were used in zones B, C and D: 6.3-38.1 mm in zone B, 76.2-177.8 mm in zone C, and 38.1-177.8 mm in zone D.

In preparing the foundation for the dam below the sloping core and the transition zones, the unconsolidated deposits were removed entirely down to solid rock. This work was begun with bulldozers and drag lines, and final cleanup was done with water jets from monitors and manually. The rock slopes of the banks adjacent to the dam were cut away with explosives to give a gradient of 2:1. Concrete was used for best attachment of the dam core to the foundation and to smooth out the unevenness of the foundation. Only clay and sand deposits were removed from the foundation of the dam within the area of the main rock fill prism and transition zones C and D. Loose deposits (gravel and boulders) were not removed. The rock foundation under the core of the dam was grouted.

The sloping core of the dam was made of clay having a density of 2.7 tons/m^3, a plasticity of 28.1, a content of clay separates equal to 58%, and a percolation coefficient of $A \cdot 10^7$ m/sec.

The clay was packed with 13-ton pneumatic-tired rollers in 15-cm layers. At the higher levels, where the core was thinner, the loam was compacted only with bulldozers. The loam was placed mainly during the warm season, but at the end of September 1959 (when the dam was already 6 m below the planned level) difficulties appeared in relation to the freezing of the clay. Placement of the core continued with the addition to the loam of a 78% solution of calcium chloride at a rate of 3 kg/m^3. The loam was placed one small area at a time. The loam still froze, however, and it was necessary to stop, cut away, and remove the frozen surface layer (the frozen layer removed was sometimes as thick as 15 cm).

Placing of the loam went on this way at daytime temperatures from +1.7°C to -5.6°C. As the outside air temperature dropped, the clay was placed in a special enclosure covered with polyethylene film. Within the enclosure the temperature was kept within a range of +1.7°C to -3.9°C. As the loam was packed in place, the enclosure was moved along the core. The amount of chloride added to the loam was increased to 6-9 kg/m^3 and work continued as long as outside air temperatures were -18°C or higher.

In building the central dike of this installation (which was 260 m long) as well as the No. 1 west dike connecting the spillway to the right bank and the No. 1 east dike connecting the power station building with the left bank, permafrost containing lenses of ice was stripped completely down to the bedrock (see Figure 5-21).

Several small marginal dikes up to 1.8 m high were built along the edges of the reservoir to prevent flooding of various low areas. Only the topsoil was removed to prepare the foundation for these dams. The dams were built during the winter by placing earth from borrow pits located close to the dams. As the loam was placed in the body of the dam occasional lumps of frozen ground were included. These were broken up with bulldozers. A total of 4,130 m^3 of loam was placed in these dams (see Figure 5-22).

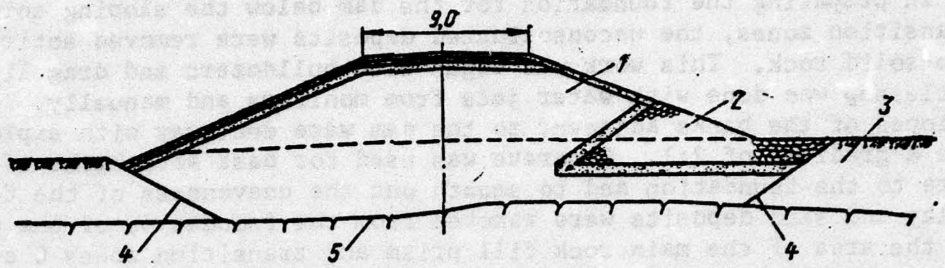

Figure 5-21. Kelsey No. 1 east dike.
1 - Dike; 2 - Drainage; 3 - Permafrost subject to thermal subsidence; 4 - Outline of excavation; 5 - Surface of the rock.

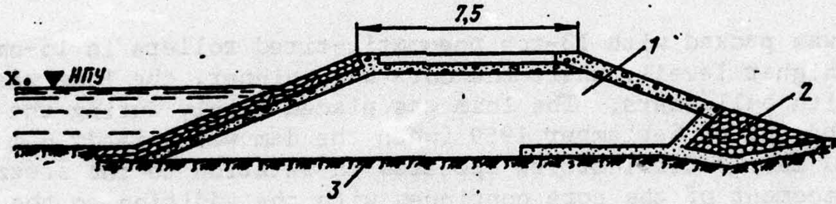

Figure 5-22. Marginal dike.
1 - Fill; 2 - Drain; 3 - Foundation after clearing away topsoil.
Key: x - NHL.

Experience with the construction of two of the lateral dikes is of particular interest -- No. 2 east and No. 2 west, where thawing of the entire layer of compressible permafrost susceptible to thermal subsidence in the foundation is permitted during operation of the facility.

Each dam is about 600 m long with a maximum height of 6 m. The foundation of these dams contains a top layer of peaty earth from 0.3-1.2 m thick. Below this is mud and a layer of moraine more than 6 m thick underlain by bedrock.

The maximum thickness of the unconsolidated material is 14 m, with an average thickness of 6-8 m.

The entire layer of compressible earth is permanently frozen, and permafrost of all types is found -- from plastic to hard-frozen. Ice is present in the ground as small crystals and lenses up to 15 cm thick. In some places lenses of unfrozen ground are found (taliki). The active layer is about 0.9 m thick.

Water-saturated and moist ground, of course, has a low shear strength. When dams are built on such ground, foundation material can disintegrate, and in order to prevent this, steps are taken to speed up consolidation of weak materials.

Compressive deformation of the ground takes place chiefly through reduction in the volume of pores within it, since reduction in the volume of skeletal particles has an insignificant influence on this process. If the pores in the ground are completely full of water, compression can take place only when excess water is removed from the pores.

The process of consolidation of relatively impermeable clay materials in the foundation of a dam is very slow, since it can be compacted only after the water that fills the pores is squeezed out. In ground of this type the compressive stress produced by loading is initially applied to water in the pores of the ground. Then, as time passes, some of the water is forced out into more permeable adjacent areas. At the same time the solid skeletal particles are more closely approximated and the ground is capable of accepting some of the load imposed upon it.

Thus through consolidation the density of the ground gradually increases as a natural result of its own weight or that of a structure built on it.

According to the theory of seepage consolidation, the compaction time for a layer of saturated ground is proportional to the percolation coefficient and the square of the thickness of the layer of ground compacted. Since the strength and deformation characteristics of the ground increases as it is compacted, there is a need to speed up this process. One of the techniques for speeding up the process of ground compaction is the construction of vertical sand pile drains: sand drains are built in the layer of saturated ground being compacted, so that pathways for the movement of water squeezed out of the compacted ground are reduced and the compaction time is abbreviated.

When drainage of this sort is provided, excess pore water is removed from the layer of ground in a radial direction toward the sand pile drains over much shorter routes. The depth of the drain holes depends on the thickness of the layer being drained. Caps of sand are placed at the top of the vertical sand drains as well as horizontal drains to carry the water away. The combined drainage system (sand caps and vertical drains) promotes consolidation of the dam foundation through shortening the distance through which water moves as it is squeezed out of the pores in the ground.

Since thawing of the foundation materials at the Kelsey installation is unavoidable, free settlement of the dams is possible without disturbing the stability of the ground as a whole.

Because of the high ice content in the clay foundation materials and their low permeability after thawing, they lose bearing capacity. Therefore sand pile drains 25-40 cm in diameter were installed to promote drainage and consolidation of the thawing clay material in the foundation of the dikes. The distance between the drains varied depending on the height of the dikes: the distance was 3 m when the dike was 4.5 m high, and 6 m when the height was less than 3 m. The drains were 7.6 m deep, and the last row of drains under the downstream wedge was 12 m deep and reached to bedrock (see Figure 5-23).

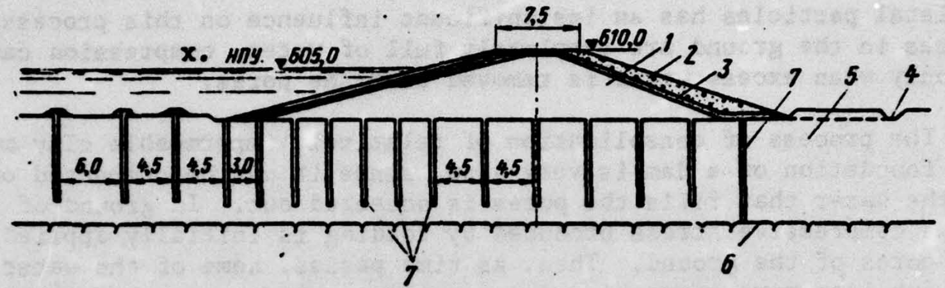

Figure 5-23. Kelsey No. 2 east dike.
1 - Dike body of sandy material; 2 - Rock facing; 3 - Transition zone of fine gravel; 4 - Collector ditch; 5 - Watertight permafrost subject to thermal subsidence; 6 - Rock surface; 7 - Sand drains. Key: x - NHL.

The dikes were built of sandy material, since good packing was possible during the winter. If uneven settlement of the foundation occurs, such a dike readily adapts to the deformations.

During the planning stage it was found that uneven settlement was possible because of the irregular distribution of permafrost in the foundation, and such settlement might lead to disturbance of the integrity of the structure. It was thought that the stability of the foundation is determined by the rate of thawing, the rate of drainage or redistribution of the meltwater, improvement in the shear characteristics of the thawed clay, etc.

Judging on the basis of the thickness of ice layers in the ground, the settlement of the dam should measure about 1.8 m. It was also assumed, given rapid drainage of the water formed from the melting of the ice layers, that consolidation of the ground would keep pace with the melting, and the loss of foundation strength would be negligible.

Seepage of water through the permeable body of the dike evidently is complex in nature. It is assumed that the varved clay in the foundation is watertight, and that with a particular water level in the reservoir the seasonal freezing of the dike from the surface will reach some depth below the level of the water in the reservoir.

The rate of thawing of the foundation depends on the temperature of the water and decreases considerably during the winter. During the early years the rate of thawing reached a level of 1.5-1.8 m per year, dropping in succeeding years to 0.6-0.9 m/year with a clear tendency toward decrease with time.

Settlement was observed during the period of operation in the form of depressions at the crest and on the slopes, giving the crest a wavy outline. Significant subsidence of the crest (up to 1.5 m) occurred in July 1962, but during the summer the crest of the dike was filled back to the design level. Material was also added to the crest in 1964 and 1966. The maximum total settlement amounted to 2.2 m (see Figure 5-24).

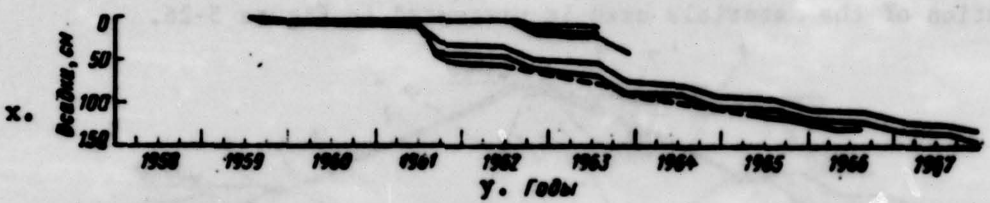

Figure 5-24. Settlement of Kelsey No. 2 east dike [108].
Key: x - Settlement, cm; y - Year.

The significant deformations of the dike were associated with longitudinal cracks at the crest and depressions in the downstream slope. Scattered depressions also apparently formed on the upstream side. But no transverse cracks were reported. Longitudinal cracks from 2.5-5 cm wide and 0.6-0.9 m deep formed mainly in the unconsolidated layer placed on top of the main sand core. At the end of 1967 settlement of the foundation measuring 1 m and 1.5 m was recorded where the full depth was 3.6 m and 4.8 m, respectively. At one of the points of measurement the settlement was 0.3 m with a dike height of 2.4 m [108].

The rate of settlement of the ground corresponds to the rate of thawing of the foundation. But the variation in thawing intensity diminishes quite rapidly with foundation depth, and the nature of the variation in intensity of subsidence remains practically the same from year to year at any depth. The intensity of subsidence decreases with time.

The successful construction and operation of the dams at the Kelsey installation where sand pile drains were used in the foundation show that the solution adopted was correct.

2. The Kettle Hydroelectric Power Station on the Nelson River is located 160 km from the Kelsey Hydroelectric Power Station. The engineering geology and frozen-ground conditions of the entire area where the Kettle Power Station dam was built are similar to those described for the Kelsey Station.

The construction at the Kettle installation includes: a 1,000,000 kw electric power station, a fixed concrete dam, a sluiceway, and earth dams running to the left and right banks. In addition, to reduce the area of land flooded, five marginal dikes and two dams were built along the sides of the reservoir. At this installation, just as at the Kelsey Hydroelectric Power Station, the principal structures (the dam, the power station, and the sluiceway) are located on a rock foundation.

The marginal dikes and dams are based on ice-saturated permafrost and their method of construction is quite similar to that used at the Kelsey Power Station.

The earth dam connecting to the left bank is 760 m long with a maximum height of 40 m. It was designed with a Class 1 central core of impermeable earth, and the lateral prisms of Class 2A semipermeable earth. The construction of the dam is shown in Figure 5-25, and the particle size distribution of the materials used is presented in Figure 5-26.

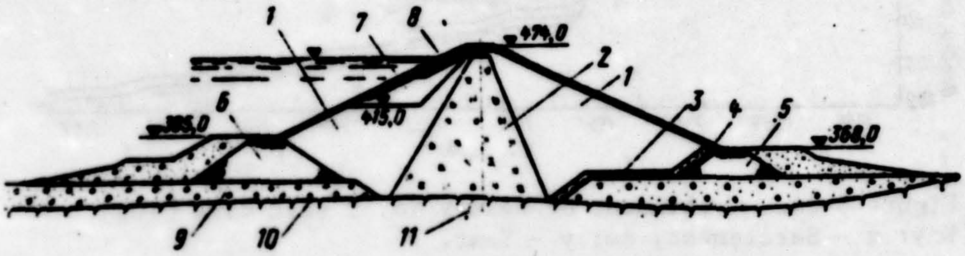

Figure 5-25. Left-bank Kettle dam.
1 - Semipermeable Class 2A material; 2 - Core, Class 1 material; 3 - Filter, Class 4 material; 4 - Filter, Class 5 material; 5 - Downstream cofferdam; 7 - Facing, Class 3 earth; 8 - Riprap (Class 9) on crushed rock filter (Class 5); 9 - Sand with gravel and cobbles; 10 - Rock surface; 11 - Grout curtain in foundation of core.

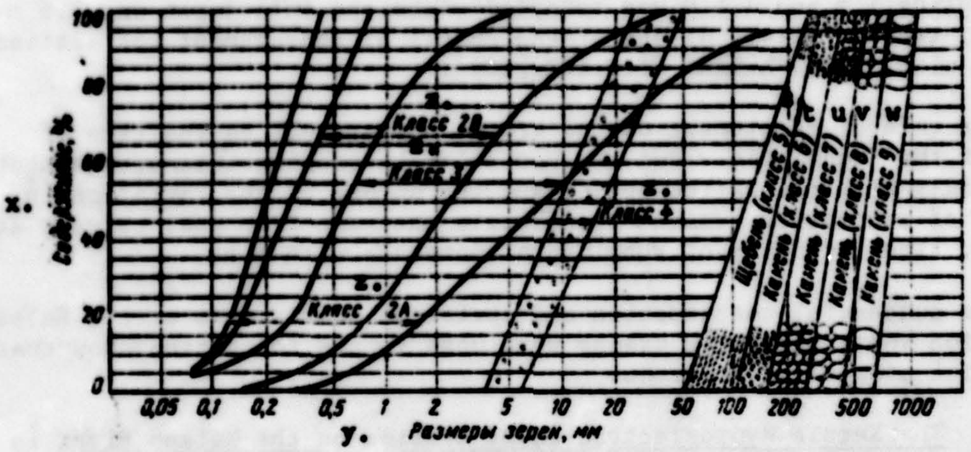

Figure 5-26. Particle size distribution of materials in the Kettle dam.
Key: s - Crushed rock (Class 5); t - Rock (Class 6); u - Rock (Class 7); v - Rock (Class 8); w - Rock (Class 9); x - Content, %; y - Grain size, mm; z - Class.

Friable material in the foundation of the dam was stripped down to the rock only under the central core. The plan called for grouting the rock foundation of the dam to connect with the core.

A different treatment was chosen for the earth dam connecting with the right bank. This dam was designed to be about 300 m long with a maximum height of 48 m (see Figure 5-27). Since the compressible earth in the foundation of the dam was quite thick (up to 30 m), connecting the central core with the rock, as was planned for the channel and left-bank dams, would make it necessary to dig a deep excavation and remove considerable quantities of earth. Therefore in this area it was decided to remove from the foundation only weak materials which would lose their bearing capacity when thawed, leaving the compact boulder clays and sandy-gravelly and rubble material covering the rock. In order to assure integrity of the structure it was planned to build an antiseepage curtain along the axis of the dam, butting it into the rock foundation. Drainage was also provided under the downstream wedge in the form of vertical sand pile drains with an outlet to the horizontal drainage in the foundation of the downstream supporting prism.

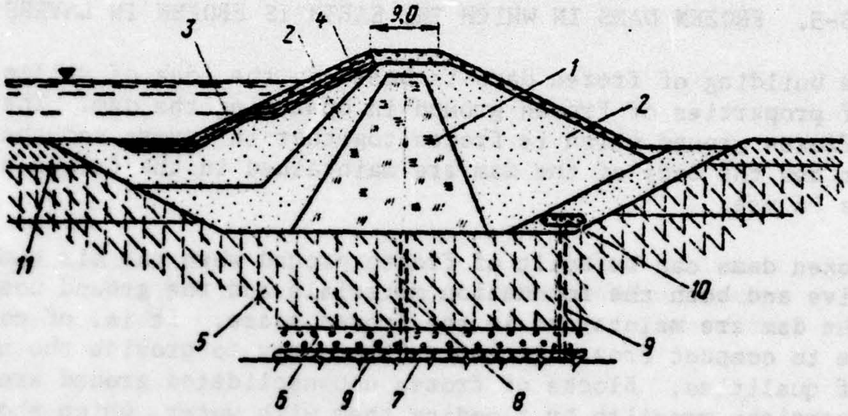

Figure 5-27. Right-bank Kettle dam.
1 - Core, Class 1 material; 2 - Lateral prism, Class 2A material; 3 - The same, Class 3; 4 - Riprap on crushed stone; 5 - Compact grouted boulder clays; 6 - Sand with cobbles and gravel; 7 - Rock; 8 - Antiseepage curtain; 9 - Drainage shaft; 10 - Drainage prism; 11 - Earth subject to subsidence

As far as the marginal dikes were concerned (the Saddle Dam, 1230 m long with a maximum height of 40 m, and the Butno Dam, 3400 m long with a maximum height of 20 m), which were built along the sides of the Kettle reservoir, different structural designs were adopted depending on the engineering geology and frozen-ground conditions, but both were built as unfrozen dams permitting thawing of the foundation during the period of use (see Figure 5-28).

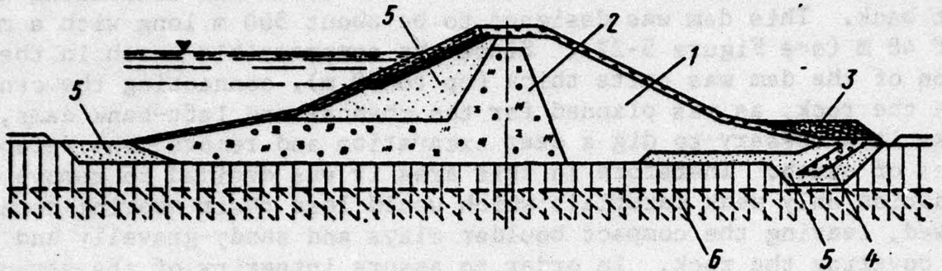

Figure 5-28. The Saddle dam.
1 - Semipermeable Class 2A material; 2 - Core, impermeable earth, Class 1; 3 - Rock toe, Class 6 material; 4 - Permeable material, Class 4; 5 - Permeable material, Class 5; 6 - Dense ground moraine loams.

5-5. FROZEN DAMS IN WHICH THE EARTH IS FROZEN IN LAYERS

The building of frozen dams is based on the idea of utilizing the waterproof properties of frozen ground in operating the dam. The dams are built of frozen ground which is frozen together in layers and then the foundation and the body of the dam are maintained in the frozen state while the dam is in use.

Frozen dams can be built of frozen ground when the air temperatures are negative and both the foundation materials and the ground used in the body of the dam are maintained in the frozen state. It is, of course, impossible to compact frozen ground sufficiently to provide the necessary waterproof qualities. Blocks of frozen unconsolidated ground are converted into a watertight monolith by flooding them with water, which should fill all voids and pores in the fill.

Dams of this sort are not discussed in the current "Temporary Instructions for Planning Earth and Rock Fill Dams in the Far North" (VSN-08-65). But frozen dams can be built of frozen ground. Dams of frozen ground which is frozen together in layers can, in our opinion, be constructed with additional artificial freezing of the core while in use. In this case the freezing system can be installed before the blocks are frozen together in order to reduce construction costs.

The prospects for building dams of frozen ground are very attractive. Under particular conditions, such as when materials suitable for building antiseepage elements are lacking in the vicinity of the construction site or when it is impossible to pack unfrozen ground properly without complicated engineering procedures, this type of dam may be the most suitable for low-pressure use.

When dams built of frozen ground are in use, steps should be taken to maintain the body and the foundation of the dam in the frozen state. This is accomplished by providing the necessary thermal insulation of the slopes and crest of the dam. In addition, the crest of the dam should extend above the maximum water level in the reservoir a distance greater than the depth of seasonal thawing at the crest of the dam. This is necessary to avoid having the water reach the zone subject to seasonal thawing.

An example of a dam built of frozen ground is the experimental dam built in the winter of 1963 on the Irelyakh River at the town of Mirnyy. The bedrock layer consists of marly clays and marls interstratified with limestones. The river channel alluvium 3.5 m thick consists of sand-gravel deposits. The frozen region under the river channel is up to 7.5 m thick.

The dam was 285 m long with a maximum height of 4.1 m impounding a head of 1.5 m of water. The frozen core of the dam, which was designed to act as the antiseepage element, was built by packing frozen ground in layers, pouring water over it, and freezing it. Studies showed that the ratio of ice to earth was 1:1 by volume, and the density of the ice-earth mass was 1.2-1.5 tons/m^3. The top of the frozen core of the dam was covered with a thermal insulation layer of earth thicker than the depth of seasonal ground thawing in the region.

The crest of the dam was 2.75 m higher than the maximum water level in the reservoir, and the top of the frozen core was 1.1 m above the maximum water level in the reservoir (Figure 5-29).

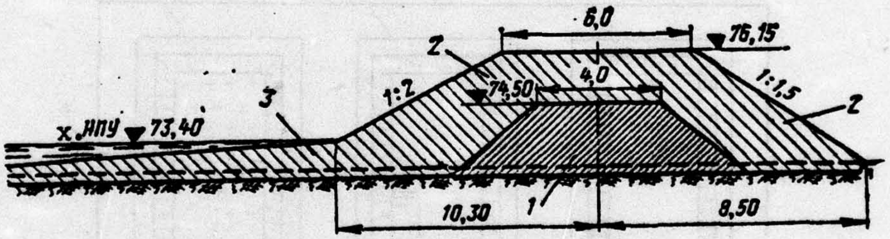

Figure 5-29. Experimental dam made of frozen ground on the Irelyakh River.
1 - Frozen ground placed and saturated with water; 2 - Frozen ground placed without pouring water on it; 3 - Unfrozen ground placed in the spring.
Key: x - NHL.

The downstream slope of the dam was covered with a thermal insulation layer of frozen ground, and the upstream slope with a layer of unfrozen loam, which was rolled in the spring. The covering on the upstream slope was built as an apron.

The design of the dam was crude and the desired temperature regime was not maintained in it, but it is of considerable interest as the first attempt to build a dam of this type.

The frozen core can be recommended for building low-head, low-performance dams, provided steps are taken to cut off the thawed zone under the river bed completely and reliably and provided that the frozen core is not permitted to thaw where it comes in contact with the spillway.

Building a frozen core only in the floodplain segments of a dam is possible and feasible, but unless the thawed zone below the stream bed is cut off during use it is not very reliable. Construction of an apron and building the upstream slope of unfrozen loam, i.e. of a relatively impermeable material, will tend to reduce thawing of the frozen core.

The Institute of Permafrostology of the Siberian Division, USSR Academy of Sciences, in the winter of 1961 conducted a very interesting experiment on layer-by-layer freezing at Mirnyy in order to gather field data on the construction of dams from frozen ground.

Three experimental mounds of fill measuring 10x30 m and 2 m high were built of frozen blocky earth (see Figure 5-30). Each mound was built by successive freezing of layers of different thickness (10, 20, and 40 cm). In order to obtain different ratios of water to earth, each experimental mound was divided lengthwise into three zones and the earth in each zone was compacted to a different extent: no compaction, compaction by three passes of an S-80 bulldozer, and compaction by ten passes.

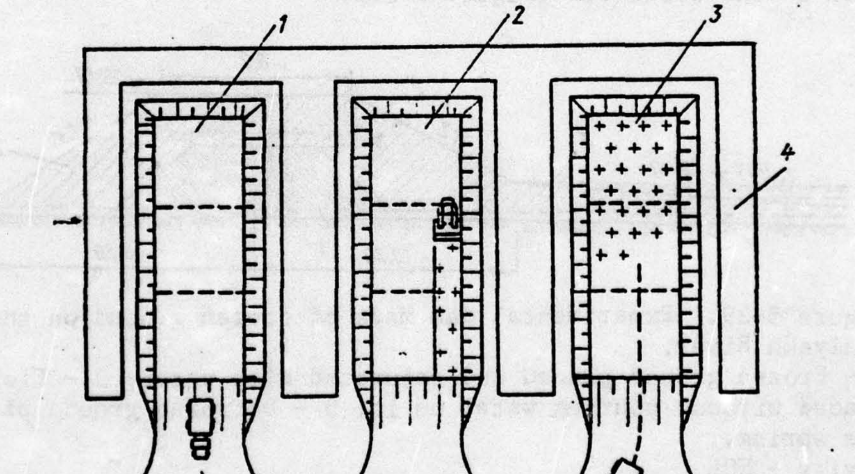

Figure 5-30. Frozen fill experiment.
1 - Wetting down a layer of earth; 2 - Smoothing a layer;
3 - Dumping a layer; 4 - Ridges of snow.

Frozen ground delivered to a zone of a mound was smoothed out with a bulldozer and compacted by a definite number of passes by the bulldozer. This broke up the lumps of frozen ground, and individual large lumps were removed from the plot. Then the plot was flooded with water after building a ridge of snow around it.

The water was poured on all at once, since sprinkling the surface of the layer formed a crust which prevented complete saturation of the earth in the layer added.

The average daily air temperature during the operation varied from -29.8° to -7.3°C, with an average temperature during the period of freezing operations of -19.2°C. Wind speed did not exceed 1-3 m/sec.

At temperatures of -25°C the water in the ground froze after 40 min, while at -8 to -12°C freezing of a layer took 1-1.5 days, depending on the thickness of the layer. Special pits were dug to monitor the freezing of the layers.

Good saturation of the layers was observed during the experiment, and the porosity of the ice-earth mass was negligible. *Kerosene was used to determine the percolation capacity of the ice-earth mounds. This was poured into holes bored in the material. The kerosene level in the holes remained unchanged for two weeks.

The experiment showed that the earth in all nine fill plots was ice-saturated, did not permit seepage, and contained sufficient reserves of "cold." The water-earth ratio depends on the degree of compaction: it was less in areas compacted with ten passes of the bulldozer than in areas stacked without compaction. Maximum accumulation of cold within the mound was observed in the areas with the minimum water-earth ratio, and *vice versa*. A water-earth ratio of 1:2.5 (by weight) can be considered optimum for layer-by-layer freezing.

A dam up to 12 m high can be built during one winter season by placing layers 40 cm thick.

The search for economic methods of building dams from frozen materials in the Far North is now directed toward wider use of the watertight properties of frozen ground.

We have proposed a technique for building water-retaining structures in the Far North using any materials available at the construction site, including clay, sand, mud, gravelly sand, or other cohesive frozen materials, plant and moss materials, etc. These materials are packed into the core of the dam in layers 0.5-1.0 m thick without special requirements as to density (see Figure 5-31). After placement each layer is flooded with water until completely saturated, i. e. until all voids in the layer of earth are full. The water freezes and imparts a monolithic structure to the earth, forming a dense, impermeable ice-earth core. Where necessary to prevent leaks into the lateral prisms, a film is applied to the interface when a layer is put in place, and in order to avoid tears in the film, a thin protective layer of dry sandy earth, peat, or snow is put down.

AD-A035 905   COLD REGIONS RESEARCH AND ENGINEERING LAB HANOVER N H   F/G 13/13
DAMS ON PERMAFROST (PLOTINY NA VECHNOY MERZLOTE),(U)
NOV 76 G F BIYANOV
UNCLASSIFIED   CRREL-TL-555   NL

2 OF 3
AD
A035905

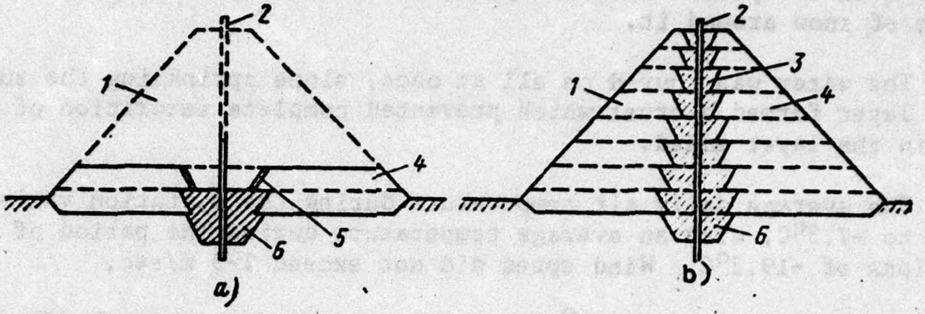

Figure 5-31. Engineering diagram for building an ice-earth antiseepage core in a dam.
a - Initial stage; b - Final stage.
1 - Upstream wedge; 2 - Freezing shaft; 3 - Ice-earth core; 4 - Downstream wedge; 5 - Polymer film; 6 - Tooth of core.

The upstream and downstream supporting prisms of the dam are built up, as in construction under ordinary conditions, of any available earth in layers 1.0-2.0 m thick slightly in advance. A trench is formed in the core zone, into which is packed the material for the future ice-earth core.

A dam with this sort of ice-earth antiseepage core can be used with or without subsequent freezing. This has been demonstrated by the successful use of dams made of materials from the ground (Ege-Khaya, Oyuur-Yurege, the temporary dam on the Irelyakh River, the dam on the Halednaya River, etc.), the bodies of which froze as a result of outside air temperatures below freezing. This would also be enhanced by winter filling of the supporting prisms, which as experience has shown, introduces a considerable quantity of cold into the body of the dam. This cold persists when outside air temperatures are positive, a fact of particular importance for the downstream wedge. As for the upstream wedge, which during the use of the dam is always subject to the influence of the positive temperatures of the reservoir, in order to reduce heat and mass exchange, i.e. to reduce the influence of reservoir heat on the frozen core, it is desirable to build the upstream wedge of less-permeable earth.

To assure reliability of such dams, it is necessary to build a supplementary cooling system into the ice-earth core built at negative temperatures. The cooling columns should be installed from the level of the foundation in order to reduce the amount of boring necessary.

The use of ice-earth cutoff cores in dams made of natural materials permits us to build them at any time of the year, and specially treated earth is not required for the core of the dam. The dam can be built of any earth materials, construction time is reduced, the dam is placed in operation more quickly, and its cost is reduced.

Until we have more experience in bulding and operating such dams, they can be recommended only for low heads of water. When appropriate studies have been completed, we feel that dams of this type could be built also for higher-head use.

5-6. DAMS OF FROZEN EARTH IN WHICH THAWING IS PERMITTED DURING OPERATION

Further search for new efficient methods of building dams on permanently frozen foundations has produced a method for building dams of frozen ground during the winter in which the earth is permitted to thaw during operation [9].

We shall describe several examples of such dams.

1. Two dams of the same design were built in the winter of 1962. One of them was 265 m long at the crest and 6.1 m high. The second, located 2.3 km below the first, was 125 m long and 8.1 m high. The anti-seepage component of the dams was a cutoff made of wood panels sunk into the marly clay of the foundation, which was ice-saturated to a small extent. The dams were connected to the foundation with a tooth 2.2 m deep (see Figure 5-32). Cutoff panels were erected on wood piles sunck in holes bored along the bottom of the tooth. The panels were made of double-layer 25 and 40 mm boards covered with two layers of tarpaper.

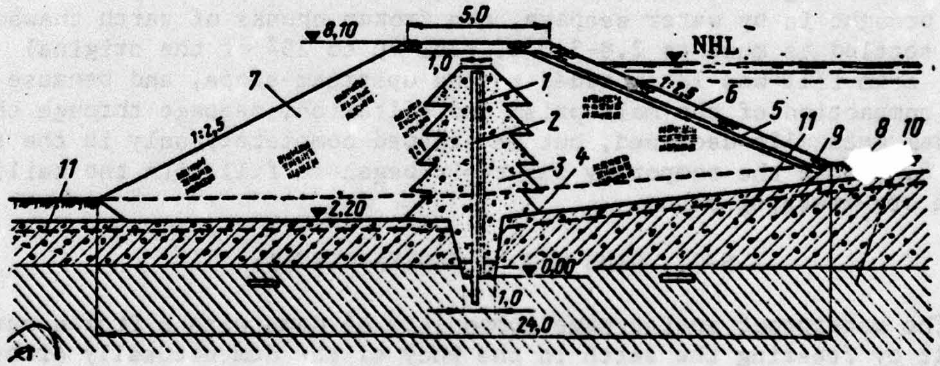

Figure 5-32. Dam made of frozen earth.
1 - Wood panel cutoff; 2 - Piles; 3 - Sand-gravel core; 4- Loam cutoff; 5 - Crushed stone bedding; 6 - Facing of small stones; 7 - Body of the dam; 8 - Topsoil, mud, peat; 9 - Ice-saturated loam with gravel; 10 - Semiconsolidated marls; 11 - Line of seasonal thawing of the ground.

As the frozen loam thaws, the dam is deformed. Therefore, in order to reduce the harmful effect of deformation of the supporting prisms on the cutoff walls and to prevent them from failing, the cores of the dams were built of dry sand-gravel mixture, which would also prevent piping if the cutoff panels came apart.

At the same time this sand-gravel layer protected the cutoff panels from mechanical damage as the frozen blocks of earth were packed into the body of the dam. The minimum thickness of the sand-gravel layer in vertical section was 1.5 m.

During the spring and summer of the first year of operation the frozen marly loams of which the dam was built thawed and compacted, so that the dam showed up to 50 cm of settlement.

The day after the water pressure in the left-bank portion of the upper dam had reached the design level, seepage up to 6 l/sec was observed. The reason for the seepage was probably the spreading of the cutoff panel joints due to inadequate packing and rolling of the frozen blocks of loam.

After the downstream slope of the dam was heaped with earth at the site of emergence of the seeping water, piping of the earth stopped, and then seepage also stopped completely. Seepage was not observed in the lower dam.

2. A dam 12 m high and 360 m long at the crest was built of crushed frozen overburden materials -- marly loam and semiconsolidated kimberlite. The earth was packed as the dam was built merely by the passage of the delivery vehicles -- MAZ-525 dump trucks. The dam is a starter dam for a tailing pond, so that the specifications for it were those of a temporary structure. The first spring flood was passed over the body of the dam, since at this stage water loss from the reservoir was not important. Because of heat brought in by water seepage, the frozen chunks of earth thawed and the dam settled as much as 2.8-3.0 m, i.e. up to 25% of the original height. Loam fill was later added to the upstream slope, and because of natural compaction of the main prism and siltation, seepage through the body of the dam gradually declined, but it stopped completely only in the third year of use, when the temporary reservoir began to fill with the tailings from the ore-dressing plant.

5-7. DAMS WITH A FROZEN CURTAIN

The waterproof antiseepage components of dams with a frozen curtain are built by freezing the earth in the body of the dam naturally or by artificial means. A frozen antiseepage curtain is created by joining the artificially frozen core of the dam with the permafrost in its foundation. This method of creating a curtain is, in our opinion, the best method of building structures to hold water on permafrost.

The earth is frozen by installing special cooling systems consisting of freezing columns to admit cold air or using brine as an artificial coolant.

As the coolant circulates in the cooling columns, ice-earth cylinders form around them. As heat is withdrawn from the core of the dam and the flow of heat to the ice-earth cylinder diminishes, the diameter of the cylinders increases until they merge into an ice-earth curtain (see Figure 5-33). As the cooling system continues to operate, the system gets colder and the wall becomes thicker. The growth of the curtain depends on the length of time during which the cooling installation operates and the temperature of the coolant. Many factors affect the process of freezing of the earth. These factors include, above all, the operating conditions of the freezing installation, the temperature and circulation rate of the coolant, the thermophysical properties of the earth being frozen, the distance between the cooling columns, etc.

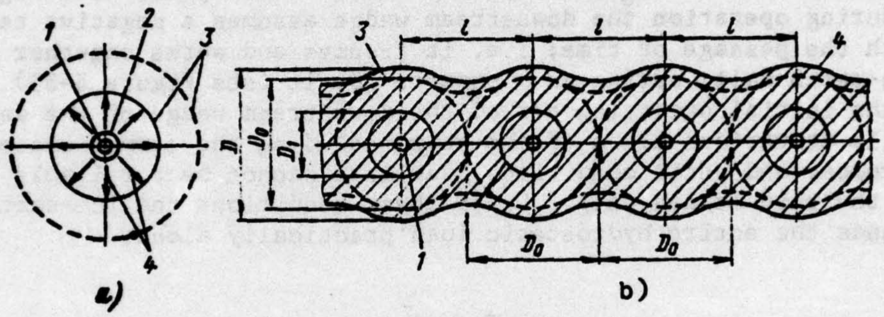

Figure 5-33. Diagram showing the creation of a frozen ice-earth curtain.
a - Formation of an ice-earth cylinder; b - Merging of the individual cylinders into a curtain. 1, 2 - Internal and external pipes of a cooling column; 3 - Outlines of the intermediate ice-earth cylinder; 4 - Direction of movement of cold, opposite to the withdrawal of heat; 1 - Distance between cooling shafts; D_1, D_0 - Diameters of the ice-earth cylinders, at an intermediate stage and at the moment of merging; D - Thickness of the ice-earth wall when the curtain is in use.

The rate and temperature of seeping water significantly affect the rate of freezing of the earth when seepage is involved.

The frozen curtain, like every antiseepage cutoff wall in the body of a dam made of earth materials, is the most critical element of the dam. But while seepage through the antiseepage elements of water-impounding structures may be permitted under ordinary conditions, the appearance of seepage through a frozen antiseepage element means the beginning of its total destruction, since it brings heat into the body of the dam and thus destroys the ice-earth wall. Further increase in seepage flow leads to an increase in thawed spaces in the cutoff and to a partial or total loss of ability to resist seepage.

The role of the frozen seepage curtain is particularly important during the initial period of operation of the structure. Water-impounding structures made of earth materials are, of course, subject both to settlement and to displacement in the horizontal direction under the action of hydrostatic pressure. Since dams made of earth materials are relatively flexible structures, they easily adapt to these deformations without any irreversible changes, erosion, or loss of seepage or general integrity. At the same time the frozen curtain during the initial period (before the downstream wedge freezes) is like a thin plate in the body of a flexible dam, which is imbedded in the permafrost foundation and the valley slopes.

After the reservoir has been filled, practically the entire hydrostatic head P_w, which at this stage is significantly larger than the design head impounded by the dam (P_0), is applied to this thin plate (see Figure 5-34). Ordinarily during operation the downstream wedge assumes a negative temperature with the passage of time; i.e. it freezes and works together with the thin ice-earth wall, acting as a support for it (see Figure 5-35). But during the initial operating period the downstream wedge of the dam, which is built at various times of the year including the summer season, remains unfrozen, and until conditions stabilize cannot be a reliable support for the thin frozen wall. Under these conditions the ice-earth wall withstands the entire hydrostatic load practically alone.

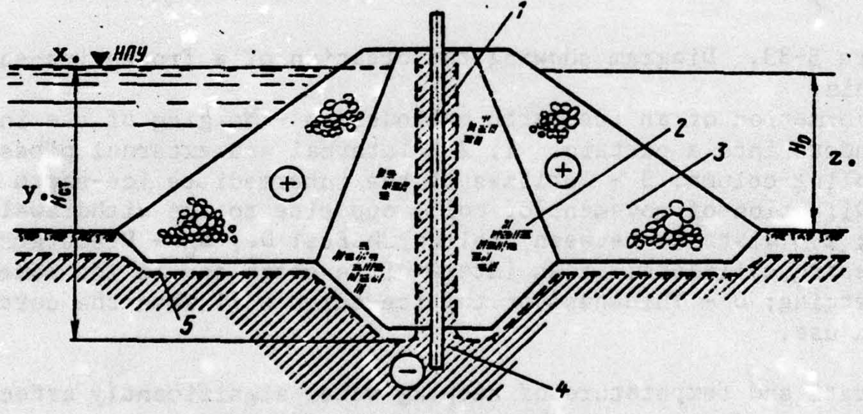

Figure 5-34. Frozen ice-earth curtain.
1 - Cooling column; 2 - Core of cohesive earth; 3 - Supporting prism;
4 - Contact between the frozen curtain and the foundation;
5 - Zero isotherm; P_w - Pressure head applied to the curtain;
P_0 - Net head.
Key: x - NHL; y - P_w; z - P_0.

Since the length of this ice-earth wall is large in comparison with its height, each meter of length acts as a cantilever beam inserted in the foundation. As a result of hydrostatic pressure the wall is subject to horizontal and vertical deformation (Δl) and shifts toward the lower pool (see Figure 5-36). Bending fractures at the point of insertion of this cantilever are prevented by the fact that the wall represents a "strongly reinforced structure." Here the pipes of the cooling columns act as the reinforcement on which the tensile forces act. These deformations also take place without the formation of cracks, since frozen ground -- and this is important -- is an elastic-plastic material with definite creep and relaxation characteristics.

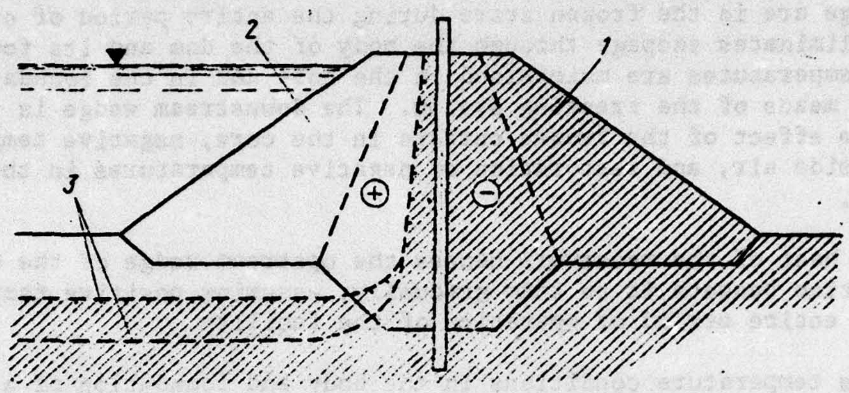

Figure 5-35. Frozen dam.
1 - Frozen zone; 2 - Thawed zone; 3 - Boundary of thawing of the dam foundation and the bed of the reservoir due to the heat from the water.

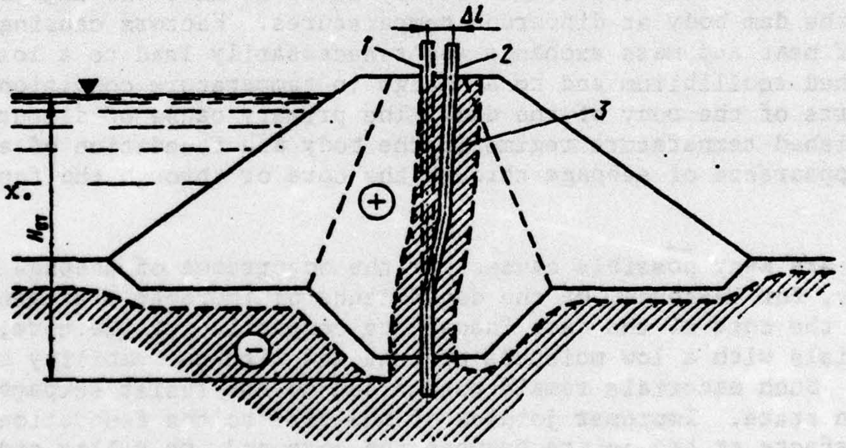

Figure 5-36. Diagram of the deformation of a frozen cutoff wall.
1, 2 - Freezing shaft; 3 - Frozen cutoff wall.
Key: x - P_w.

Cold-air freezing systems are ordinarily used to freeze the earth in the body and foundation of a dam in regions where the average annual temperature is no higher than -5°C. Cold atmospheric air is used in such systems during the winter.

The advantage of air cooling installations over brine installations is that much less electricity is consumed in using this source of cold. The system of freezing column pipes in the body and the foundation of dams may be installed in both the vertical and the horizontal positions.

The method of building dams on permafrost is selected in the light of actual engineering geology and frozen ground conditions. In dams of the frozen type where artificial freezing is used, the central core and downstream wedge are in the frozen state during the entire period of operation, and this eliminates seepage through the body of the dam and its foundation. Negative temperatures are maintained in the core and in the foundation of the dam by means of the freezing system. The downstream wedge is frozen through the effect of the frozen curtain in the core, negative temperatures of the outside air, and restoration of negative temperatures in the frozen foundation.

The heat of the reservoir causes the upstream wedge of the dam and the foundation beneath it to thaw gradually, assuming positive temperatures during the entire period of operation of the facility.

Thus temperature conditions in the body and foundation of a frozen dam vary over the cross-section: the core, downstream supporting prism of the dam, and the foundation benearth them are continuously in a frozen state, while the upstream wedge and the foundation beneath it are unfrozen. Structures of this sort functioning under different temperature conditions have proved reliable in practice. At the same time, of course, steps should be taken to maintain the established steady state of heat exchange between portions of the dam body at different temperatures. Factors causing unstable conditions of heat and mass exchange won't necessarily lead to a loss of the established equilibirum and to a change in temperature conditions within different parts of the body of the dam. The primary cause of disturbance of an established temperature regime in the body and foundation of a dam may be the appearance of seepage through the core or through the foundation of the dam.

There are many possible causes for the occurrence of seepage in frozen dams. It may, for instance, be the consequence of improper placement of the earth in the core of the dam, inadequate compaction of the core, or the use of materials with a low moisture content and high permeability coefficient in the core. Such materials remain porous and do not resist seepage even in the frozen state. Improper joining of the core to the foundation or structural defects at the joints between the core and the valley sides may also be responsible for the occurrence of seepage. Defects in the design of the freezing system or system malfunctions which may occur during use can also be the reason for seepage in a dam.

Among freezing-system design defects we might mention the use of a large interval between freezing columns, so that the ice-earth cylinders do not merge and form a cutoff curtain over the entire profile of the dam before head pressure is applied to the structure, or again, inadequate development of the curtain at the juncture with the valley sides. In the latter case, when the reservoir is filled, bypass seepage develops in fissured rock and causes a change in temperature conditions in the dam and its foundation.

It is particularly important to assure a good connection between the frozen curtain and the foundation. To assure this the freezing columns should be sunk into the frozen ground of the foundation to a definite calculated depth. It is essential that the unfrozen ground beneath the stream bed be cut off by the frozen curtain, otherwise after the reservoir has been filled, seepage flow by this route may intensify.

In summary, when the foundation is cleared, the tooth trench remains open for some time depending on how the work goes, and this may cause the zero isotherm to drop below the planned installation level for the frozen curtain piping system. In this case a thawed layer may remain under the core in the foundation. Therefore steps should be taken to prevent thawing of the foundation.

The first attempt to make a cutoff by freezing earth was in the construction of the Kama Hydroelectric Power Station (1937), but the work was not finished because construction of the station was stopped. Then during construction of the Gor'kiy Hydroelectric Power Station (1952-1954) a temporary frozen antiseepage curtain was created in the body of a cofferdam along the foundation site for the power station and the concrete dam. This was intended to function only during the period of operation of the cofferdams surrounding the foundation site. The curtain was 1200 m long with 853 freezing columns spaced 1.40 m apart. The columns were 19-21 m high and the coolant was calcium chloride with an average temperature of -16°C [94].

Seepage seriously affected the area of the cofferdam next to the valley wall as sands with percolation coefficients of 8-32 m/day were being frozen. This possibility had not been taken into account in the planning. The heat flux introduced by the seepage interfered with the growth of the ice-earth cylinders. In order to eliminate the effect of the seepage flow it was necessary to install apparatus to lower the water level ahead of the curtain and to increase the number of freezing columns and use a more powerful cooling plant.

Thereafter frozen antiseepage curtains were widely used in the construction of dams from local materials on permafrost. A number of dams with frozen seepage curtains have now been built. Some of these are described below.

1. A dam on the Dolgaya River was built in 1942 near the city of Noril'sk. The earth dam with an ice-earth curtain and a puddle-clay core is 10 m high and 130 m long at the crest. The foundation contains frozen icy loamy sands and loams. The core, which is 2-4 m thick, is attached to the frozen foundation by a tooth 3.6 m deep, which covers the unfrozen ground beneath the channel of the Dolgaya River. A loam apron was installed below the upstream prism with a tooth under the toe of the upstream slope. The dam was built of sandy materials (Figure 5-37). A thermal insulation layer of peat 40 cm thick was applied to the downstream slope. An ice-earth curtain was formed in the core of the dam by installing 48 freezing

columns of 100 mm pipe in holes. Pipes of smaller diameter were inserted in them. The freezing columns are 7-15 m long and are sunk 2 m into the foundation.

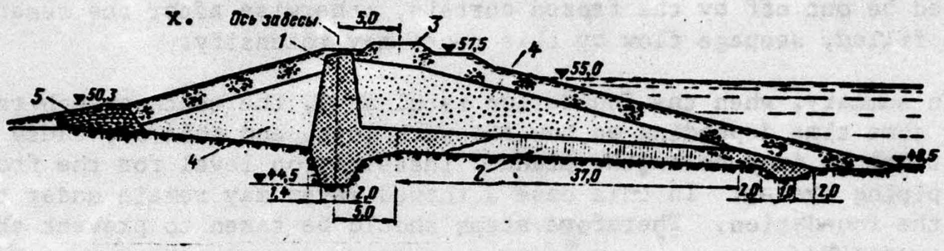

Figure 5-37. Dam on the Dolgaya River.
1, 2 - Core with apron; 3 - Riprap; 4 - Loamy sand; 5 - Drain.
Key: x - Axis of curtain.

The ground was initially frozen with a brine cooling plant. The coolant, a calcium chloride solution, was cooled by cold air as it circulated in special tanks and in pipe mains (brine conductors, distributors, and collectors). The brine was pumped from the tanks through the main pipe into the freezing columns. Then it moved through the space between the pipes and was returned to the tanks via the collectors. The brine cooling plant was used for four years [27, 94].

Experience in operating the plant revealed a number of deficiences in the cooling system. Specifically, cooling of the brine as it circulated through the outside pipes was not adequate; the brine temperature was significantly higher than the temperature of the outside air (15-20°C higher). Natural freezing of the dam from the downstream slope did not take place because a snow cover up to 4 m deep accumulated on the slope. In addition, when contaminated calcium chloride was used, the bad mixture settled out of solution onto the walls of the freezing pipes, and this interfered with heat exchange between the brine coolant and the earth. Settlement accumulated at the bottom of the freezing columns and obstructed them. As a result of corrosion of the pipes and improper installation of some of the freezing columns the brine leaked out into the freezing ground and created an unfrozen zone in the body of the dam.

As a result of defects in the freezing system, water which had seeped through the body of the dam began to appear at the toe of the downstream slope after two years of operation. Because of this the brine cooling system was replaced by an air system, in which cold air was forced through an air duct into the freezing columns with a high-pressure fan. But with this method of delivery the air was strongly heated in the fan: the temperature difference between the atmospheric air drawn in and the air forced into a column averaged as much as 10°C. The temperature difference between the air entering the freezing columns and that emerging from them averaged 4.8°C.

The method of delivery of cold air to the freezing columns was later changed: fans drew air from the freezing columns and cold air moved in behind it directly from the atmosphere. This method of delivering air to the columns eliminated the early heating of the air in the fan, but snow was also drawn into the freezing columns with the air. Changing the direction of the air supply made it possible to increase the efficiency of the freezing system 1.5-2 times. In order to prevent obstruction of the columns, a wooden roof was built over the freezing columns. The temperature difference between the outside air in the gallery under the roof from which the air was drawn and the air leaving the columns averaged 5.4°C.

As has already been mentioned, natural freezing of the downstream slope of the dam did not take place in spite of the negative temperature of the outside air because of the thick snow cover on the downstream slope.

In 1953 a refrigerator based on M. M. Krylov's system was built on the downstream slope. This assured reliable freezing of the downstream slope of the dam. Figure 5-38 which is borrowed from the paper by S. G. Tsvetkova [97], shows the thermal regime of this dam before and after building a refrigerator on the downstream slope.

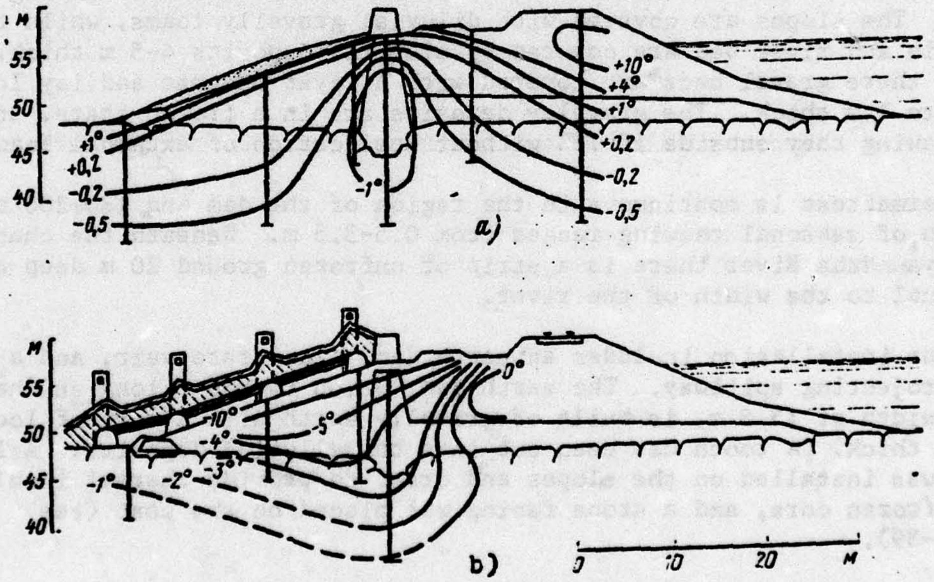

Figure 5-38. Temperature field in the body of the dam on the Dolgaya River.
a - Before construction of the refrigerator; b - After construction of the refrigerator on the downstream slope.

Fifteen observation holes in two lines were installed to make systematic observations on the thermal regime of the body and foundation of the dam.

The temperature regime only reached a steady state 13 years after the dam was placed in operation [27].

2. A dam was built on the Myaundzha River in 1952-1959 to provide a cooling reservoir for a thermal electric power station. The dam impounded a reservoir of 4,400,000 m^3.

The Myaundzha River empties into one of the tributaries of the Kolyma River in its upper reaches. The river is fed by snowmelt in the spring and by rains in the summer. A flow of 338 m^3/sec with 1% frequency was used in planning the spillway. During the winter open flow of the river stops entirely and the river freezes to the bottom.

The average annual precipitation is no more than 270 mm (75% of the precipitation falls in summer and fall). The snow cover is 350 mm deep. The mean annual air temperature in the region of the dam is -12.7°C. The highest mean monthly temperature of +16.6°C is observed in June, and the lowest (-44°C) is observed in January. The absolute minimum temperature is -55°C.

The bedrock in the foundation of the dam consists of basalts, andesites, and andesitic basalts which have been weathered to varying degrees. The slopes are covered with diluvial gravelly loams, while the floodplain and river bed are covered by alluvial deposits 4-5 m thick. Here and there gravel beds are covered with a layer of peat and icy loamy sand up to 1 m thick. The gravelly deposits are in a frozen state, and after thawing they subside 10-15% without application of external load.

Permafrost is continuous in the region of the dam and 180-200 m thick. The depth of seasonal thawing ranges from 0.5-3.5 m. Beneath the channel of the Myaundzha River there is a strip of unfrozen ground 20 m deep and wide, equal to the width of the river.

The installation includes an earth dam, a concrete weir, and a bypass with a projecting spillway. The earth dam, which is 850 m long and has a maximum width of 15.5 m, is built of gravelly earth with a core of local loam 6 m thick. A tooth has been cut into the alluvial deposits. A layer of peat was installed on the slopes and crest to provide thermal insulation for the frozen core, and a stone facing was placed on the peat (see Figure 5-39).

The supporting prisms of the dam were built of gravelly material (rough-rolled chunks of clayey sandstone of variable grain size, with a density in the naturally bedded state ranging from 1.8-2.0 tons/m^3). The material contained 20-60% coarse gravel, 40-60% gravel, and 10-20% sand with an admixture of silt particles.

The core material of the dam in the borrow pit had the following grain-size distribution: gravel and gruss 40-50%, and fine earth with particles smaller than 0.25 m, 15-20%. The plasticity index ranged from 8.5-9.3. The thickness of useful material in the borrow pit was no more than 1 m.

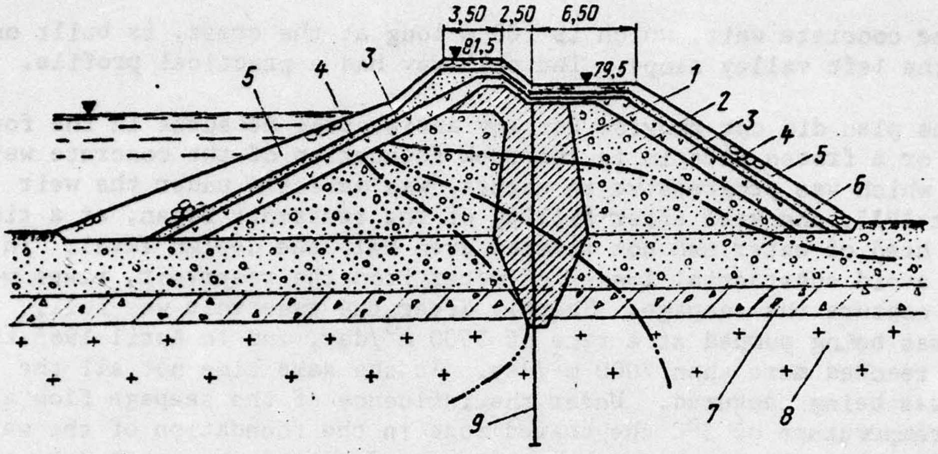

Figure 5-39. Dam on the Myaundzha River
1 - Core; 2 - Sandy-gravelly material; 3 - Peat; 4 - Sand; 5 - Riprap; 6 - Alluvial sandy-gravelly material; 7 - Weathered rock; 8 - Fissured rock; 9 - Frozen curtain.

The loam was recovered from the borrow pit with bulldozers a layer at a time as it thawed naturally and was packed into the tooth of the dam in the thawed state at positive air temperatures. The loam in the lower part of the tooth was tamped manually, and thereafter the earth was compacted with bulldozers or 10-ton rollers. Winter placement of cohesive earth was attempted in January-March 1953 at negative temperatures from -17° to -52°C [33]. Frozen clay taken from the borrow pit was thawed at the placement site using a primitive stove heater: about 65-70 m^3 were thawed in 10 hours. Then the thawed earth was placed in the tooth in 10-cm layers and compacted manually. Up to 10% frozen lumps were added to the thawed ground. The placement site was not heated, and before the next layer was put in place the previous layer had frozen. In spite of careful supervision of the work, the packed ground was so saturated with water that it turned into a liquid mass and was later replaced.

A freezing system consisting of 494 columns was sunk 1-2 m into the rock foundation. The distance between the freezing columns is 1.5 m in the stream bed area and 2 m in the remainder of the dam. The columns are 16-24 m high.

The freezing columns consist of of pipes 100 and 57 mm wide. The inside pipes are attached to a metal plenum from which air is evacuated by a fan at a rate of 10,000 m^3/hr. The air flow through one column is 200 m^3/hr. One fan serves 50 columns.

With the freezing system operating in winter the radius of the zero isotherm at a depth of 10 m was 4 m with an average rate of earth freezing of 2 mm/day.

The concrete weir, which is 200 m long at the crest, is built on rock at the left valley slope. The spillway had a practical profile.

The plan did not provide for any antiseepage measures in the form of grouting or a frozen curtain in the rock foundation of the concrete weir. Seepage, which was progressive in nature, was detected under the weir in August 1955, one year after filling of the reservoir began, at a time when the head of water had not reached even half the design level. In order to avoid substantial losses of water from the reservoir, steps were taken to capture the seepage. Shortly after the reservoir was full, seepage was being pumped at a rate of 3500 m^3/day, and in April 1965 the rate had reached more than 7000 m^3/day. At the same time not all the seepage was being captured. Under the influence of the seepage flow at a water temperature of 5°C the thawed zone in the foundation of the weir increased rapidly in depth and began to spread toward the earth dam, also involving the foundation of the adjacent wall.

Since the thawing began to threaten the integrity of the structures, the foundation was quickly grouted and the freezing system was strengthened. In addition a loam apron 0.3-0.5 m thick was installed in the area of the weir. This became a blanket seal in the river-channel area of the earth dam and the lower slope was faced with gravel.

At the end of August 1957 the air cooling system was replaced by a continuously operating brine system in the river bed area of the dam. A second row of freezing columns spaced 1.5 m apart was installed 2 m from the first row.

These steps reduced the seepage flow to 1500 m^3/day and stopped the growth of the thawed zone under the earth dam.

In 1961 additional deep grouting was applied to the thawed foundation of the earth dam below the frozen curtain and also to the rock foundation of the weir. This tended to reduce the speed and volume of the seepage and made it possible to create a continuous ice-earth curtain under the earth dam. But the temperature field in the body and foundation of the dam at the beginning of 1962 had been stabilized only in those areas where no seepage was observed.

The zero isotherm in the body of the earth dam and its foundation seven years after the beginning of freezing is shown in Figure 5-40.

In our opinion the principal defect in the plan for this installation that became apparent during operation was that the dam and the water-escape structure were planned and built with different temperature regimes to be stabilized during operation. The concrete weir was built without preserving the permafrost in the foundation, and the earth dam was built with the permafrost preserved. As a result the rapid and deep thawing of the permanently frozen fissured rock in the foundation of the weir began to have a warming effect on the ice-earth wall in the earth dam itself. Elimination of this influence required major repair work and considerable investment.

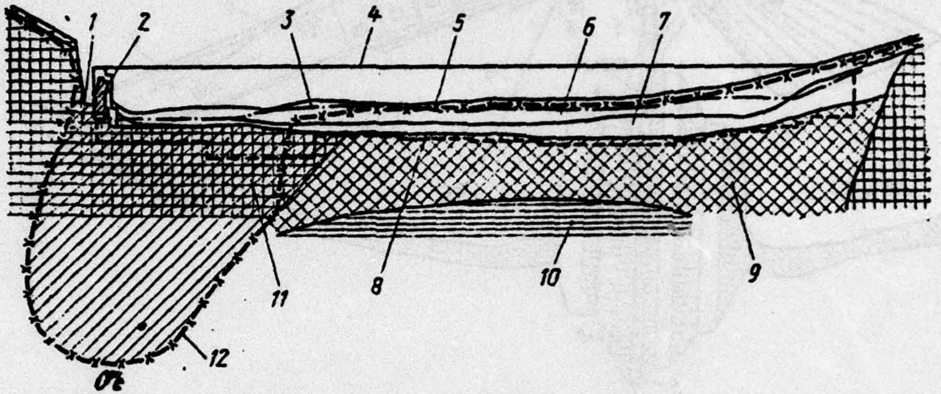

Figure 5-40. Thawing in the foundation of the dam on the Myaundzha River.
1 - Bypass; 2 - Spillway; 3 - Diluvium; 4 - Crest of dam; 5 - Natural ground surface; 6 - Alluvium; 7 - Eluvium; 8 - Depth to which freezing columns were sunk; 9 - Andesites; 10 - Argillites; 11 - Andesitic basalts; 12 - Limit of foundation thawing.

Experience in the construction and operation of this dam is very instructive and is definitely of interest to those building dams. The facilities have now been operating for more than 20 years, but it is still not possible to say that an established temperature regime exists so that the question of the reliability of the dam cannot be considered entirely solved.

A defect in the design of the dam is that the tooth of the core does not extend deep enough into the foundation: the tooth is attached to highly ice-saturated alluvial deposits and was not cut into the bedrock.

Another unfortunate feature of this dam is the system for supplying air. Air is forced into a header and through the header to the inside pipe of a freezing column. As studies on the Irelyakh Dam have shown, in this sort of air supply the air is heated as much as 10°C, which, of course, reduces the efficiency of the system.

3. <u>A dam with a frozen curtain was built on the Irelyakh River</u> in 1964 to create a reservoir for household and industrial purposes at the town of Mirnyy [8], and it is properly considered unique among dams built in this country which preserve permafrost in the foundation. The design and the methods used to build it were based on the specific natural and climatic conditions in the region where it was built (see Figure 5-41).

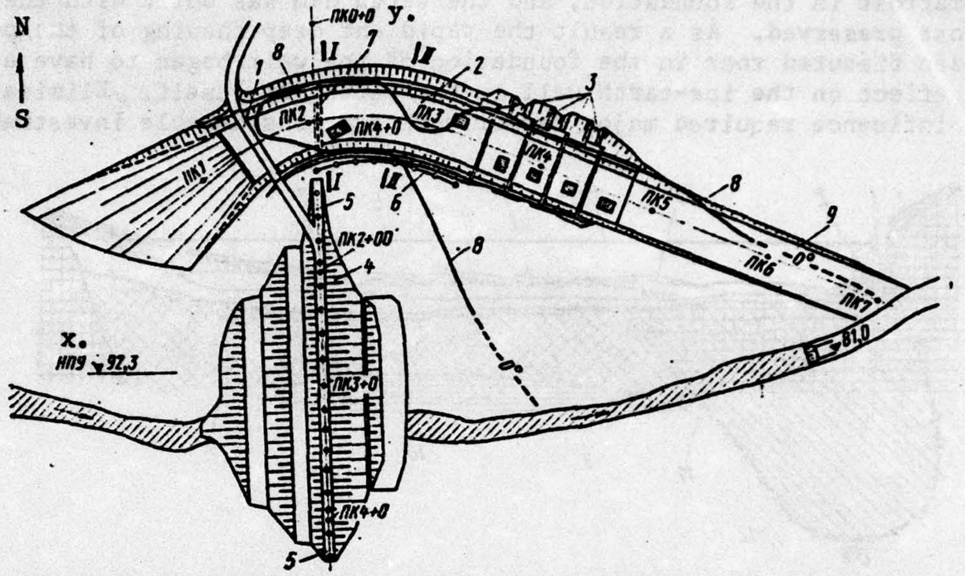

Figure 5-41. Plan diagram of the installation for a permanent reservoir on the Irelyakh River.
1 - Automobile bridge; 2 - Sluiceway; 3 - Cascade walls; 4 - Dam; 5 - Frozen curtain of the dam; 6 - Frozen curtain along the side of the sluiceway; 7 - Frozen curtain under the sluiceway; 8 - Outline of the thawed zone; 9 - Tail race.
Key: x - NHL; y - CPO+O, etc.

The average annual temperature in the vicinity of the construction site is -8.2°C. The daily amplitude of air temperature fluctuations in mid-August reaches 33°C (+22.6°C during the day, and -10.5°C at night). There is little precipitation; the average long-term precipitation does not exceed 250 mm/yr and this falls mainly in the spring and summer.

The Irelyakh River is fed mainly by surface runoff. The river is characterized by a spring flood, low water in the summer, and high water from showers in the summer and fall. From 50-90% of the annual discharge occurs during the summer (May-June). During the winter, beginning in October-November channel flow practically ceases and the river freezes up completely. A zone of thawed ground up to 10 m thick persists under the river bed, however, and supports the flow of ground water.

The bedrock in the foundation of the dam consists of marls and marly clays with layers of limestones, dolomites, and dolomitized rocks bedded in the form of separate layers 0.5-0.8 m thick and disintegrated to a depth of 40 m (see Figure 5-42). The cracks in the rock are filled with ice in the form of veins ranging in thickness from 0.6-1.0 cm to 6-8 cm (see Figure 5-43).

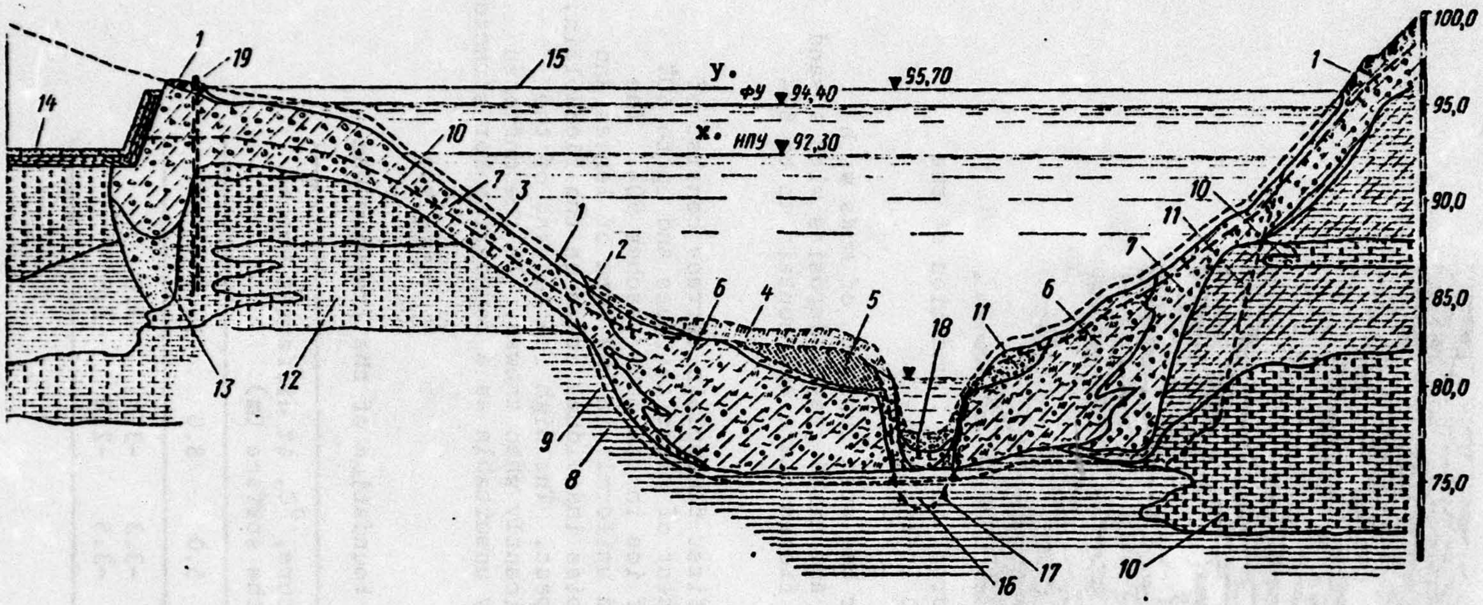

Figure 5-42. Geological profile of the foundation for the permanent reservoir dam on the Irelyakh River.
1 - Topsoil; 2 - Natural ground surface; 3 - Limit of seasonal thawing; 4 - Peat excavation; 5 - Mud excavation; 6 - Muddy loams, ice content 60%; 7 - Loams with gravel, fragments of bedrock, ice content 20-60%; 9 - Outline of bottom of trench under the tooth of the dam; 10 - Fissured dolomites; 11 - Fissured marls; 12 - Thinly stratified limestones; 13 - Old river bed filled with large and small gravel; 14 - Sluiceway channel; 15 - Crest of dam; 16 - Thawed zone below channel; 17 - Boundary of thawed zone below channel; 18 - Alluvial deposits cleared away in the river bed; 19 - Frozen cutoff along margin of sluiceway channel.
Key: x - NHL; y - WL.

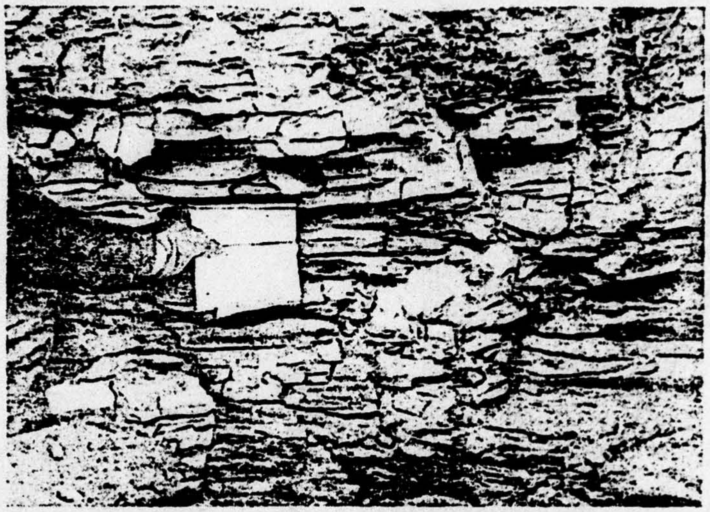

Figure 5-43. Fissuring of bedrock in the foundation of the permanent reservoir dam on the Irelyakh River

Alluvial deposits in the river bed area consist of muds with large and small gravel, muddy loams, and mixed gravel deposits with a sand filler, which are in the permanently frozen state and contain up to 60% ice.

The alluvial deposits on the first floodplain terrace consist of muddy loams and muds with a high content of plant residues and up to 60% ice. The content of various types of ice in the muds reaches 90%. The ice in these materials is distributed uniformly in the form of lenses and layers up to 0.5 m thick. Peat occupies individual areas on the floodplain, with lenses of fossil ice under the peat. The high ice content of the ground means that it subsides significantly when thawed. In the unfrozen state these materials are practically unsuitable as a foundation for structures.

Table 5-2

Ground Temperature in the Foundation of the Irelyakh Dam

Period of observation	Ground temperature, °C, at different depths below the surface (m)				
	1.0	3.0	5.0	8.0	10.0
October	-0.1	-2.3	-3.3	-3.6	-3.0
February	-9.8	-6.0	-3.5	-2.8	-2.8

Table 5-3

Ground Temperature in the Foundation of the Irelyakh Dam

Slope exposure	Ground temperature, °C, at different depths below the surface, m					
	1.0	2.0	4.0	8.0	12.0	1.50
South	-11.2 - -2.8	-11.2 - -0.2	-5.6 - -1.5	-3.1 - -2.1	-2.2 - -2.6	-2.6 - -1.9
North	-14.6 - -0.6	-11.2 - -1.3	-8.4 - -2.2	-4.0 - -2.8	-3.3 - -2.6	-3.3 - -2.8

Data on temperatures in the frozen ground are given in Table 5-2 and data showing ground temperatures as a function of slope exposure at different depths are presented in Table 5-3.

The facilities include an earth dam with a maximum height of 20 m and a length at the crest of 320 m (see Figure 5-44), a sluiceway channel, and an automobile bridge, the piers of which serve as an ice-protective wall.

The dam was planned with a flattened upper half-profile (the dam is 8 m wide at the top and 115 m wide at the base), since after filling the reservoir it is expected that the foundation materials under the upstream slope will thaw and subside. At the same time, of course, the upstream slope of the dam will unavoidably be deformed due to the thawing and settlement of the earth, and therefore will assume some new position approximating a straight dam slope without a berm. It was assumed in planning that over a period of 50 years the ground in the bed of the reservoir will thaw to a depth of 10 m, and that the downstream slope of the dam will freeze over its entire height in 30 years as a result of the effect of outside air temperatures, the negative temperature of the permafrost in the foundation, and the frozen curtain.

The central part of the dam was built of unfrozen loam, and the lateral topping prisms of fine-grained sand. The upstream slope of the dam was stabilized with a layer of stones 1.0 m thick, while a layer of 0.5 m thick was applied to the downstream slope.

Statistics on the work are as follows (in thousands of cubic meters):

Excavation	449
Including rock	214
Fill	600
Including high quality fill	376
Stone facing of slopes	20
Concrete, sectional and monolithic	13
Boring of holes for the freezing system (meters)	11,500

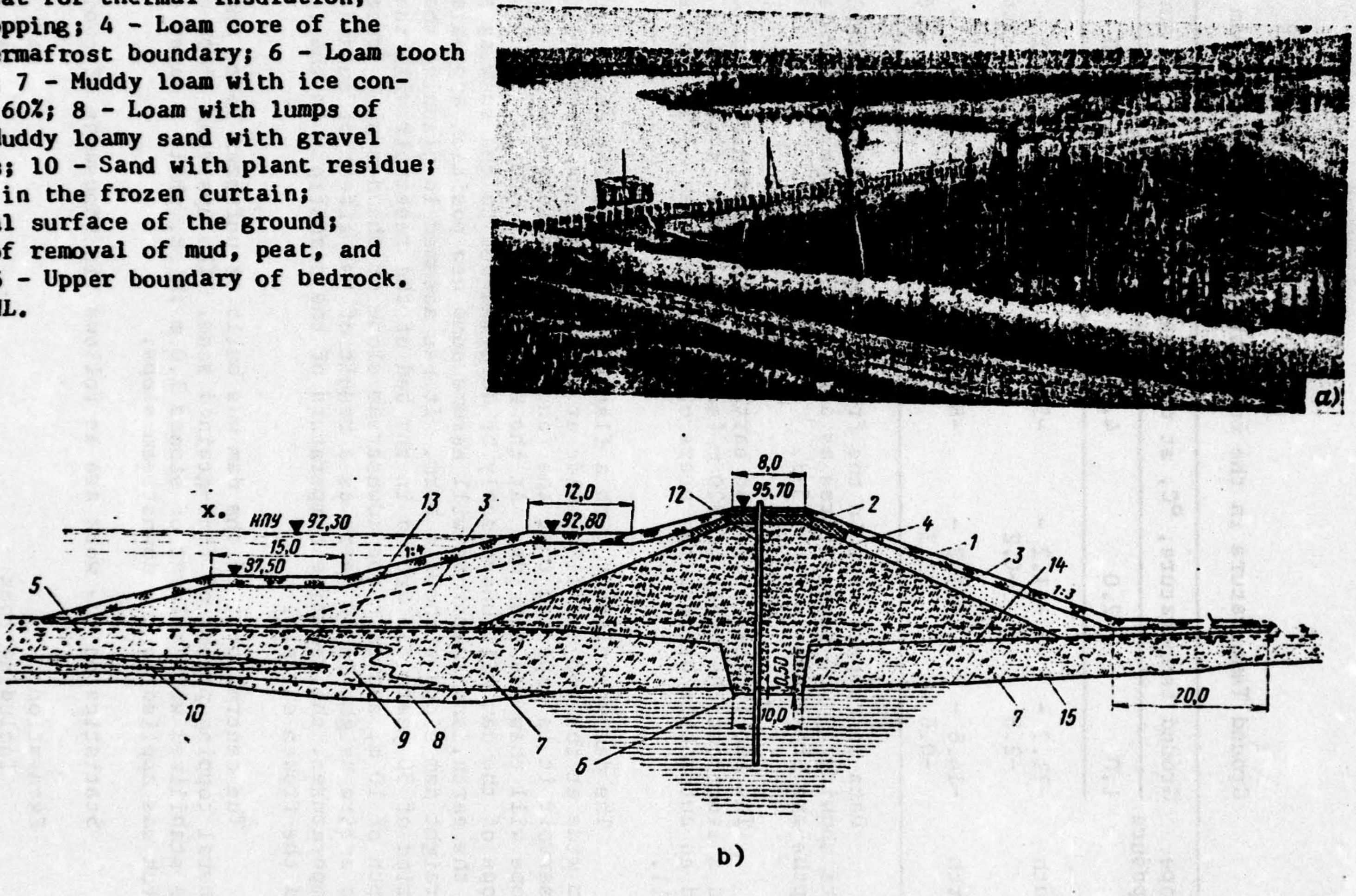

Figure 5-44. Permanent reservoir dam on the Irelyakh River.
a - Overall view; b - cross-section.
1 -- Stone facing of slope; 2 - Layer of moss and peat for thermal insulation;
3 - Sand topping; 4 - Loam core of the dam; 5 - Permafrost boundary; 6 - Loam tooth of the dam; 7 - Muddy loam with ice content up to 60%; 8 - Loam with lumps of marl; 9 - Muddy loamy sand with gravel and cobbles; 10 - Sand with plant residue;
12 - Holes in the frozen curtain;
13 - Natural surface of the ground;
14 - Line of removal of mud, peat, and topsoil; 15 - Upper boundary of bedrock.
Key: x - NHL.

A layer of moss for thermal insulation 1.0 m thick was applied to the crest and the downstream slope. It was initially planned also to build a thermal insulation shield on the downstream slope by freezing ice on top and covering it with sawdust in the summer. It was planned to wet down the sawdust with a lime solution for better reflection of the direct rays of the sun, but this decision was reversed during construction.

At our suggestion changes were introduced into the design during construction. Specifically, a tooth was planned to join the body of the dam to the foundation, which in the floodplain and channel areas would cut completely through the unconsolidated Quaternary ice-saturated material into the bedrock a distance of 0.5 m. This made the dam impervious to seepage and provided more favorable conditions for freezing the body and foundation of the dam during operation.

The dam was built of unfrozen materials but without filters and drains, and it was subsequently frozen using an air freezing system consisting of 207 freezing columns grouped in five cooling systems (see Figure 5-45). The holes for the frozen curtain were from 8.2-26 m deep and spaced 1.5 m apart. They were extended 3 m into the bedrock.

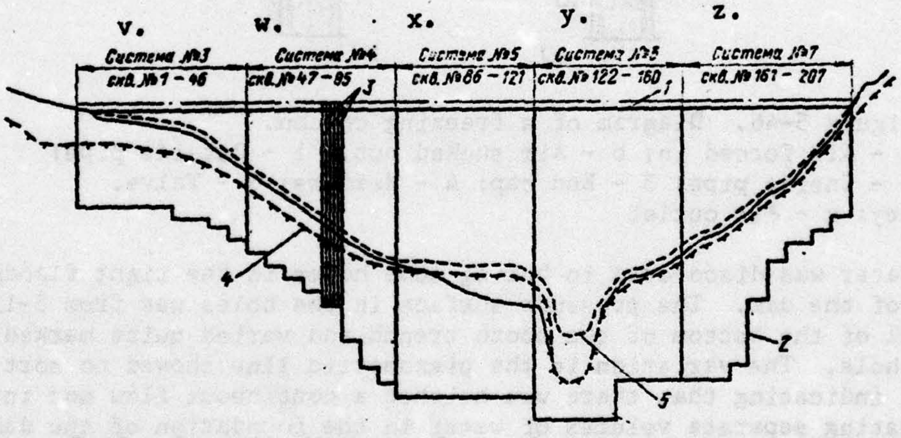

Figure 5-45. Overall diagram of the frozen curtain in the dam on the Irelyakh River.
1 - Crest of dam; 2 - Level of penetration of freezing columns; 3 - Freezing columns; 4 - Permafrost boundary under natural conditions; 5 - Bottom of river.
Key: v - System No. 3, holes No. 1-46; w - System No. 4, holes No. 47-85; x - System No. 5, holes No. 86-121; y - System No. 5, holes No. 122-160; z - System No. 7, holes No. 161-207.

A freezing column consists of two pipes -- an outside pipe 219 mm in diameter and an inside pipe 189 mm in diameter (see Figure 5-46). The outside pipe is hermetically sealed at the lower end and placed on the bottom of the hole. The inside pipe is installed in such a way that a space about 20 cm high is created between its lower end and the end cap of the outside pipe. Air passes from the annular space between the pipes into the inside pipe, moves up this pipe, and is expelled into the atmosphere.

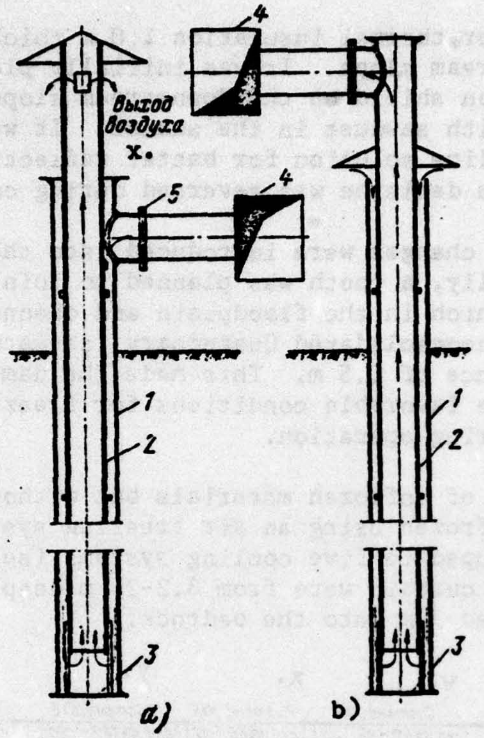

Figure 5-46. Diagram of a freezing column.
a - Air forced in; b - Air sucked out. 1 - Outside pipe;
2 - Inside pipe; 3 - End cap; 4 - Headers; 5 - Valve.
Key: x - Air outlet

Water was discovered in boring some holes in the right floodplain segment of the dam. The pressure surface in the holes was from 5-12 m above the level of the bottom of the tooth trench and varied quite markedly from hole to hole. The variation in the piezometric line showed no sort of regular pattern, indicating that there was neither a continuous flow nor intercommunicating separate volumes of water in the foundation of the dam. Evidently the appearance of water in the boreholes is the result of change in suprapermafrost ground water conditions within the dam construction area. After the dam foundation was cleared and the trench was dug for the tooth, the icy ground thawed to a greater depth than in neighboring areas, and local thawed saucers formed. This increased the flow of ground water from the surrounding area to the right-bank juncture of the dam, an effect intensified by the fact that the ground of the right-bank slope, with its northern exposure, had a high content of ice. Ground water flowed into the bottom of the trench as cracks opened. Thus it can be concluded from what we have said that in building dams on permafrost we encounter the formation of a localized hydrogeological regime.

Analysis of the altered hydrogeological conditions in the foundation of the dam showed that the water found in the holes bored for the freezing columns was in a state of rest and would not cause serious difficulties in freezing the dam, merely extending the time necessary to accomplish the freezing.

Experinece with the freezing of the core of the Irelyakh Dam and its operation has shown that the presence of ground water in the foundation of the tooth had no effect on the progress of freezing and the reliable operation of the frozen curtain.

The operation of the freezing system was monitored by measuring the flow of air and the temperatures of the air entering and emerging from the holes. The flow of air was measured for most of the columns, but temperature differences were determined only for characteristic columns in the system located at the ends and in the middle of each system, as well as in columns adjacent to the fan. The results of the measurements were used to plot the relationship $t_{out} = f(t_{o.a.})$ (Figure 5-47). Using this relationship it is possible to obtain data on temperature differences. In some cases the temperature differences, especially at the end of the freezing period, are positive in sign, indicating that during this period heat is carried into the freezing system.

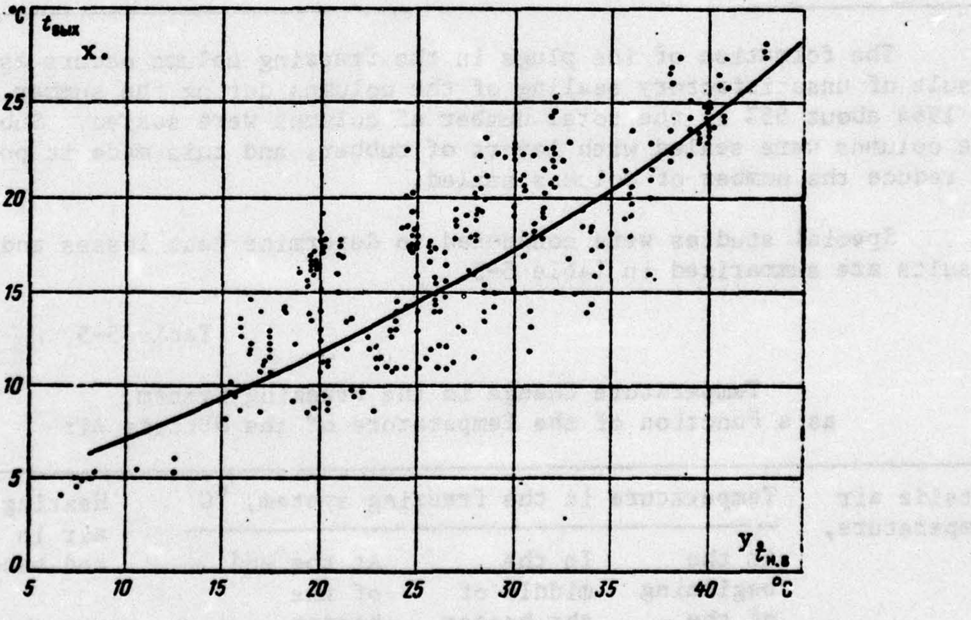

Figure 5-47. Temperature of the air emerging from the columns (t_{out}) as a function of the temperature of the outside air ($t_{o.a.}$).
Key: x - t_{out}; y - $t_{o.a.}$.

The temperature drops in the freezing columns are functions of many variables: the temperature and flow-rate of the outside air, the type and temperature of the earth, the length of the column, and the diameter of the ice-earth cylinder. But during the day it may depend mainly on the outside air temperature alone.

Table 5-4 gives temperatures of the outside air and of the air issuing from the columns, as well as the temperature drops for three winter periods of dam freezing. As is apparent from the data in Table 5-4, the temperature drop depends mainly on the temperature of the outside air.

Table 5-4

Air Temperature Drops in the Freezing Columns

Index	Year of observation		
	First	Second	Third
Average temperature of the outside air during the freezing period, °C	-23.2	-32.5	-28.5
Average temperature of the air emerging from the columns, °C	-11.8	-19.6	-16.5
Temperature drop, °C	11.4	12.9	12.0

The formation of ice plugs in the freezing column occurs as the result of unsatisfactory sealing of the columns during the summer. Thus, in 1964 about 55% of the total number of columns were sealed. Subsequently the columns were sealed with layers of rubber, and this made it possible to reduce the number of columns sealed.

Special studies were conducted to determine heat losses and the results are summarized in Table 5-5.

Table 5-5

Temperature Change in the Freezing System as a Function of the Temperature of the OUtside Air

Outside air temperature, °C	Temperature in the freezing system, °C			Heating of the air in the fan and header
	At the beginning of the header beyond the fan	In the middle of the header	At the end of the header	
-13.6	-11.1	-11.5	-10.3	-3.3
-12.4	-11.5	-11.1	- 9.7	-2.7
-25.0	-23.8	-23.3	-22.5	-2.5

Growth in thickness of the ice-earth wall was monitored with mercury inertial thermometers and electrical resistance thermometers graduated in degrees and tenths of a degree Celsius. An array of five thermometers was lowered into the test hole at least once a day. Some of the test holes for measuirng temperature in the body of the dam were equipped with electrical resistance thermometers spaced 2-5 m apart. The temperature field in the Irelyakh Dam is shown in Figure 5-48.

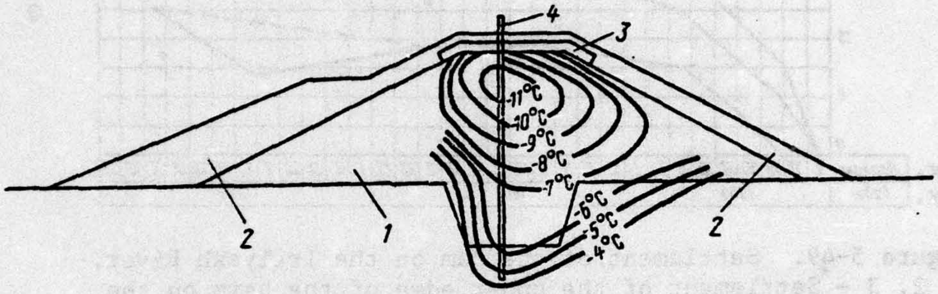

Figure 5-48. Temperature field in the dam on the Irelyakh River after six years of use.
1 - Loam core; 2 - Sand covering; 3 - Moss-peat layer; 4 - Frozen curtain holes.

The data show that minimum temperatures were observed in the earth after the freezing period was finished and maximum temperatures before the freezing systems were started up at the beginning of winter. Maximum temperatures in 1966 were significantly below those in 1965: before startup of the freezing system in the left-bank floodplain segment a maximum temperature of -0.9°C was observed in 1965, while in 1966 the same maximum temperature was -3.0°C. The temperature in the lower part of the holes was about -3.0 to -4.0°C winter and summer.

It must be reported that the ice-earth cylinders did not touch completely during the first year of operation. An unfrozen "window" persisted in the left-bank floodplain segment, but during the next winter a continuous wall of frozen ground 6 m thick was created. The wall thickness reached 10 m after the freezing system had been in operation for three years, so that the operating time of the system could be reduced.

During the first three years of use the system was placed in operation at outside air temperatures below -15°C, and thereafter only when air temperatures were -20° to -25°C.

Data on the subsidence of the dam are presented in Figure 5-49.

4. A dam with a frozen curtain was built on Pavek Creek to provide a water supply for the Pavek settlement. The facility included (see Figure 5-50) a dam 464 m long at the crest, 6 m wide, and with a maximum height of 21.4 m; a spillway constructed along the left-bank slope, and

a water-intake facility consisting of a reinforced concrete turret well adjoining the upstream slope. The water intake was connected to the crest of the dam by a dike.

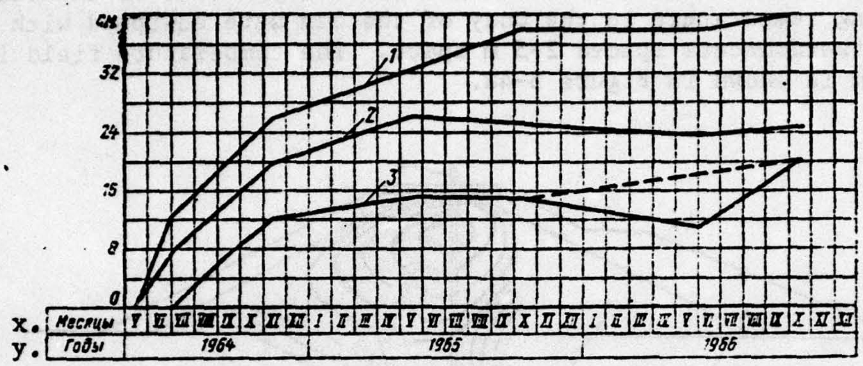

Figure 5-49. Settlement of the dam on the Irelyakh River.
1, 2, 3 - Settlement of the upper edge of the berm on the pressure slope.
Key: x - Months; y - Years.

The plan called for making the core of loam fill and the lateral prisms of loamy sand with gravel, but the dam as built was homogeneous, consisting only of loamy sand with gravel. The core was cut into the foundation to a depth of 1 m. The gradient of the upstream slope is 1:2.5, and that of the downstream slope 1:1.75.

The bedrock, clay shale, is found at depths of 2-8 m on the left-bank slope, and at depths of 8-30 m on the right bank. The materials on the right bank, which has a southern exposure, are more weathered. The loose deposits covering the bedrock on the slopes consist of gravelly and lumpy earth containing loamy sand and a considerable amount of ice.

On the right slope there are lenses of loamy sand with some mixed gravel, layers of gravelly-bouldery sand, and thick lenses of ice. The ice content of the Quaternary deposits varies from 13-87%. Subsidence of the earth after thawing is considerable.

The air freezing installation consists of a single row of vertical freezing columns spaced 2 m apart (see Figure 5-51). The holes are from 12-29 m deep. The outside pipes of the freezing columns are 150 mm in diameter, and the inside pipes 100 mm. The columns are grouped into three systems, each with its own 16,000 m^3/hr fan.

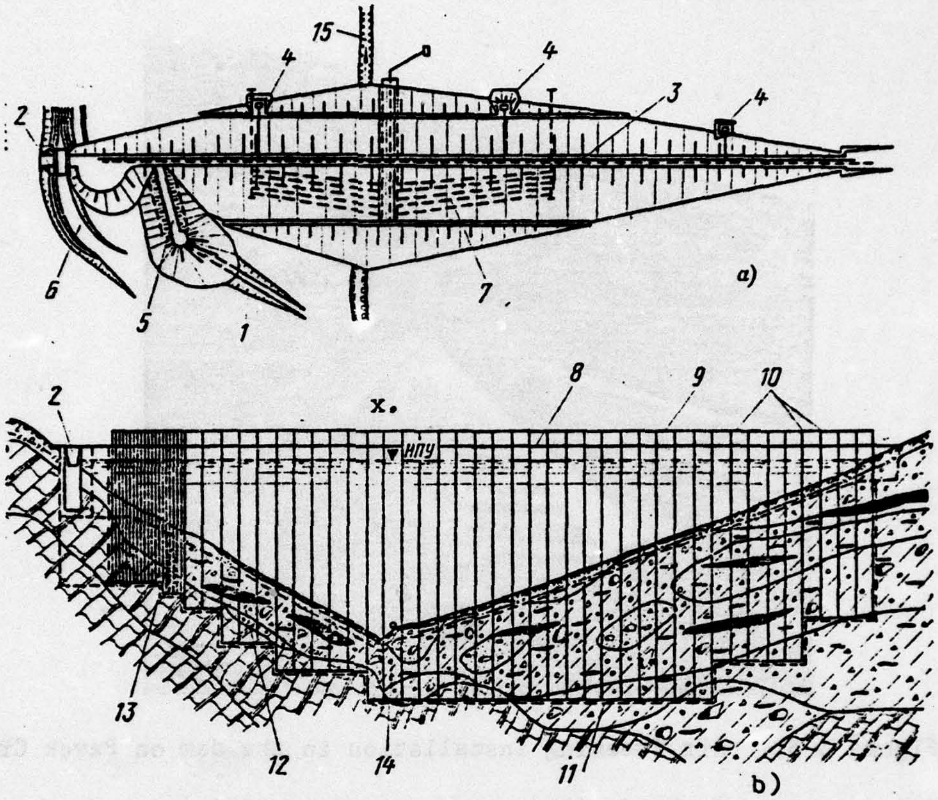

Figure 5-50. Installation on Pavek Creek.
a - Plan view; b - Longitudinal section along the axis of the dam. 1 - Supply channel for the bottom intake-discharge; 2 - Spillway; 3 - Frozen curtain; 4 - Fans; 5 - Crown of the water intake; 6 - Intake area of the spillway; 7 - Freezing system pipes; 8 - Crest of the dam; 9 - Header; 10 - Freezing columns; 11 - Insertion depth of the frozen curtain; 12 - Ice; 13 - Trench beneath the conduit of the water intake; 14 - Pipe to eject construction discharge; 15 - Creek bed.
Key: x - NHL.

The spillway was originally planned to be a self-regulating bypass around the dam on the left slope. It would have a trapezoidal profile, a bottom width of 2 m, 1:0.5 slopes, and would be designed to handle a flow of 5 m^3/sec. As finally built, however, the facility was planned with a concrete weir with a practical profile 4.2 m high and with aprons to absorb the force of the flow.

The counteract seepage it was planned to grout the rock foundation under the spillway of the water-escape installation and to construct a concrete cutoff wall in the left junction of the dam.

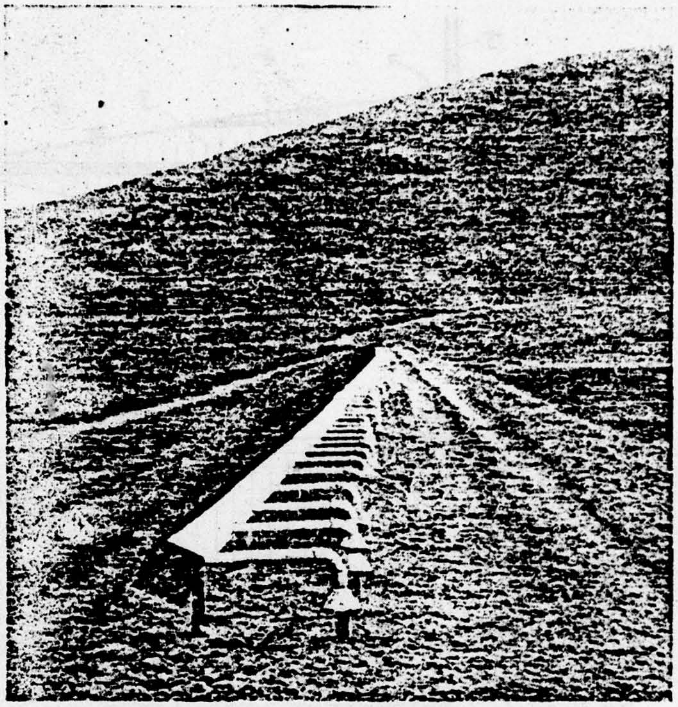

Figure 5-51. Air freezing installation in the dam on Pavek Creek.

Construction of the facility was started in 1960 and the main work, except for the freezing of the foundation and body of the dam, was completed in 1966. Only 112 of the planned 210 freezing columns had been installed at the time the reservoir began to fill (in the spring of 1967). Hence the dam was frozen only in the central area with a short freezing time. Test holes to observe temperature conditions in the foundation and body of the dam were not provided. Therefore it has not been possible to determine the extent of development of the frozen curtain in the body of the dam when filling of the reservoir started and to judge its condition during operation. Nevertheless the dam was placed in operation in October 1969.

5. A dam with a frozen curtain wall was built on Oyuur-Yurege Creek where it emerges from Lake Oyuur-Kyuyel'. It creates a reservoir for household and industrial water supply needs [91].

The installation includes, in addition to a dam, a spillway, an ice-retaining structure, and stream-directing dikes. A spillway was built along the left slope to carry flood waters around the dam. The dam is 17 m high, 660 m long along the crest, and 18 m wide at the crest. The crest is wide to accommodate piping for the frozen curtain and an automobile road (see Figure 5-52).

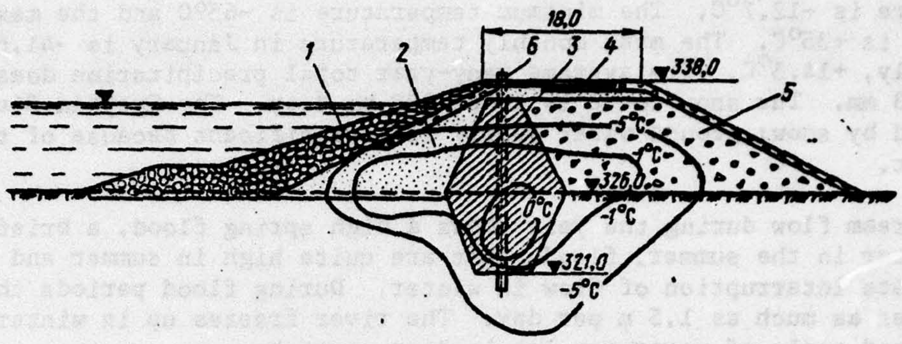

Figure 5-52. Dam on Oyuur-Yurege Creek.
1 - Stone; 2 - Loamy sand; 3 - Core; 4 - Road; 5 - Supporting prism of semiconsolidated material; 6 - Frozen curtain.

The supporting prisms of the dam are built of semiconsolidated marly rock and limestones as well as weathered diabases. The core of the dam, which is 5-12 m wide, is cut into the bedrock a distance of 6-7 m. The core is made of fine loamy sand material with the following granulometric composition:

Particle size (mm)	Content (%)	Particle size (mm)	Content (%)
>10	21.8	0.25-0.1	2.2
10-5	2.5	0.1 -0.05	1.2
5-2	5.7	0.05-0.01	25.8
2-1	2.5	0.01-0.005	11.5
1-0.5	0.7	<0.005	24.8
0.5-0.25	1.3		

Figure 5-52 shows the temperature field in this dam during the first year of operation. A zone with positive temperatures persisted in the core. The persistence of the unfrozen zone evidently is explained by the fact that the tooth of the core was built partly of frozen loamy sand material and during a flood was inundated over almost its entire length, so that the earth was heavily saturated with water. The body of the dam froze completely during the winter of the second year of operation.

The dam is in good condition at the present time.

6. A dam with a frozen curtain on the Sytykan River in the Yakut ASSR creates a reservoir of 35,000,000 m^3 for domestic and industrial water-supply purposes. This is the largest dam in the USSR built on permafrost with a frozen curtain.

The construction site is located directly on the Arctic Circle. The climate of the region is strongly continental with a long cold winter, a short summer, and brief transition periods. The average many-year air temperature is -12.7°C. The minimum temperature is -65°C and the maximum in summer is +35°C. The mean monthly temperature in January is -41.6°C, and in July, +14.5°C. The average many-year total precipitation does not exceed 273 mm. The snow cover is about 350 mm deep. The Sytykan River is mainly fed by snow; ground water supply is insignificant because of the permafrost.

Stream flow during the year shows a high spring flood, a brief period of low water in the summer, floods that are quite high in summer and fall, and complete interruption of flow in winter. During flood periods the water level rises as much as 1.5 m per day. The river freezes up in winter, but disconnected pools of water persist in deep stretches.

Since observational data on the hydrology of the Sytykan River were not available when planning started for the dam, runoff parameters at the dam site were established by an analogue technique. Discharge calculated by this method was as follows:

Frequency (%)	0.1	0.5	1	3	5	10
Discharge (m^3/sec)	398	347	322	284	266	236

The bedrock in the foundation of the dam consisted of clay-carbonate deposits in the form of limestones, thinly interstratified dolomites, dolomitized limestones, marls, and calcareous sandstones. The thickness of the individual strata varies from several centimeters to 1.5-2.0 m. These beds are irregular and their bedding is mainly horizontal.

Diluvial and alluvial deposits cover the bedrock. The diluvial deposits consist of loams with large gravel and platy fragments of calcareous rock. The alluvial deposits contain fine sands, cobbles and gravel, and vary in thickness from 1.3 to 5.0 m on the floodplain and the left bank terrace of the river. One frequently finds lenses and layers of ice up to 2-4 cm thick in the alluvial deposits; the natural moisture content of these deposits is 10-74% and the bulk density of the skeleton is 1.6-1.96 tons/m^3. When thawed their subsidence is up to 25-30 cm per meter of thawed material. Scattered thermokarst sinkholes may form during thawing, and this might involve large seepage losses of water from the reservoir. The earth on the left floodplain and slope is highly supersaturated; the unconsolidated deposits contain large amounts of buried ice in the form of individual lenses with a maximum thickness reaching 1.8 m. The loams on the right slope range in thickness from 1.1-2.7 m, and on the left bank from 2.0-5.8 m.

The loams have the following characteristics: density 1.3-2.27 tons/m^3, natural moisture content 21.2-93.5%, plasticity index 6.9-10.4, and relative subsidence under a load of 3 kg/cm^2, from 1.8-40% or more. The loams assume a fluid consistency when thawed and subsidence is both considerable and irregular.

In the less weathered semiconsolidated limestones and sandstones ice is distributed in cracks, and when these materials thaw, the rock may shift and subsidence may occur. The extent of the subsidence depends on the extent to which the rock is broken up as well as the moisture content. In some areas subsidence may amount to 10-15 cm per meter of thawed material. The percolation coefficient of the rock when thawed ranges from 50 to 200-300 m/day.

The permafrost is more than 300 m thick. The active layer ranges in thickness from 1.5-2.0 m. The temperature of the frozen ground at the dam site is not uniform, varying over a wide range depending on the composition of the ground, the distance from the river channel, and the exposure of the valley slope.

Ground temperatures based on one-time measurements on the right side (north slope) of the valley and in the floodplain of the river are given in Table 5-6.

Table 5-6

Ground Temperatures in the Foundation
of the Dam on the Sytykan River

Depth, m	Ground temperature, °C				
	Right bank		Floodplain		
	Date of observation				
	Sept. 15	Oct. 15	Sept 15	Oct. 15	Nov. 28
2,0	—1,0	—0,8	+1,0	—0,2	—0,3
4,0	—2,4	—2,4	—0,2	—0,3	—0,7
6,0	—3,0	—3,1	—0,3	—0,7	—1,0
8,0	—3,0	—3,0	—0,7	—0,9	—1,1
10,0	—3,4	—3,2	—0,7	—1,4	—1,2
12,0	—3,0	—3,0	—0,9	—1,3	—1,4
14,0	—3,0	—2,9	—1,1	—	—1,7
16,0	—2,8	—2,6	—	—1,3	—1,3
17,0	—2,4	—2,5	—1,5	—1,4	—1,6

The dam, which is of the frozen type with an air freezing system, is 600 m long with a maximum height of 22 m (see Figure 5-53). The upstream and downstream prisms are built of rock and semiconsolidated material from the excavation for the by-wash channel. The antiseepage element is a core made of loam, which is joined to the foundation with a tooth 2-3 m deep cut into the loam or the Ordovician bedrock.

The granulometric composition of the material in the core of the dam is as follows:

Particle size (mm	Content (%)	Particle size (mm)	Content (%)
>10	18.4	0.5 -0.25	3.5
10-5	2.7	0.25-0.1	3.7
5-3	1.2	0.1-0.05	17.8
3-2	0.8	0.05-0.01	20.7
2-1	0.5	0.01-0.005	9.4
1-0.5	1.6	<0.005	19.7

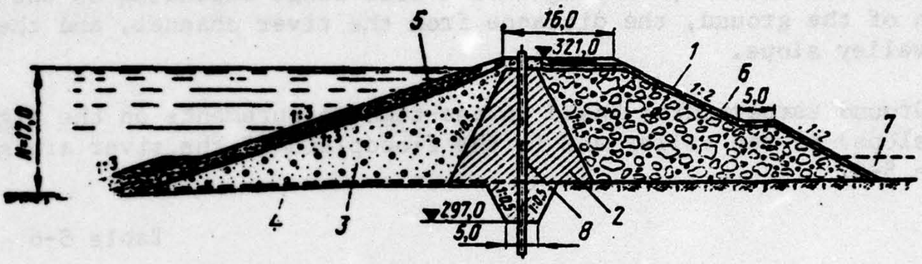

Figure 5-53. Dam on the Sytykan River.
1 - Rock; 2 - Core of gravelly loams; 3 - Sandy-gravelly material; 4 - Rock surface; 5 - Stone facing; 6 - Insulation; 7 - Backfill of river bed; 8 - Frozen curtain.

The gradient of the upstream slope is 1:3 and that of the downstream slope is 1:2.5. There are berms on the slopes. The slopes are faced with riprap on a bed of crushed rock. The crest of the dam is covered with a thermal insulation layer of moss and mixed sand and gravel 0.5 m thick.

In order to release water from the reservoir for sanitary purposes, a siphon discharge was designed of pipe 1000 mm in diameter with a capacity of 6 m^3/sec. One purpose of this facility was to reduce the length of time taken to discharge water through the by-wash, which in turn significantly reduces the probability of thawing beneath it. Water is discharged from the siphon spillway into a speed-arresting well.

Two air cooling systems are being installed to freeze the dam -- a main system and an auxiliary system. The main system assures the creation of a permanent frozen curtain in the dam and its foundation. At the juncture with the right bank it connects to a frozen curtain running along the by-wash channel to the floor of the weir. The system for creating the frozen curtain consists of vertical freezing columns spaced 2 m apart. The diameter of the outside pipes is 219 mm, and that of the inside pipes 133 mm. The freezing columns penetrate 4-7 m below the base of the core tooth. In the area of the thawed zone beneath the river bed the freezing columns are extended deep enough to freeze it completely and cut if off.

The dam contains 290 freezing columns with a maximum depth of 37 m and a minimum depth of 6 m (the average depth of the columns is 20 m). The total length of the boreholes for the freezing columns in the dam is 5650 m. The columns are grouped into eight separate systems, each of which is served by a single fan.

The cold air is fed to the inside pipe from a header 600 m in diameter and escapes to the atmosphere via the outside pipe through a second header.

Calculation shows that 20 m^3/hr of cold air per meter of cooling column is required to freeze the ground and create the ice-earth curtain. The freezing columns are grouped in such a way that 14,000 m^3/hr of cold air is required for each system.

The freezing system is placed in operation when the outside air temperature is below -20°C.

The dam was built in two stages. After it had been raised to an intermediate height of 12 m and the core and foundation of the dam had been frozen, the reservoir was filled with the 1973 spring flood, excess water being spilled over the crest. It is planned to use the dam in this fashion for two years. At the end of the third year of construction, the plan is to raise the dam to the design level, freeze it, and pass the flood waters through the operational bypass.

A system of test holes in three lines are being installed to take measurements on the temperatures in the body and foundation of the dam during operation.

Temperatures in the foundation of the by-wash will be monitored using four rows of four test holes each from 12-25 m deep.

Chapter 6

DAM CONSTRUCTION OPERATIONS

6-1. CHARACTERISTICS OF DAM CONSTRUCTION FROM LOCAL MATERIALS IN THE FAR NORTH

Construction operations in the Far North, especially construction related to the management and use of water, must cope with serious difficulties related to the cold severe winter, the universal presence of permafrost, the lack of knowledge about these regions, and the absence of adequate experience with construction under these conditions. Necessary structural materials are very frequently lacking in the region and they are difficult to provide because of the remoteness and primitive state of the areas.

The industrial materials industry is usually poorly developed in regions where dams and related structures are being built. Permanent transportation routes are lacking. Because the territory is so vast, settlement of these regions is concentrated at particular locations.

All this makes conditions extremely difficult for delivering construction cargoes. Most of the cargoes usually arrive during the short summer navigation season of the rivers, which are used as the highways. Final delivery is made by winter motor roads made on frozen bogs and swamps.

Under these conditions the problem of maximum utilization of local construction materials to build water-management structures is particularly important.

In this region the earth in borrow pits usually is extremely wet. It can be dug and used for construction only if special steps are taken for engineering melioration of the borrow pits.

As has already been mentioned, the warm season in these regions is so short that it usually is not possible to restrict the placement of high-quality cohesive earth fill to the summer season alone. In winter this work either must be delayed for long periods, and this ultimately leads to disruption of the entire progress of operations, delaying completion and raising the cost, or one must find ways to eliminate the seasonal factor from the engineering activities.

The existing technology for building earthen water-retaining structures, both in this country and abroad, which is based on accumulated practical experience, does not permit the packing of cohesive materials at temperatures below zero or it allows such work only within a strictly limited range of negative temperatures (usually no more than -10°C). On the other hand, the remoteness of regions in the Far North and the exceptionally difficult transport situation for delivering structural materials make it necessary to utilize local structural materials to the maximum extent possible in building dams. For this reason the problems of dam construction, the search for methods of prolonging the construction season and packing cohesive earth into high-quality fill at negative temperatures assume primary importance.

Successful construction of dams from local materials in the Far North requires the development and implementation of a technology for placing cohesive earth both in the summer and in the winter at low negative air temperatures.

Year-round use of cohesive earth in dams and particularly in their antiseepage components will permit more efficient use of machinery and trucks, shortening construction time and eliminating the seasonal pattern of work. A technology for year-round placement of cohesive earth had essentially not been developed until quite recently, since experience in building dams of local materials under the severe climatic conditions of the Far North was inadequate.

In recent years water-related construction in the Far North has become quite widespread. Some experience in the placement of cohesive earth acquired in the building of the dam on the Irelyakh River (1962-1963) was used in building the dam for the Vilyuy Hydroelectric Power Station. A technique for placing cohesive earth at extremely low negative temperatures was developed and successfully used on this construction project (1964-1966). Thereafter this technique was used in building the dams for the Khantayka Hydroelectric Power Station (1968-1969) and the dams on the Oyuur-Yurege (1971) and Sytykan (1972) Rivers. Elements of this technique were also used in building the dam for the Bilibino Nuclear Power Station. It is planned to use this technology in building the dam for the Kolyma Hydroelectric Power Station.

The following types of operations are involved in building dams from local materials used with this technology, which has come into wide use: opening the borrow pits; scraping up the earth in the borrow pit and stowing it in ridges to dry and accumulate solar heat, which usually is necessary because of the high moisture content of the earth; breaking up the ridges later on and moving the earth to mounds for winter storage; shielding the outside layer of the mounds, as well as operations concerned with covering the surface of the mounds with a thermal insulation layer and electrical heating of the earth in the mounds; digging and delivering it to the placement site; and finally, installing the earth at the site, which consists of a whole series of operations.

The specificity and complexity of the problems encountered in building dams and related structures also flow from the fact that the conditions and methods used in building them are usually individualized, and in these cases it is necessary to develop special engineering guidelines for carrying out the work which take into account the specific local conditions and involve a range of investigations to determine the methods and means to be used in building the structure, the types of machinery to be used, etc.

The process of placing cohesive earth in the antiseepage elements in a dam during the winter as well as the preparation of this material in the summer involve the one-time use of large numbers of machines. Therefore any interruptions that occur in particular links in the chain necessarily result in much lost time for the machinery and their operators, thereby significantly increasing the cost of the work. Costs can be reduced significantly only by properly organizing the operations for continuous earth placement and high productivity of the entire complex of machinery. Maximum use of machinery and complete elimination of hand work are necessary everywhere. The need to use manual labor usually arises when the technologica chain for placing cohesive earth in a dam is broken.

6-2. PREPARATION OF THE DAM FOUNDATION

The demands imposed on preparing the foundation for a dam are based on the fact that joining the body of a dam, and particularly its antiseepage elements, to the foundation is the most important aspect of its design. The correct design and proper implementation of this work are the basis for reliable operation of the structure, i.e. for the general and antiseepage strength of the dam as a whole.

Everything must be removed from the foundation of a dam which might subsequently affect its strength, the seepage resistance of the foundation, and subsidence.

The volume and nature of the operations involved in preparing the foundation depend to a considerable extent on the method of dam building used, and, of course, on the purpose of the dam, which will determine its performance requirements. Other conditions being equal, the requirements may be different for preparing the foundation under different sections of a dam. Quality requirements are higher for preparing the foundation in the area where the antiseepage elements are joined to it, and they are higher under the upstream wedge than under the downstream wedge. It is often enough to do the minimum amount of work in preparing the foundation under the downstream wedge. Thus alluvial deposits, consisting of mixed gravel deposits and boulders with a total thickness of 3 m, were left in the foundation below the supporting prism of the Vilyuy rock fill dam.

In building the dam for the second-stage tailing pond on N'yukka Creek, only trees and brush were cleared away beneath the supporting prism of the earth dam with blanket seal [20]. Beneath the upstream wedge, where the seal was connected to the foundation, the topsoil and the peat layer were stripped away down to the mineral soil.

It is recommended that the topsoil be removed in the spring as the surface layers thaw and can be removed. At that time, as soon as the snow is gone, bulldozers can be used very successfully to clear away trees and brush.

In deciding how to prepare the foundation for a dam with a frozen antiseepage curtain on the Irelyakh River, several alternative methods of preparation were considered (see Figure 6-1). The foundation of the dam in the floodplain sectors contained alluvial deposits up to 6 m thick, which consisted mainly of muddy loams and muds with a high content of plant residues.

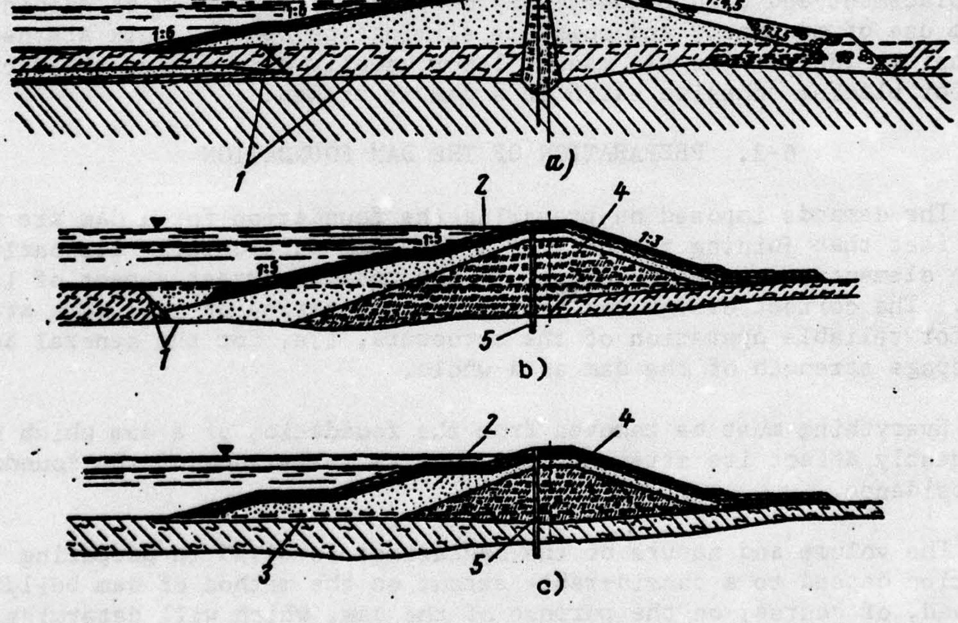

Figure 6-1. Alternative methods used to prepare the foundation of the frozen dam for a permanent reservoir on the Irelyakh River.
a - Method I: 1 - Outline of region cleared; 2 - Body of dam; 3 - Core. b - Method II: 1 - Outline of region cleared; 2, 4 - Body of dam; 5 - Frozen curtain. c - Method III: 1 - Outline of region cleared; 2, 4 - Body of dam; 5 - Frozen curtain.

According to alternative method I, the alluvial deposits in the foundation of the dam would be removed down to the semiconsolidated limestone and marl bedrock.

According to alternative method II, the alluvial deposits would be removed only beneath the upstream wedge of the dam, and they would be preserved in the frozen state beneath the downstream wedge.

According to alternative method III (the one adopted), the alluvial deposits in the foundation of the dam would be retained completely.

The amount of work required (measured in thousands of cubic meters) to prepare the foundation according to the various alternatives considered was as follows:

	Alternative		
	I	II	III
Topsoil removal	2.8	3.9	9.5
Excavation of alluvial mud and clay	133	97	16.4

Analysis of the amount of work involved in excavating material from the foundation and the amount of work involved in filling the body of the dam showed that two-thirds of the volume of fill for the body of the dam, according to the first two alternatives, would consist of replacing one type of material with another. Under these conditions, and considering that the material removed from the foundation was in the permanently frozen state and the replacement materials would also subsequently be frozen, the question arose as to whether it would be possible to retain the alluvial materials in the foundation without replacing them.

At our suggestion a tooth was constructed along the axis of the dam which cut completely through the very muddy alluvial deposits which would be hazardous when thawed and made connection with the semiconsolidated bedrock.

The topsoil was removed with bulldozers and the earth was piled up in ridges in the upper and lower pools beyond the limits of the dam foundation. In addition, ice-saturated muddy earth was removed from beneath the upstream wedge on the left floodplain together with lenses of buried ice and peat. Gravelly material was also removed from the river bed (see Figure 6-2).

The earth was removed from the floodplain areas in the second half of winter, after the seasonally thawed layer had merged with the permanently frozen layer. The earth was broken up using bulldozers and equipment of the BSN type. It was dug with shovels and carried away in dump trucks. Breaking up the earth with explosives was difficult before the seasonally thawed layer had merged completely with the permanently frozen layer. When explosions were detonated within the layer that was not yet frozen, comouflets were formed and gases exploded through the hole, breaking the frozen crust into large pieces and raising them a little at the site of the hole.

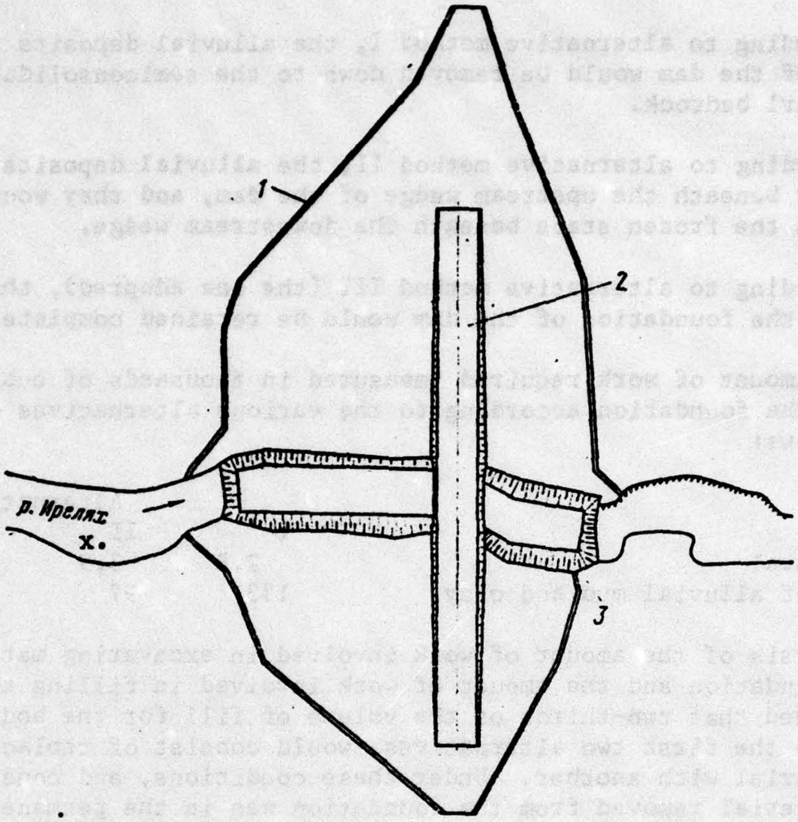

Figure 6-2. Preparation of the foundation of the dam for a permanent reservoir on the Irelyakh River.
1 - Outline of foundation area cleared; 2 - Trench for tooth; 3 - River bed cleared.
Key: x - Irelyakh River.

Preparing the foundation for this dam required the following work:

Clearing trees and brush (ha)	60
Removal of topsoil (m^3)	9,500
Excavation of mud, peat, and buried ice (m^3)	16,400
Removal of gravel from the river bed (m^3)	2,500
Excavation of the tooth trench (m^3)	12,000

In preparing the foundation for the dam on the Oyuur-Yurege River (Figure 6-3) the topsoil and alluvial deposits were cleared away only under the upstream wedge. A trench for the tooth of the core was dug into the alluvial deposits and the tooth was cut into the bedrock a distance of 0.5 m.

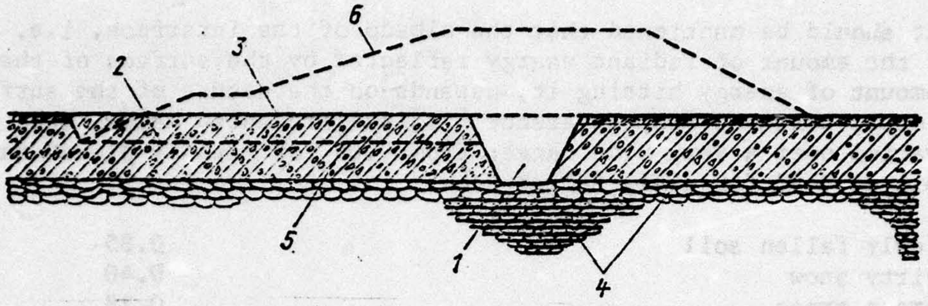

Figure 6-3. Preparation of the foundation for the dam on the Oyuur-Yurege River.
1 - Trench for the tooth; 2 - Outline of the region to be cleared under the upstream wedge according to the plan; 3 - Ground surface; 4 - Limestone; 5 - Icy boulders with clay; 6 - Outline of dam.

The plan called for the placement of 20,000 cubic meters of alluvial deposits, which consisted of gravelly loam with an ice content as high as 40%, but at our suggestion this material was left in the foundation of the dam.

Under particular operating conditions it may be desirable not to remove weak materials from the foundation but rather to thaw such material before construction. Dam foundations may be thawed before construction when the foundation contains materials with a high ice content and their ice content is not uniform under the foundation area of the dam, and when replacement of such materials under existing operating conditions is impossible or not economically feasible. The purpose of this method is to improve the bearing capacity of the foundation material and to prevent uneven subsidence.

There are several methods for the engineering melioration of frozen ground, which have been developed and used successfully in mining as well as in dam construction practice. These include the use of direct solar radiation on the surface of the frozen ground, the use of a natural heat source (the waters of rivers and lakes), and the use of artificial heat sources (such as electricity and steam).

Direct use of solar radiation. Solar radiation in permafrost regions is quite adequate in most cases. The summer there is short, but the air sometimes warms up to 30-36°C. In the summer the sky is usually clear and heat transfer conditions as well as direct utilization of external heat exchange are quite effective. Solar energy warms up the surface layers and causes the ice in the ground to melt.

A significant portion of the solar radiation, of course, is reflected from the surface of the foundation where the radiant energy is converted to heat. A large portion of the energy is consumed in evaporating moisture from the ground. Hence the efficiency of preconstruction thawing of the ground by direct use of external heat exchange is quite low. Never-the less, use of this method is still effective and widespread.

It should be mentioned that the albedo of the interface, i.e. the ratio of the amount of radiant energy reflected by the surface of the layer to the amount of energy hitting it, depends on the nature of the surface. Certain data, borrowed from reference [71] and presented below, show that albedo varies over a very wide range: from 0.05 for freshly plowed dark moist soil to 0.85 for newly fallen snow:

Newly fallen soil	0.85
Dirty snow	0.40
Green grass	0.28
Bare light-colored soil	0.35
Bare dark dry soil	0.15
Bare moist soil	0.10
Bare freshly plowed soil	0.05

Analysis of the data presented above shows that reflective losses of radiant energy may be reduced by appropriately altering the properties of the surface of the layer being treated, which in turn determine its reflectivity. Such alternations may affect the moisture content, structure, and color of the surface layer.

But the process of controlling heat exchange between ground and atmosphere is not limited to these measures. A number of investigations have shown [71] that by installing an appropriate screen it is possible to turn reflected radiation back to the earth again. These studies have shown, for example, that a screen of plastic film freely passes solar radiation and at ordinary temperatures traps up to 90% of the infrared radiation. When this is done, the temperature of the surface layer of the ground beneath the film is raised up to 8°C higher than under natural conditions because of the reduced intensity of radiation and particularly of evaporation.

These methods of controlling heat exchange are widely used at the present time: the ground surface is cultivated, darkened, or even covered with various plastic films, etc.

Thus, in preparing the foundation for dams on the Sytykan River (Figure 6-4) and Novyy Creek, ice-saturated earth was removed only under the upstream wedge, since ice lenses were found there up to 1.5 m thick. In the rest of the area engineering melioration alone was used -- natural thawing of the ice-saturated ground during the summer. For this purpose the peat layer was removed down to the mineral earth in early spring and the foundation was allowed to warm up during the entire summer, with the result that subsidence of the foundation had taken place before the dam was built.

<u>Melioration of foundations with water heat</u>. This is the most widespread method for the engineering melioration of frozen ground. This method includes the thawing of frozen ground with water injection rods (points), infiltration, and sprinkling combined with infiltration. The heat contained in river or lake water is used here to accelerate the thawing of the frozen ground.

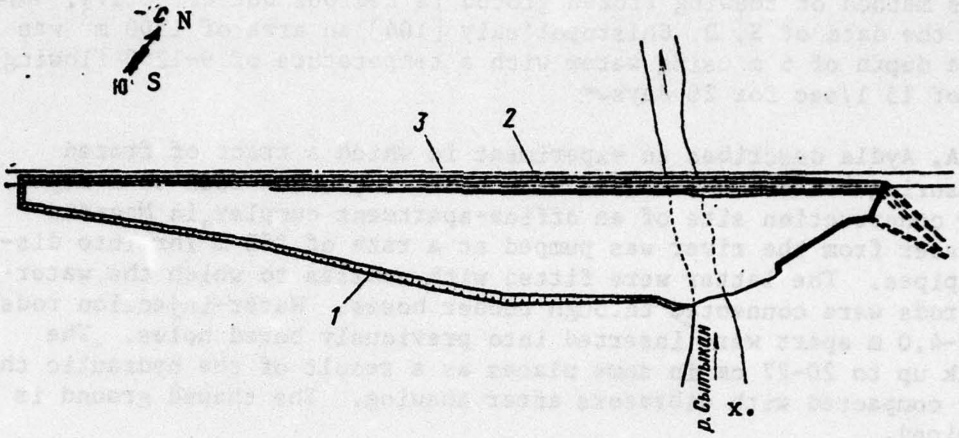

Figure 6-4. Preparation of the foundation for a dam on the Sytykan River.
1 - Outline of stripped area under the upstream wedge;
2 - Axis of the dam; 3 - Trench for the tooth.
Key: x - Sytykan River

The thawing of frozen ground with water injection rods (points) first began to be used in the USA in 1900 to thaw frozen ground for dredging. In this method tubular rods with tapered tips and openings in the tip are inserted into the frozen ground. These rods are similar to those used for lowering the water table (PVU, IFU), but they lack the perforated water-intake component equipped with a filter screen. Water is delivered under pressure through the openings in the tip, and this creates proper conditions for directed convective heat transfer from the thawing layer into the adjacent frozen layer. The rods are inserted into the ground just as in hydraulic drilling [39, 45, 48, 55]. Ascending currents of water erode the earth and transport it to the surface through a space around the tube (see Figure 6-5).

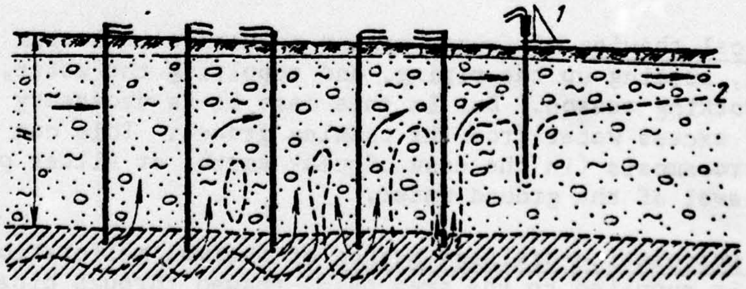

Figure 6-5. Diagram of foundation thawing with water-injection rods.
1 - Rod fitting; 2 - Surface of frozen ground; H - Thickness of thawing layer; the arrows show the direction of water movement.

This method of thawing frozen ground is tedious but effective. According to the data of S. D. Chistopol'skiy [104] an area of 1500 m^2 was thawed to a depth of 6 m using water with a temperature of 9-12°C flowing at a rate of 15 l/sec for 20 days.

T. A. Aydla describes an experiment in which a tract of frozen ground measuring 93x23 m was thawed with water-injection rods to a depth of 7 m at the construction site of an office-apartment complex in Magadan Oblast. Water from the river was pumped at a rate of 250 m^3/hr into distribution pipes. The latter were fitted with nozzles to which the water-injection rods were connected through rubber hoses. Water-injection rods spaced 3.5-4.0 m apart were inserted into previously bored holes. The ground sank up to 20-27 cm in some places as a result of the hydraulic thawing, and it was compacted with vibrators after thawing. The thawed ground is easily drained.

But the duration of such thawing is brief and it is possible only in the warm summer months. Heated water can be used to increase the effectiveness of the water-injection-rod thawing of frozen ground.

According to data given in reference [39], water with an initial temperature of 20°C retains a temperature of 8-12°C as it flows away from the site of the operating rods. The relative heat output of the ascending percolating flow in the growing cylindrical thawed area around a rod is equal to 0.4-0.6. When this water is recycled and heated again to 20°C, only half as much heat is required. When the water temperature is raised, the spacing between the rods can be increased and there is a corresponding reduction in the drilling work required.

A relationship has been established between spacing of the holes and water temperature based on experience in thawing 500,000 cubic meters of ground to a depth of 9.5 m:

Water temperature (°C)	3	4	6	12
Spacing of rods (m)	3.5	4	6	6

Electrical thawing of frozen ground permits elimination of the investment in boring, setting up the piping, and supplying the water, and it prolongs the working season. But in this case it is necessary to provide for removal of excess water from the thawing ground. This can be accomplished by using electroosmosis (in the case of clay earth) or filter point equipment to lower the level of the ground water.

In the percolation-drainage method of thermal melioration of frozen ground, water is supplied to the area being thawed through pipes or ditches. The water percolates through the thawing ground and flows into a drainage ditch. The method can be effective in well-drained materials. To accelerate thawing, fine-grained material at the surface and mantle loams should be removed. Several seasons may be required to achieve the desired effect.

The frozen dam on the Sytykan River may be cited as an example. The foundation of this dam on the left floodplain contained, under a thick layer of moss and peat, as has already been mentioned, ice-saturated material and individual lenses of fossil ice which were quite thick. In order to thaw the ground in a single season, the plan provided for engineering melioration of the ground in addition to natural thermal melioration, which was discussed above. After removal of the moss-peat layer with bulldozers, the intention was to run several percolation-drainage ditches spaced 30 m apart and positioned according to the topography to collect and carry away the water formed as the frozen ground thawed. The average depth of the ditches was 2 m. The total discharge of water with simultaneous replenishment of 50% of the drain output was calculated to be 1 m^3/hr per meter of drain length or 220 m^3/ hr.

The sprinkling-percolation method was first used in the USA at the end of the nineteenth century. The technology of this method is simple. Water is applied to the surface of a tract being thawed from a sprinkling apparatus (see Figure 6-7). Since this work is done in the warm season of the year when the air temperature is above the water temperature, the water receives additional warmth in the air, and as it percolates, releases the heat to the ground. The vertical percolation flow becomes horizontal and moves toward a drainage ditch or natural depression. This method was used very successfully for engineering construction purposes to thaw ground heavily saturated with ice in preparing the foundation for the Hess Creek dam in Alaska.

Ground containing tree roots and other plant residues as well as ice-saturated alluvial deposits up to 12 m thick had to be removed from the foundation for the Hess Creek dam in order for the structure to rest on the permanently frozen bedrock. But in this case materials with moisture contents up to 31% were allowed to remain in the foundation of the dam.

According to the original plan the stripping work was to be done with hydraulic excavators, and for this purpose 27 hydraulic excavators were installed in the dam excavation. But this attempt proved unsuccessful because, since they discharged a large quantity of water, they did not give satisfactory results, especially in areas where the ice content of the ground was high or the ground contained thick lenses of ice with considerable horizontal dimensions.

Subsequently stripping was successfully accomplished using sprinklers. Up to 400 sprinklers were installed for this purpose over the dam excavation area, and water was supplied to them at a temperature of 11°C. The continuous irrigation of the area to be stripped caused rapid thawing of the frozen ground. The flowing water containing particles of soil eroded and furrowed the surface of the area. The slurry flowed into a pond. In this fashion the ground to be stripped away was thawed at a rate 10-15 cm/day.

Well-planned organization of the work made it possible to remove by erosion 306,000 cubic meters of earth from the foundation for the dam.

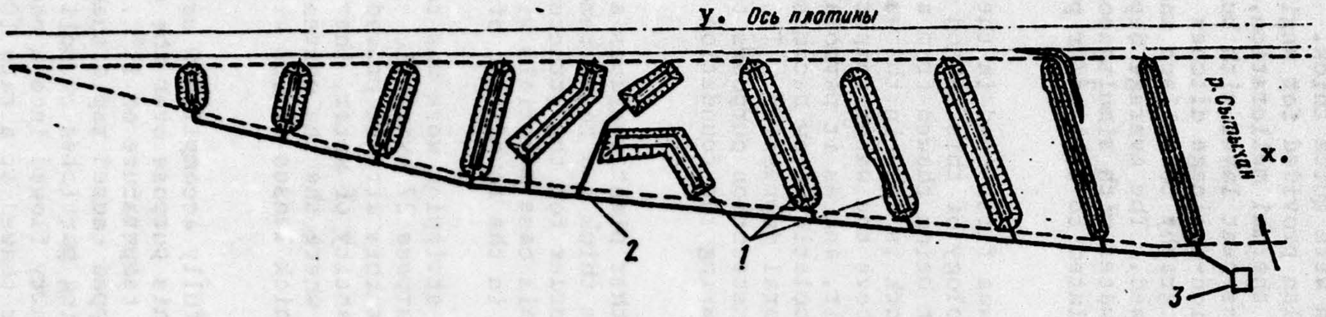

Figure 6-6. Percolation-drainage method of thawing a foundation.
1 - Drainage ditches; 2 - Collector for water supply; 3 - Pumping station.
Key: x - Sytykan River; y - Dam axis.

N. A. Tsytovich emphasizes that with preconstruction thawing a large percentage of the total settlement of the foundation occurs before the structure is built (thermal settlement), and only settlement due to compaction of the ground remains during the operating period. The later is relatively small and distributed quite uniformly over the area (depending, of course, on the foundation materials and their thickness). Tsytovich recommends that hydraulic thawing be used only for gravelly and other large-grained materials, which lose little of their bearing capacity when they become supersaturated.

In our opinion hydraulic thawing with water-injection rods can be used for any materials, including cohesive earth, but supersaturation of the ground is unavoidable. When there is sufficient reason to use this method, drainage of the ground is possible, depending on how operations are organized or other conditions. Artificial drying is required also when clay soil is thawed electrically, since in this case supersaturation is also unavoidable [46].

We have dwelt in detail on the methods of artificial preconstruction thawing of permanently frozen loose materials because we feel that these methods can be used for dam construction in the north. We do not consider the strict restrictions contained in VSN-08-65 to be justified, as is indicated by both domestic (Sytykan River, Novyy Creek) and foreign experience (Hess Creek) in constructing dams where foundation materials were thawed prior to construction.

Frequently when the foundation is clear and tooth trenches are dug, additional information becomes available as to the engineering geology of the foundation which calls for additional geological investigations and also will lead to changes in the type and amount of work to be done in preparing the foundation.

In some cases it may become necessary to install a grout curtain or to grout a portion or all of the dam foundation area. In the case of permanently frozen rock this work is done after first thawing with steam or water injected into previously bored holes. The ice filling cracks is melted and the cracks either empty or filled with water, which permits their subsequent grouting.

Requirements as to dam foundation materials depend on the method used to build the dam.

Rock may be used as a foundation without limitation provided the rock is watertight. When the rock is highly fissured, grouted or other cut-off seals are installed to reduce seepage losses.

Dams are most difficult to build on foundations subject to thermal settling and foundations susceptible to thermokarst phenomena. In these cases engineering melioration techniques may be used on the foundation, the hazardous materials may be removed entirely from the foundation, or dams of the frozen type may be built on such foundations.

Highly permeable foundation materials are no obstacle to building unfrozen dams on them if antiseepage structures are included.

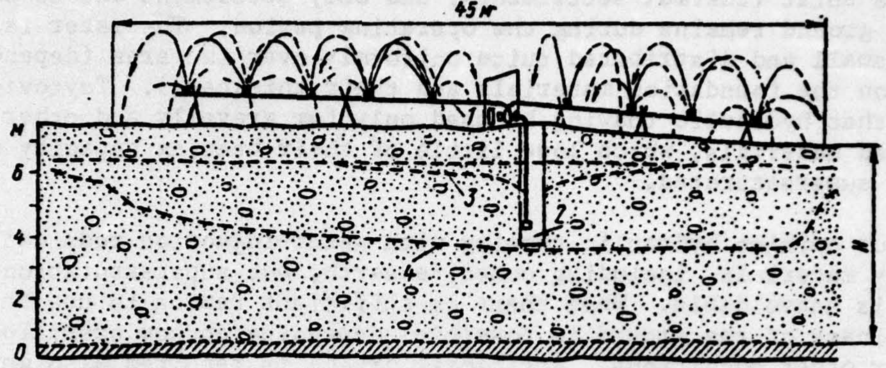

Figure 6-7. The sprinkling-percolation method of thawing foundations.

1 - Sprinkling apparatus; 2 - Drainage wall; 3 - Water table;
4 - Surface of frozen ground.

6-3. BUILDING EARTH DAMS IN THE SUMMER

a) "Dry" Placement of Cohesive Soils

The principal method of building dams from cohesive soils in the Far North, if conditions permit, is still the dry placement of the soil with appropriate compaction. One must only keep in mind that soils in the Far North usually are highly supersaturated, so that placement is quite difficult.

We know that the relationship between density, moisture content, and the amount of effort invested in compaction is described graphically by the so-called Proctor curves. The peak of the curves reflects the maximum soil density for a given compaction effort and is achieved only with a particular soil moisture content, which is known as the optimum moisture content. Analysis of the curves indicates that soil compaction is not efficient when its moisture content is below or above the optimum.

In order to assure proper packing it is necessary to resort to supplementary measures designed to reduce the mositure content of the soil, i.e. to dry it. This is usually done by breaking up and turning over the waterlogged soil, both in the borrow pit and at the fill site.

The thickness of the layers to be compacted usually ranges from 10-40 cm, but the most common thickness is 20-30 cm. Rolling tests are used to determine the number of passes of compaction equipment required and the optimum layer thickness.

The purpose of compaction in building water-retaining structures is to create a soil structure which will make the dam watertight and sufficiently flexible and will assure that possible settlement during operation will not exceed allowable values and will not result in the formation of cracks and fissures.

The degree of compaction is determined by the density of the soil used and is based on the technical specifications for the structure.

Construction practice shows that a high degree of fill compaction is not always desirable, since, in spite of the high overall stability of the structure, minimum settlement, and high resistance to seepage, achieving it requires large investments of time and machinery, which are not always economically justified.

At the same time, when soil is highly compacted, the structure becomes more rigid and cannot flexibly adapt to foundation subsidence (and the seal cannot adapt to subsidence of the rock fill below the seal), with the result that failure may occur at joints with other structural elements.

As long as the basic technical requirements for earth dams are met (strength, elimination of the possibility of slope failure, impermeability to water), the degree of compaction of the earth in the structure should be planned in such a way that the cost of this operation is held to a minimum. The economically desirable degree of compaction of the earth in the body of a dam is determined by engineering and economic computations.

The techniques and methods of compacting soil depend on many factors, so that it can be correctly organized only if one takes fully into account the characteristics of the materials being compacted, climatic conditions, the requirements placed on the structure, the time in which it is built, etc.

The soil is usually rolled with special rollers of various types, which are distinguished on the basis of the principle of their action on the soil being compacted: dynamic, oscillating, or static. The fundamental characteristic of rollers is their weight and specific pressure on the ground. Loamy soils usually are compacted with static action rollers, mainly with smooth drums, sheepsfoot rollers, or pneumatic tire rollers.

The earth is compacted under constrained conditions, and where access for compaction machinery is impeded, the compaction is done with various kinds of impact or other compaction equipment for use in tight spaces.

Self-propelled rollers are most effective because of their maneuverability; towed rollers are inconvenient for working on fill. Very good compaction results are obtained with loaded MAZ-525 dump trucks. Their advantages include their availability, their good maneuverability, and vibrational effect.

Rough data on the thickness of layers of loose fill used for different types of compaction equipment are given in Table 6-1.

Table 6-1

Thickness of Layer Compacted as a Function of Compaction Equipment.

Compaction equipment	Weight, tons	Thickness of layer compacted, cm	Number of passes over one location	
			Cohesive soil	Noncohesive material
Pneumatic-tired rollers	10	16-24	4-10	2-8
	25	28-40	4-10	2-8
	50	35-50	4-10	2-8
Sheepsfoot rollers	5	19-27	4-18	6-12
	30	50	4-10	2-8
Smooth rollers	5	12-18	6-12	6-10
	10	16-24	4-10	2-8
S-80 tractor		10-16	6-12	4-10

A general deficiency of existing compaction equipment is the fact that it cannot be used to compact sloping areas, and for this reason this portion of the fill is least compacted. Where the slopes are gentle, they can be partially compacted with tractors or bulldozers. Sometimes fill is placed in embankments with an extra width allowance of 0.4-0.5 m, the excess soil subsequently being cleared away and used in the embankment. This technique, of course, cannot be used when structures are built at negative temperatures and the slopes are subject to freezing.

It is very important in building a structure to conduct compaction tests in order to determine optimum soil compaction conditions: to determine the thickness of a layer to be compacted, the number of passes by the compaction equipment along a single track, the optimum moisture content, etc.

According to studies by V. I. Biruli and N. Ya. Kharkhuta, the strongest soil structure is obtained as a moisture content close to the lower plastic limit, i.e. about 0.55-0.60 of the moisture content at the liquid limit and close to the optimum moisture content as determined by the standard compaction method.

The moisture content of the soil never remains constant during the period when a structure is being built. It is affected by climatic characteristics, the varying degrees of the wetness of the soil in the spring and fall, etc. As a rule, drying is required in order to obtain the optimum soil moisture content under the conditions of the Far North. Less often moisture must be added, both directly at the work site and as naturally bedded in the borrow pit: a plot prepared for wetting is surrounded by a ridge and water is poured over it. For best penetration of water to the full depth of borrow material desired, holes are also bored or pits are dug [18]. Irrigation of the soil in the borrow pit provides uniform wetting of all the soil and permits placement of the soil at the work site without delay.

For operating convenience the compaction machinery is usually required to move parallel to the axis of the dam. When the open face is wide, as is usually the case at low levels in dams, the soil should be packed in place by dividing the full width of the dam into strips (2 or 3). When this is done the layers should be overlapped where they meet and carefully compacted.

b) Building Dams by Hydraulic Fill

In the Far North building hydraulic fill dams is complicated by a number of special conditions resulting from the presence of permafrost, the severe winter, and the short period with favorable temperatures. But it is possible to build dams by the hydraulic fill method. An obvious example of the use of this method is the delivery of slurry through pipes and its placement in a walled-up tailing pond even at very low outside air temperatures.

Under some conditions building dams by hydraulic fill is the only method possible. A very interesting example in this respect from both the planning and implementation points of view, is the building of the Hess Creek dam in Alaska.

The Hess Creek dam is 508.5 m long and 25.2 m high, and impounds a reservoir of 9.2 million cubic meters. The dam is 103.2 m wide at the base and 4.8 m wide at the crest. The slope gradients range from 1:2.5 at the base to 1:1.3 at the crest.

The Hess Creek Dam was built in a place difficult to access, where outside air temperatures dropped as low as -68°C (during the construction period the temperature ranged from -21° to -32°C). During the summer the ground thaws to a depth of no more than 0.6 m.

Since the cost of transporting imported structural materials would be very high, it was decided to build the dam by hydraulic filling, a technique which required minimum importation of materials and equipment.

The local materials have a high content of large rock fragments. These materials would be suitable for packing dry into the body of the dam only after preliminary separation. For this reason the dry method of construction was rejected. Also, the construction area has a short but rainy summer.

The biggest problem with the alluvial fill method was obtaining a sufficient quantity of fine material for the core of the dam, which accounted for 16% of the total dam volume. The necessary quantity of fine material was obtained from another borrow pit by additional hydraulic sorting of the earth. A special washing system was used for this purpose. An entirely satisfactory hydraulic fill core, constituting 1/6 to 1/10 of the width of the dam, was built in this fashion.

Construction on the Hess Creek dam was begun in 1940 to provide a water supply for mining operations, but during the war, when the hydraulic fill had reached a height of 17.2 m, work was stopped. A temporary spillway structure was built at this intermediate level. Construction of the upper part of the dam to the full design height (25.2 m) was completed in 1946, and the earth was packed dry [106].

A steel sheet-pile bulkhead was provided to prevent seepage through the thawed material in the foundation, and the foundation which had thawed during construction operations was refrozen.

The sheet-pile bulkhead was installed by thawing the foundation with steam points. The foundation was frozen after removal of the overburden and exposure of the bedrock. Before starting the hydraulic fill, trenches were dug every 3 m in the foundation along the axis of the dam. Pipes 50 mm in diameter were laid in these trenches to freeze the foundation. The freezing would create a good connection between the underlying permafrost and the foundation of the hydraulic-fill body of the dam. In order to freeze the foundation where the dam joined the valley slopes, a special freezing system was built of arrays of piping (one on each slope) and a freezing solution of brine was pumped through them. Each array consisted of a system of 25 pipes 37 mm in diameter and 91 m long. Pipes were installed 0.3-0.8 m above the surface of the ground so that the cold arctic air could reach them. These pipes terminated at both ends in risers, from which the freezing brine was pumped back into the piping system within the valley wall connections. The temperature of the brine dropped to -40°C, so that a considerable portion of the valley slopes froze quite satisfactorily and remained in this state until hydraulic fill of the body of the dam began.

6-4. BUILDING EARTH DAMS OF COHESIVE SOIL IN WINTER

a) Dry Packing of Cohesive Soil

As has already been stated, the principal method for packing cohesive soil in the Far North is the dry filling of the soil in layers 0.3-0.4 m thick where the top of the fill is divided into working areas [4]. In some cases (depending on the width of the structure being built) the work area may be divided into two or more longitudinal strip zones. For operating convenience such strips may in turn be divided into individual working areas of equal size. This provides the basis for a clear-cut order of operations. While soil is being placed on one of the work plots, previously placed soil is being leveled on a second, and the soil is being compacted on a third. When the soil is excessively wet, a fourth stage may be involved, a soil drying stage. But since it is impossible to dry excessively wet soil in the winter, it is necessary to resort to a somewhat different method of placement, such as that used in building the Khantayka Hydroelectric Power Station.

The soil dumped on a work area is spread evenly over the entire area. Sometimes, in order to use the transportation equipment to compact the soil, the filling is done by the so-called "pioneer method." With this method there is no clear separation between the dumping and spreading stages. The deposited soil is spread immediately to make way for the approach of the next vehicle.

When high-grade embankments are being built at negative air temperatures, evaporation from the unfrozen soil interferes with the work of men and machinery, This, of course, should be taken into account from the viewpoint of providing safe working conditions.

When placing soil by the so-called "backup" method, boulders, large lumps of frozen ground, and other inclusions can be removed quite successfully after the soil is spread and before it is compacted. In packing soil into the seal of the dam for the Vilyuy Hydroelectric Power Station, special rakes were attached by hinges to the blade of a bulldozer to remove boulders (see Figure 6-8)

Figure 6-8. Removing boulders and frozen lumps.

The thickness of the soil layer installed depends on the method of compaction used and the type of compaction equipment, the degree of compaction required by the technical specifications for the structure. If uniform compaction of the soil is to be achieved, the layer thickness must be uniform.

When building structures of cohesive soil during periods with negative temperatures, it is extremely important to keep the soil in the thawed state until compaction has been completed. In order to accomplish this, the soil in a working area is covered with film as it is delivered until the required amount is in place and has been spread (see Figure 6-9). This very effectively protects it from cooling off too early. Observations have shown that soil covered in this fashion cooled no more than 1.2-1.5°C under the film during the entire period of delivery.

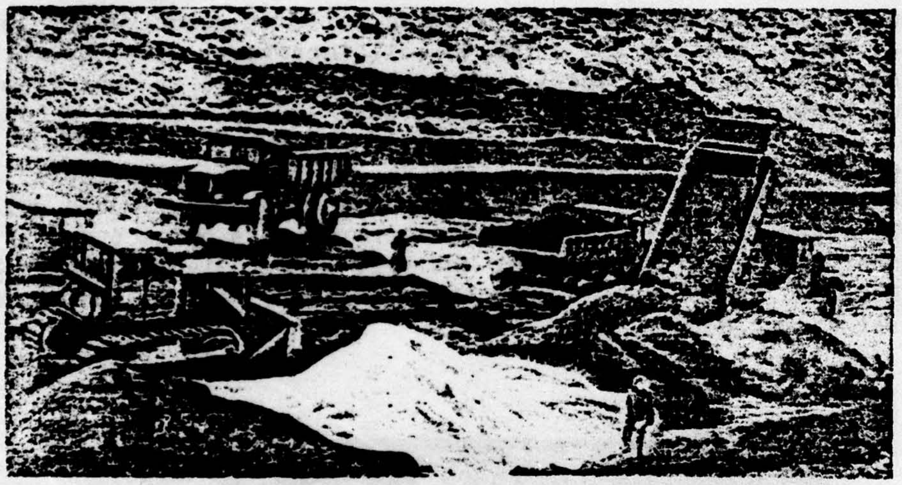

Figure 6-9. Covering soil with film.

Soil can be packed into a blanket seal both after the supporting prism of the dam has been raised to its full design height and simultaneously with its construction. But in the latter case contaminated areas of the seal should be carefully cleared of materials from the main body of the dam and from the filters below the seal.

The inverted filter layers below the blanket seal usually are not very thick. In order to protect the seal from piping, they should be no thicker than 0.3-0.5 m, but depending on construction conditions they usually are made wider, with the idea that transport equipment can pass over them. This reduces construction costs. Wider use must be made of cranes equipped with special buckets, portable conveyors, and the like in placing filters.

Packing density specifications for the soil in blanket seals should be designed to provide flexibility in the seal itself so that it can adapt to settlement of the main part of the dam body without loss of integrity and the formation of cracks.

A somewhat different technique was used in the winter placement of supersaturated soil in building the Khantayka Hydroelectric Power Station. The average moisture content of the Khantayka soils was 15.3%, with an optimum content of 9-10%. Soil was packed into the core in a space between transition zones. The transition zones were built up slightly in advance. Access to the work space was difficult, so that the trucks dumped their loads from the edge of the transition zone. The soil was spread over the working area in a layer 0.5-0.7 m thick and compacted with bog tractors having wide tires. The density of the soil packed in this fashion was 5% lower than that of soil installed with optimum moisture content [44].

b) Filling with Cohesive Soils in Water

Filling with cohesive soils in water is a technique widely used under ordinary conditions [32]. Some practical experience has been gained in this country in building high-grade embankments by filling in water with cohesive soils during winter as well. The technology of winter filling work, of course, is more complicated. There are serious problems of maintaining the pool, preventing the formation of ice along the edges and on the bottom, and acquiring and storing sufficient supplies of unfrozen soil. At the same time, filling in water may be the only way to achieve high-quality packing of the soil when building particular elements of a structure.

At the same time it must be said that there are unfortunately still no generalized analyses and recommendations for this promising method. Careful study should be given to problems of specifications for soils to be used for building structures by filling in water during the winter, questions concerning the permissible content of frozen ground in the total mass of soil used for filling in water, the effect of the frozen ground content on the quality of an embankment as well as the effect of their thawing and factors influencing this process, the temperature regime of the water in the pools, methods of controlling the formation of bottom ice in the filling work-area and the formation of ice along the edge of the work area, problems of building dikes around the work areas, etc. In our opinion dikes can be successfully built around work areas using, for instance, dry sandy soils and fill materials to prevent water from leaking into the lateral prisms.

The thermal conditions in embankments when they are built by constructing ponds in several stages may vary; i.e. in some areas there may be frozen zones in addition to the thawed zones. However, as experience with the building of the blanket seal for the Vilyuy dam and the core of the Khantayka dam has shown, even when it is built dry the seal cannot be kept unfrozen over its entire height. The presence of unfrozen and frozen zones did not adversely affect the quality of the seal and core as antiseepage elements of the dams. But it is important here, of course, to assure a high-quality joint between the layers. The frozen surface of a previous layer will thaw when a pool is formed for the next layer. This will also facilitate warming of the water in the pool, which may be unavoidable with air temperatures of -10 to -15°C.

This problem was met for the first time to our knowledge in the Iriklinskaya Hydroelectric Power Station. In building the apron and blanket seal of the rock fill dam, loam containing up to 40% frozen material was used as winter fill in water. As we know, after thawing and consolidation the embankments built by this method were of very high quality.

Placement of cohesive soils in water was also used in building the Irkutsk Hydroelectric Power Station [61]. This work was preceded by studies which showed that loam placed under water after two months acquires a density of 1.63 g/cm^3 with a moisture content of 23% and a percolation coefficient of A·10^{-7} cm/sec. The structure of the loam was uniform regardless of whether it was packed in the thawed state or included frozen lumps. Filling of loam into the core of the left-bank dam was undertaken at air temperatures from -3° to -24°C in layers 2 m thick.

Studies of the engineering geology of the frozen and thawed loam showed that the soil did not lose compaction. From this the important conclusion was drawn that it was not necessary to wait for complete thawing of the soil in order to fill the succeeding layers.

In building the Irkutsk Hydroelectric Power Station at air temperatures as low as -25°C, loam was quite often placed under water which was electrically heated to +15°C. Pockets in the excavation for the power station building were filled in this fashion with about 50,000 m^3 of loam. The earth was placed in layers up to 9 m thick into a closed space. The water was forced out into the lateral prisms of the dam and subsequently pumped out. The content of frozen lumps in the soil was no more than 5%. The placed earth had adequate density and was uniform.

Relatively warm ground water (up to +5°C) from the water evacuation system of the excavation was also used on the project to form the pools.

Much experience in the winter placement of soil under water at low temperatures (from -20° to -50°C) was obtained in building the Vilyuy Hydroelectric Power Station when the cofferdams were built surrounding the foundation site for the dam in the river bed area [4], and later in connection with the construction of the Serebryanskaya and Khantayka Hydroelectric Power Stations [44].

The design of the cofferdam for the Vilyuy Hydroelectric Power Station was of the mixed type at our suggestion -- the supporting prism was made of rock fill, the blanket seal of gravelly loam, and the transition zone of mixed sand and gravel (see Figure 6-10). The width of the blanket seal at the top of the structure was set at 8 m, and the gradient of the upstream slope was 1:3. In order to reduce seepage along the foundation in the channel area, since it was not cleared of boulders and alluvium, an apron was built of loam 20 m long and about 1 m thick.

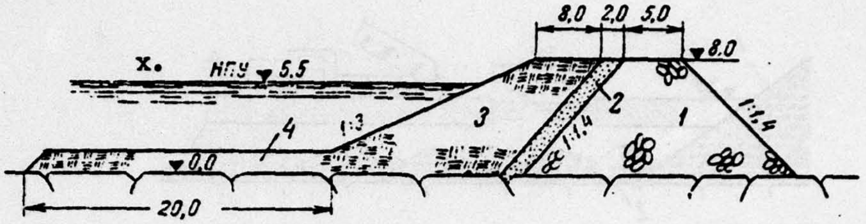

Figure 6-10. Cofferdam along the foundation site for the Vilyuy Hydroelectric Power Station.
1 - Supporting prism; 2 - Filter; 3 - Blanket seal; 4 - Apron;
Key: x - NHL.

The loam fill was placed under water at air temperatures from -20° to -50°C. The ground temperature where the soil was dug ranged from +4° to +7°C, and at the placement site from +1° to +4°C. The blanket seal was built with an extra height allowance from 0.1 to 0.5 m above the water level in a water area that was free of ice. When fill work was interrupted, the brow of the slope at the water line was sprinkled with salt and a perforated hose was submerged in the water to deliver compressed air, which prevented the formation of layers of ice.

An opening was made in the ice with explosives after digging a trench 0.6-0.7 m around its perimeter to prevent cracks in the ice from spreading beyond the desired margin of the opening. Then holes were cut all the way through the ice in a 4.0x4.0 m pattern. Charges of 4-5 kg were submerged in the water and detonated. The underwater explosion produced uniform pieces of ice with a minimum amount of fine material, which is very difficult to remove from an open area. Creation of an open area measuring 20x10 m with an ice thickness of 1.2-1.3 m by this method required two or three shifts.

In order to keep small pieces of ice out of the fill, a barrier of logs was strung in front of the filling area to force the sludge away. A turbojet apparatus was also used at the construction site for the same purpose. The strong blast of exhaust gases cleared the open water of sludge and small pieces of ice.

In placing the loam for the apron of the cofferdam an opening in the ice 2 m wide and 40-50 m long was cut manually along the line where fill work was to begin. Then the loam was unloaded from the ice directly into the water in such a way that it covered the upstream slope of the previously built blanket seal of the cofferdam. When the first line of fill was complete the opening was widened by cutting out a strip of ice 2 m wide and shifting it toward the fill that had already been placed in the apron (see Figure 6-11). Then more loam was unloaded into the water, etc. The individual mounds were not spread out under the water. Each vehicle was unloaded at strictly defined locations.

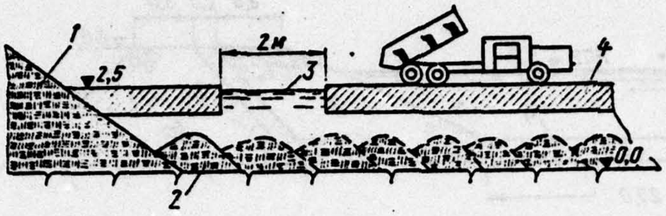

Figure 6-11. Winter fill work for an apron under water.
1 - Blanket seal of cofferdam; 2 - Loam fill for the apron;
3 - Open water; 4 - Ice.

Test pits showed good-quality packing of the loam in the cofferdam seal. The bulk density of the soil skeleton ranged from 1.85 to 2.05 g/cm^3. Seepage through the cofferdam was insignificant, so that work could be done in preparing the dam foundation, excavating rock, and placing cement in the grouting tunnel and its wings with practically no need to pump water. The maximum head pressure on the cofferdam was 9.5 m.

Thus the possibility of wide use of soil fill under water in the winter under conditions of the Far North is very attractive. But basic difficulties with this include inadequate information on questions of the consolidation of soil placed as underwater fill, particularly when partial freezing occurs, and also the difficulty in maintaining normal temperature conditions in the pool to prevent the formation of ground ice, etc. Serious difficulties are encountered in solving the problem of controlling ice formation on the surface of water openings and removing substantial amounts of ice and sludge from the working area. And ice-removal techniques must be used which will entirely prevent the incorporation of ice in the body of the seal.

In building the dam for the Serebryanskaya Hydroelectric Power Station in 1968, the water in the pool was heated to +8°C using special tanks. In order to reduce heat losses, the surface of the pond was covered with sheets of polystyrene foam [95].

Another problem that has received little attention is that of the consolidation of soil placed on a frozen foundation. We know that the consolidation of soil placed under water depends on the rate of percolation of water from the soil to the foundation and the lateral prisms, which usually are placed dry. Under construction conditions in the Far North the foundation of the first section of a blanket seal or core consists of very cold permafrost, so that consolidation of the earth is possible only via water drainage through the lateral supporting prisms. The same unfavorable conditions for consolidation occur in placing the succeeding layers, since the upper portion of each layer of soil freezes as it is being placed and becomes relatively impermeable to water.

The problem of drainage through the lateral prisms of an embankment also becomes complicated because in this case water percolates through a cold porous medium, the percolation capacity of which depends on the compaction of the earth in the lateral elements of the embankment and the extent of cooling of the earth in them. Clogging of the pores by the ice that forms is inevitable, and this will prevent further percolation, and thus further consolidation of the fill work.

In 1964 a method was developed at the Vilyuy Hydroelectric Power Station construction project for building a loam blanket seal with earth fill under water during the winter, but this technology was never put to use.

According to this method, water from the river at a temperature of 1-1.5°C was to be pumped into the pool. In order to create a normal thermal regime in the pool, it was planned to heat the water in it to +20°C, and construction of a special boiler was planned for this purpose (see Figure 6-12).

In order to prevent the formation of ground ice at the time the pool was filled, it was planned to heat the foundation of the work area with a turbojet apparatus and also to treat it with a concentrated solution of calcium chloride.

In order to reduce heat loss through ice formation on the surface of the pond, it was planned to build an insulating layer of foam ice up to 20 cm thick.

Before flooding the next work area the surface of the previously placed loam was to be carefully cleansed of ice, snow, mud, and uncompacted earth, treated with a solution of calcium chloride, and heated with turbojet equipment. The pools for the second and subsequent layers may be built by mounding up any sort of earth and covering it with film. Thus, earth filling under water consists of a series of successive operations, including the "dry" building of transverse and longitudinal dikes, pumping water in, creating and maintaining an area of open water, placement of the soil in the water, etc. All these operations must be carried out in a strict and straightforward manner. For instance, unplanned interruption of the fill work before the work area is completely flooded would lead to saturation and freezing of the earth, which would cause heaving and deformation of the embankment. Nor is it permissible to stop removing ice or cease circulating warm water in the pool, and if the power supply at the construction site is not reliable, such a possibility is not out of the question, so that use of this technology should be rejected.

We feel that in the future, when reliable methods are available for controlling ice formation on the bottom and for maintaining the necessary thermal regime in the pool, and when operations are properly organized, earth fill under water may be used successfully to build watertight embankments under winter conditions.

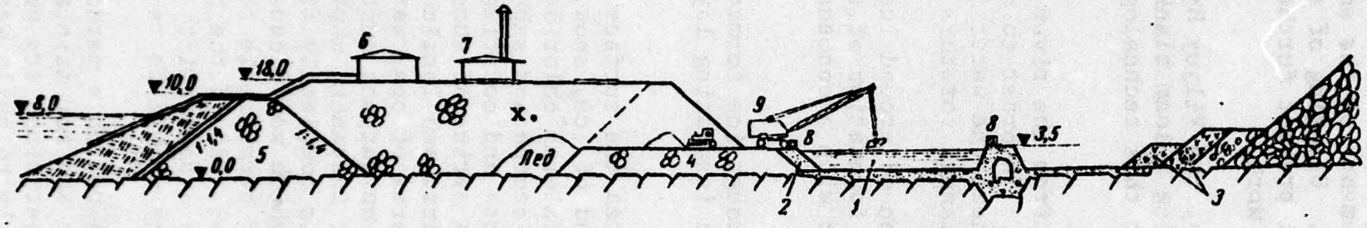

Figure 6-12. Arrangement of a pool for placing loam fill under water.
1 - Pool; 2 - Filter above the blanket seal; 3 - Filter below the blanket seal; 4 - Toe; 5 - Upstream cofferdam for the dam site; 6 - Pumping station; 7 - Boiler; 8 - Pipe; 9 - Crane.
Key: x - Ice.

Earth with any content of lumps of any size is suitable for building structures using the under water earth fill method: from uniform powdery material to large blocks. When frozen ground is used, the fill should contain 40-60% thawed material.

Table 6-2 given the granulometric composition of earth placed by the underwater fill method in building the Irkutsk, Iriklinskaya, and Vilyuy Hydroelectric Power Stations.

Table 6-2

Granulometric Composition of Earth
Placed by the Underwater Fill Method

Hydroelectric station, structure	Content, %; particle size, mm							
	>1,0	1—0,5	0,5—0,25	0,25—0,10	0,10—0,05	0,05—0,01	0,01—0,005	<0,006
Irkutsk HPS, core of dam	0,1	0,8	0,4	3,2	17,6	48,3	11,3	18,3
Iriklinskaya HPS, apron and blanket seal of dam	0,3	0,1	0,7	3,0	13,3	37,2	15,5	29,9
Vilyuy HPS, blanket seal and apron of cofferdam	57,14	5,29	5,49	6,17	9,0	9,61	2,81	4,49

6-3. SPECIFICATIONS FOR COHESIVE SOILS FOR WINTER PLACEMENT

The suitability of material for earth dams is determined on the basis of its physical and mechanical properties: its impermeability to water, strength, placement properties, etc. At the same time we keep in mind, of course, the part of the dam profile for which this material is intended: the supporting prism, which withstands the hydrostatic pressure and affords strength to the entire structure; the antiseepage components, or areas of the cross-sectional profile in which drainage is provided. Curves showing the granulometric composition of earth used in the blanket seal and filters of the Vilyuy dam are presented in Figure 6-13.

Two types of material are used in the supporting zone of the body of a dam. Sometimes, however, in the construction of unfrozen dams there is a limitation on the content of ice-saturated and muddy materials, which lose their strength when wet, the content of material with water-soluble admixtures, and the content of topsoil.

Muddy materials are not suitable for the main body of unfrozen dams because in the saturated state they have a very small angle of internal friction and very little cohesion. In order to have the necessary strength the supporting portion of the body of such a dam would have to be very flat. For instance, the South Saskatchewan Dam in Canada, which is built of silty and muddy earth, has slopes with a gradient of 1:20. It is quite possible to use muddy materials in the body of frozen dams.

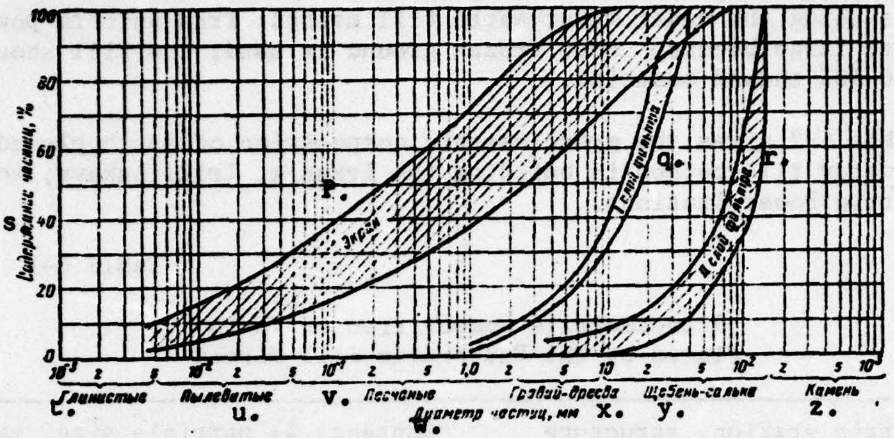

Figure 6-13. Curves showing the granulometric composition of earth used in the Vilyuy dam.
Key: p - Seal; q - Filter layer I; r - Filter layer II; s - Particle content, %; t - Clay; u - Silt; v - Sand; w - Particle diameter, mm; x - Small gravel; y - Cobbles and large gravel; z - Stones.

Cobbles, small gravel, and sandy-gravelly materials are widely used for supporting prisms. These materials can be used in the body of a dam whatever the negative temperatures of the outside air.

Loams are a good material for the basic profile of the body of a dam and for the antiseepage components. Loams are relatively impermeable to water and have adequate strength. The physical and mechanical properties of clay are also suitable for the basic profile of the body of a dam. Their drawback is that they heave when frozen. Therefore it is recommended that dams made of such materials be provided with a protective covering of material not subject to heaving over the entire exterior.

Earth for antiseepage structures should have a low permeability to water, a low percolation coefficient. Loams, clays, muds, peat, natural earth mixtures with definite ratios of component particles, as well as artificial earth mixtures are all suitable for antiseepage elements. In selecting material for antiseepage elements one should take into account their strength in addition to their permeability to water.

Natural clay materials usually satisfy the requirements for their use in antiseepage structures of dams built of local materials. Limitations which may exist with respect to these materials are concerned not with their physical and mechanical properties but with technical construction problems. For example, excessively wet earth cannot be compacted.

Earth for seals and core should meet requirements for watertightness and resistance to piping. If natural sand-gravel materials are not resistant to piping, then for use in filters they should be mixed with other materials or crushed stone should be used of appropriate particle-size composition to assure seepage resistance of the seal or core.

In building dams on permafrost, when one is not sure of the nature of deformations in the foundation and the body of the dam itself, one should make use of a transition zone containing at least two layers, which will assure resistance to piping in the seal or core even when cracks running all the way through them develop.

Before construction operations begin, borrow pits should be carefully explored and studied, and the physical and mechanical properties of the earth should be investigated. The results of these investigations should tell whether the borrow materials can be used in the various structural elements of the dam, and should also provide a basis for defining specifications for the earth to be used and for the quality of placement in the structures, i.e. specifications with respect to density. The earth is tested under laboratory conditions to determine its permeability to water, compressibility, strength, etc.

If a gravelly loam is intended for use in the body of a dam, its engineering geology characteristics should be focussed mainly on the characteristics of the fine earth (i.e. to the content of particles smaller than 2 mm), and therefore all tests in this case are conducted on the fine earth.

The percolation coefficient of most cohesive soils usually falls in the range of $A \cdot 10^{-4}$ cm/sec with pressures within the skeleton of the soil from 0-1.5 kg/cm^2. As special tests have shown (tests conducted by the Kalinin Polytechnic Institute of Leningrad on gravelly loams of various compositions which were the weathering product of diabases), some variations in the granulometric composition of the earth do not have any essential influence on its permeability. A percolation coefficient $A \cdot 10^{-4}$ cm/sec is adequate to assure the necessary impermeability to water and rapid consolidation of the soil after it is packed into the body of a structure.[1]

The shear characteristics of soil usually are determined on single-shear devices. But in the Far North soils are subject to the systematic influence of periodically fluctuating positive and negative temperatures. Hence there is a question as to the influence of repeated freezing and thawing of the soil on its strength characteristics.

As a result of experiments conducted during the construction of the Vilyuy Hydroelectric Power Station it was determined that freezing followed

---

[1] Report on a research project entitled "Study of Borrow Material for the Earth Seal on the Vilyuy Hydroelectric Power Station Dam," 1962 (Kalinin Polytechnic Institute of Leningrad).

by thawing has practically no influence on the shear resistance of a soil. The shear resistance of the Vilyuy gravelly loams had average values of $\phi = 30\text{-}33^{\circ}$ and $c = 0.1$ kg/cm^2. From an analysis of the data obtained and in view of the fact that the granulometric composition of the borrow material, even though artificially mixed, will show a certain lack of uniformity, the following design values were adopted for this construction project: $\phi = 27\text{-}28^{\circ}$ and $c = 0.1$ kg/cm^2, while the control value for the bulk density of the fine-earth skeleton was 1.6-1.65 g/cm^3. The shear characteristics indicated above are provided with this exact bulk density of the fine-earth skeleton.

At the same time, when the content of coarse material (particles larger than 2 mm) is up to 55%, the strength and percolation properties of the soil are determined not only by the bulk density of the fine-earth skeleton but also by that of the entire soil mixture, including both coarse and fine material. Therefore, specifying the density of an embankment on the basis of a control value for the fine earth of 1.60-1.65 g/cm^3, in our opinion will result in a soil density that is too high, but in the final analysis this will add to the reserve strength of the structure.

In building the Vilyuy dam the bulk density of the fine-earth skeleton was determined as follows, where we knew the values of the bulk density of the soil as a whole, the specific gravity of the coarse material, and the relative content of coarse material by weight in the total volume of soil.

The weight of the fine earth P_f (particles smaller than 2 mm) in a unit volume of soil is equal to:

$$P_f = \gamma_{s.sk.} (1-P_c) \cdot 1, \tag{6.1}$$

where $\gamma_{s.sk}$ is the bulk density of the skeleton of the soil as a whole (the mixture as a whole); and P_c is the content by weight of coarse material (particles larger than 2 mm).

If the soil volume is taken as unity, then the volume of fine earth minus the volume occupied by the coarse material will be equal to:

$$U_f = 1 - \frac{\gamma_{s.sk.} P_c}{\gamma_c}, \tag{6.2}$$

where γ_c is the specific gravity of the coarse material.

The formula for determining the bulk density of the fine-earth skeleton has the following form:

$$\gamma_{f.sk.} = \frac{\gamma_c \gamma_{s.sk.} (1-P_c)}{\gamma_c - \gamma_{s.sk.} P_c}. \tag{6.3}$$

It must be mentioned that in this case by the bulk density of the fine earth skeleton ($\gamma_{s.sk}$) we mean the density of the fine earth in the spaces between the coarse and skeletal material.

The possibility of compacting gravelly loam to $\gamma_{f.sk.} = 1.60\text{-}1.65\ g/cm^3$ was studied under field conditions in the vicinity of the construction site. Layers of loam containing frozen lumps were packed into the blanket seal of an experimental dam for this purpose.

It was concluded from the experiments that it is impossible to compact soil containing a significant amount of large frozen lumps to a control bulk density of the fine earth skeleton equal to $1.60\text{-}1.65\ g/cm^3$ using dump trucks. In order to achieve the control density of the soil it is necessary to limit the content of frozen lumps to 7% with lumps no larger than 10-15 cm.

A fine-earth skeleton bulk density of $1.65\ g/cm^3$ and a permissible content of coarse material equal to 55%, the bulk density of the skeleton of the complete soil, consisting of coarse material and fine earth, according to (6-3) should be equal to $2.2\text{-}2.3\ g/cm^3$, which is close to the bulk density of concrete.

On the basis of investigations by the NIS Gidroproyekt and the Lengidroproyekt, the "Technical specifications for packing the earth seal of the Vilyuy Hydroelectric Power Station dam" were developed. These set the following requirements for the gravelly soil intended for packing into the blanket seal:

the percolation coefficient of the soil packed in the seal should not exceed $A \cdot 10^{-4}$ cm/sec;

the shear resistance parameters for the fine earth when tested in a stabilized state under water should be at least $\phi = 27\text{-}28^{\circ}$ and $c = 0.1\ kg/cm^2$;

the moisture content of the soil being placed should fall in the range of 6-13%;

boulders and pieces of frozen ground with diameters larger than half the thickness of a layer may not be packed into the body of the seal; and

soil containing more than 7% frozen lumps may not be used.

6-6. CHEMICAL TREATMENT OF COHESIVE SOILS DURING WINTER PLACEMENT

a) Fundamentals of Chemical Protection of Soils from Freezing

High-quality dry placement of cohesive earth at negative temperatures is possible if the earth remains at a positive temperature during the entire process or is in a plastic-frozen state.

The strength of frozen ground, of course, depends on the amount of frozen water in the pores of the ground. Hence the composition, structure, and properties of frozen ground are determined by the presence of ice in it as a rock-forming mineral. But, in addition to the solid ice component, frozen ground also contains liquid and gaseous components. Hence water is present in frozen ground in three states. The freezing of ground, i. e. the formation of ice in it, occurs when water reacts with the mineral part of the ground. This process and its intensity depend on the mineralogic composition of the ground as well as on the composition of the water and its mineralization. The more highly dispersed the ground, the stronger the interaction of the water with the mineral skeleton and the more complex the processes of freezing.

The following phases are distinguished in dispersed frozen ground: a solid phase, consisting of the mineral skeleton of the ground ice; a liquid phase, consisting of water which has not frozen at a given temperature and the salts dissolved in it; and a gas phase (water vapor). There is a very clear increase in the amount of unfrozen water as the degree of dispersion of the ground, i.e. its specific surface, increases, as has been shown by Z. A. Nersesova (see Figure 6-14). For an understanding of the properties of frozen ground and the processes taking place within it, one must take into account the fact that water is present in unfrozen ground as gravity or free water and bound water, and that bound water may remain in the liquid (unfrozen) state even at negative temperatures.

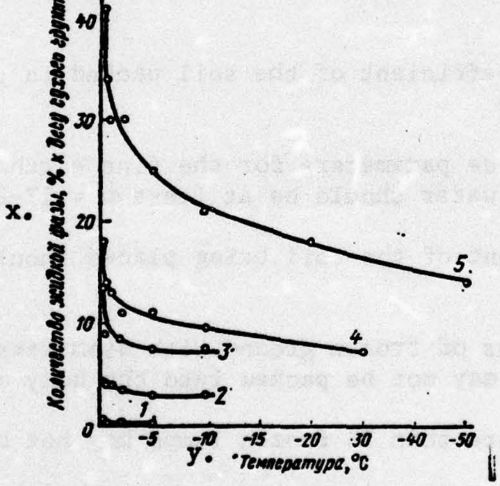

Figure 6-14. Content of unfrozen water in different types of ground as a function of temperature.
1 - Sand; 2 - Loamy sand; 3 - Loam; 4 - Clay; 5 - Clay containing montmorillonite.
Key: x - Quantity of the liquid phase, % by weight of the dry soil; y - Temperature, °C.

Z. A. Nersesova [65] distinguishes several types of water in frozen ground: water in the gas phase; water of variable aggregate state (solid or liquid, freezing or thawing at a temperature below 0°C); tightly bound water, which does not change its state with a change in temperature; and water in the solid phase, i.e. ice, which forms at a temperature of 0°C and does not thaw at temperatures below 0°C.

The temperature at which the freezing of water is initiated in the ground depends mainly on the thickness of the water films as well as on the content of salt in dissolved form. Freezing is initiated at different temperatures in soils with different granulometric compositions when their moisture content is low, as is apparent from Figure 6-14. The freezing initiation temperature in sand and loamy sand usually is about zero, while in loams and clays this temperature may be significantly lower.

If the water in the pores of the ground contains dissolved salt and the soil solution has a relatively high concentration, the freezing initiation temperature of the ground will be significantly lower.

Studies conducted under the direction of N. A. Tsytovich by staff of the Institute of Permafrostology of the USSR Academy of Sciences has established the so-called principle of the equilibrium state of water in frozen ground: the quantity, composition, and properties of unfrozen water contained in frozen ground do not remain constant but change as external influences change, being in dynamic equilibrium with the latter [98].

This principle of the equilibrium state of water and ice in frozen ground provides a basis for a correct approach to the consideration of the physical properties taking place in frozen ground, including artificially salinized ground, and for assessing the influence of external factors on changes in the mechanical properties of the frozen ground.

An important conclusion can be drawn from this, to the effect that by treating the ground in the right way one can impart to it properties such that even at negative temperatures it will remain in an unfrozen state [93].

The method of chemical protection, based on the properties and solutions, is ordinarily used to prevent the freezing of the ground or to lower its freezing temperature: the freezing temperature of a solution is lowered as the concentration of substances dissolved in it increases.

Usually the content of water-soluble salt in cohesive soil is low and the concentration of the soil solution is low. Therefore in most cases freezing of cohesive soils begins at a temperature close to 0°C. Application of a water-soluble salt to the ground has the result that, as the salt dissolves in the soil solution, its concentration increases and the freezing temperature of the soil solution declines, and therefore that of the ground itself as well.

When a water-soluble salt is applied to cohesive soil, the content of loosely bound water decreases. Then the water in the pores of the soil is present in two states: in the state of tightly bound water and in the state of a free soil solution. Since tightly bound water has no dissolving capacity, further application of salt to the ground results only in increasing the concentration of the soil solution.

By increasing the concentration of the soil solution one can produce a condition in which the ground will not freeze even though it is at a negative temperature. Thus the chemical method of controlling the susceptibility of soils to freezing during the winter consists of increasing the concentration of the soil solution by adding water-soluble salts to the ground.

When unsaturated aqueous salt solutions freeze, only part of the solvent crystallizes initially and the concentration of the unfrozen solution increases, so that its freezing temperature does not remain constant but gradually is lowered. This change in temperature continues down to a definite limit, called the eutectic point, at which both the solvent and the dissolved material are converted to the solid state simultaneously.

Formation of a eutectic mixture in solutions is observed at very definite values of temperature and solution concentration. Thus the eutectic temperature most often used in construction practice to control the susceptibility of sodium chloride to freezing is equal to -21.2°C at a concentration of 0.271 g/cm^3, and for calcium chloride this temperature is -55°C with a concentration of 0.402 g/cm^3.

Freezing temperatures for calcium chloride solutions are given in Table 6-3 [4].

Table 6-3

The Freezing Temperature of Calcium Chloride Solutions

Specific gravity of the solution at a temperature of -15°C, g/cm^3	Content of water-free salt per liter of solution, kg	Freezing temperature, °C
1,01	0,013	—0,6
1,05	0,062	—3,0
1,09	0,114	—6,1
1,13	0,166	—10,2
1,21	0,276	—23,2
1,23	0,304	—28,3
1,25	0,334	—34,6
1,27	0,368	—43,6
1,285	0,402	—55,0

When the ground temperature is not below -21°C, use of sodium chloride is recommended; if the ground temperature drops below that value, then calcium chloride should be used. The amount of salt (in kilograms) to be applied per square meter of ground may be determined from the following formula [93]:

$$Q = \frac{1000\gamma_{gr}W_d at^n \cdot 1}{(100+W_{gr}p)(\gamma_s - at^n)},$$

where γ_{gr} is the bulk density of the ground (g/cm^3); t is the absolute value of the negative temperature (°C); W_d is the amount of water involved in dissolving the salt (%); W_{gr} is the moisture content of the ground (%); γ_s is the bulk density of the solution (g/cm^3); and a and n are factors which depend on the type of salt used and the air temperature (see Table 6-4).

Table 6-4

Temperature intervals (°C)	a		n	
	NaCl	$CaCl_2$	NaCl	$CaCl_2$
From 0 to -4	0.0167	0.023	2.0	0.92
From -4 to -14	0.021	0.033	0.86	0.68
From -14 to -21.2	0.045	0.037	0.59	0.65
From -21.2 to -32	--	0.051	--	0.54

Studies have shown that a content of water-soluble salt in the ground not exceeding 2-4% of the soil mass does not produce significant changes in the basic structural properties of the soil. Salinized cohesive soil can be reliably recommended for water-retaining structures built by the unfrozen method. The process of salinization of the soil takes place over a long period of time, and the change in basic structural properties of the soil in this case is also small [93].

It appears that salinization of soils is not permissible for water-retaining structures built by the frozen method.

b) Treatment of the Layer Interfaces with Salt Solutions

More and more use is now being made of the technique of treating interfaces with salt solutions when using cohesive soils for fill in the winter. The first large-scale use of such treatment under production conditions was in the building of the Vilyuy dam. Experiments conducted at the construction site of the Vilyuy dam preceded broad use of chemical treatment of the soil.

Experimental salinization of work-area surfaces was carried out in the laboratory and then directly at the work site. After being leveled out with a bulldozer, the soil surface was flooded with a concentrated calcium chloride solution prior to compaction. After compaction the surface of the layer was soft, relatively flat, and smooth, and there was no doubt about the quality of the contact with the succeeding layers. It was determined that the portion of the layers treated with salt to a definite depth remained in a soft condition even at negative temperatures as a result of the reduction in the freezing temperature of the soil solution, and this, of course, tended to improve the compaction of the soil, especially at the interface between layers.

The moisture content of the free soil solution, or the content of water involved in dissolving the salt (W_{gr}) was determined as the difference between the average moisture content of the soil by weight and the content of tightly bound water (W_{tb}).

With an average soil moisture content by weight of 10% and a content of tightly bound water of 1%, the average moisture content of the soil solution was considered to be 9%. The average content of coarse material was 52%. Therefore with a control value for the bulk density of the fine-earth skeleton equal to 1.6 g/cm^3, the bulk density of the soil skeleton calculated by formula (6-3) was equal to 2.1 g/cm^3. The absolute moisture capacity of the soil was 13.6%. The salt concentration per liter of solution was assumed to be 368 g of calcium chloride and 271 g of sodium chloride.

When the soil was treated with the solution, the upper limit of moisture content was determined by the cumulative moisture content (the moisture content of the soil plus the water introduced with the solution), which was equal to the moisture capacity at a soil skeleton bulk density of 2.1 g/cm^3. Further increase in the moisture content, as experience has shown, degrades the compaction properties of the soil and creates supersaturated layers.

The temperature at the surface of the soil at the moment compaction of a layer is finished drops to -6 or -8°C, according to observed data, and the temperature at a depth of 3-4 cm ranges from -1° to +2°C. These results show that the amount of solution used depends not only on the moisture content and bulk density but also on the depth of salinization, the soil temperature, and the thickness to be treated with the salt solution. Calcium chloride treatment of a soil layer 10 cm thick in a compact object at a rate of 0.5 l of solution per square meter will lower the freezing temperature of the soil solution to 0.5°C, as indicated by studies conducted during the building of the blanket seal for the Vilyuy rock fill dam; salt treatment of a layer 2 cm thick will lower the freezing temperature to -2°C, etc.

But the freezing temperature of the soil solution will be significantly higher than the temperature which is established in the soil during the period of compaction. Salt treatment of the surface layer, for example, to a depth of 0.5-1.0 cm cannot significantly improve the compaction qualities of the soil, since with a freezing depth of 10-15 cm a layer only 0.5-1.0 cm thick will be in a soft condition. Therefore the thickness of the layer treated should be close to the freezing depth of the soil at the moment of compaction, and the temperature of initiation of freezing of the soil solution should be close to the temperature of the soil. Observations in the field have shown that when the freezing depth at the end of compaction is 7-9 cm, the depth of salt treatment should be 5-6 cm at a rate of 2.5-5 l of solution per square meter. In this case the depth of salt treatment is comparable to the depth of freezing and therefore the negative effect of freezing is reduced to a minimum.

The data given above are rather approximate, since it was assumed that the salt applied to the soil was evenly distributed across the depth of the layer treated. In actuality, however, the concentration of the soil solution decreases with depth, and consequently the soil freezing temperature varies and the salt applied to the soil is used more efficiently.

Experience with construction of the Vilyuy dam has shown that the rate of application of chloride solutions should be about 2-4 l/m^3 of surface treated depending on the air temperature and the rate of rolling. This will assure salt treatment in the upper portion of a layer to a depth of about 5 cm. Salt treatment must be done immediately after the soil is leveled out while the surface has still not had a chance to cool off. In this case the salt will penetrate more rapidly into the soil. The rate of application of the solution was strictly controlled in construction of the Khantayka dam; it was no more than 1.5-2.5 l/m^2 [44].

The solution concentration should be maximum. One must try to add as little water to the soil as possible, since compaction of supersaturated soil under winter conditions is difficult. When air temperatures rise, the concentration of the solution should not be reduced but only the rate of application.

For more even salinization it is necessary to furrow the surface of the soil. It is desirable to combine the processes of irrigation and furrowing in a single piece of processing equipment.

The technique for salt-treating the surface of a work area adopted at the Vilyuy Hydroelectric Power Station project was as follows.

After the surface of the soil had been spread and leveled, it was irrigated with a highly concentrated solution of sodium or calcium chloride at a rate of 2-4 l/m^2 using a special truck equipped with a sprinkler apparatus (see Figure 6-15). Since the moisture content of the soil was relatively low (8-10%), salt treatment with this small amount of solution gives good results and did not cause excessive wetness. The upper and lower parts of the layer, which encountered the most unfavorable temperature conditions, were maintained in a plastic state during the entire compaction period.

The same technique of treating the contact zones with a salt solution was used in building the Khantayka Hydroelectric Power Station [44].

Rolling was begun only after irrigation with the solution, since application of the salt to a compacted surface does not produce the desired effect. The salt penetrates a rolled layer only to a shallow depth, so that the layer of soil freezes, producing cracks in its upper surface. Additional irrigation of the surface with the solution leads to the formation of a supersaturated soil "paste" on the surface, which only hides the surface formation of cracks and creates the appearance of a good-quality surface. If such a layer of soil is examined in profile by digging pits, the following structure is seen: from the surface to a depth of 0.5-0.0 cm there is a soil-salt "paste," then a frozen layer 5-7 cm thick covered with a large number of cracks, and only below this do we find well-compacted unfrozen soil.

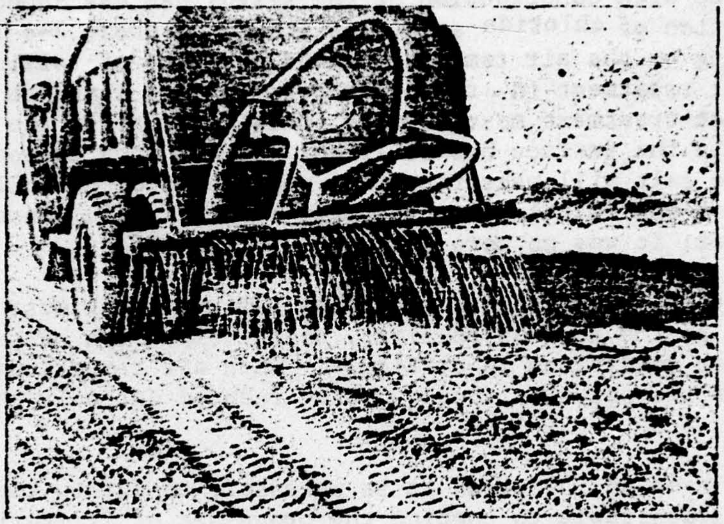

Figure 6-15. Irrigation of contact zones with a calcium chloride solution.

Before covering a compacted area with the next layer, its surface is again irrigated with the salt solution. Creation of this salt "bed" promotes relatively rapid interaction between the lower zone of the soil fill and the salt solution. This in turn assures that the soil in this zone remains in a plastic state and makes it possible to compact it before it begins to freeze. Careful salinization of the lower zone of the layer of fill is particularly important, since the temperature of the previously placed layer drops rapidly after it has been compacted, especially if there are no interruptions in the placement.

At the moment of placement there is a strong supercooling of the fill soil at the interface between the layers which interferes with compaction. In order to prevent this supercooling, as was already mentioned, a hot jet device is used during the salt treatment process. The high temperature of the exhaust gases of this device assure effective heating of the soil surface.

Experiments were conducted on the placement of loams with a high moisture content (17%) in which the soil was treated with sodium chloride in powder form. But the salt powder dissolved very slowly and even treatment of the soil was not achieved. Therefore individual areas were able to begin freezing before compaction. Harrowing the surface of such a layer also had little effect, since repeated harrowing of soil which actually was still not salinized resulted only in additional losses of heat from it.

Tests conducted under operational conditions showed that even in the case of supersaturated soil the surface must be salt-treated with highly concentrated solutions and not by applying salt in powder form, provided this does not produce excessively wet soil.

Surface treatment of the soil together with preliminary heating of the surface prevented the formation of cracks and assured a solid contact between the layers. A sufficiently smooth and compact surface was produced and the ground was in a plastic condition, which guaranteed good contact between the layers.

It must be mentioned, in spite of the steps taken, the necessary fill quality is not always achieved. Sometimes the elimination of loose frozen soil requires careful manual treatment of the surface, heating with a turbojet device, and additional irrigation with a concentrated salt solution. Such treatment of the contact zones between layers requires considerable investment of manual labor (about 130 man-hours per 1,000 m^3 of loam placed) and substantially slows down the progress of work. Therefore one must try to keep the work going continuously throughout the entire winter fill season, organizing on a sliding curve shift changes, meals, and days off, reducing interruptions in the work to a minimum.

When there is snowfall, good results are obtained from additional irrigation with the solution. This promotes melting of the snow on the surface of the work area and prevents the formation of an ice crust on it. The snow slurry that is formed is moved away from the work area with graders or it is blown away with a turbojet device operating at high speed. In spite of some additional complication and cost resulting from the use of additional salt treatment and machinery, this technique assures continuous operations and work of good quality.

A special installation is recommended for preparing the salt solution (see Figure 6-16). After being measured out, the calcium chloride is broken up into pieces, dropped into a heated 20 m^3 tank, and dissolved in water heated with steam, which is fed in under a pressure of 1 kg/cm^2. As it is ready the highly concentrated solution is poured from the first tank into a distribution tank at a lower level. From here it is delivered to the fill area in a special tank truck. This system of preparing the solution is quite simple and reliable. Up to 20-30 m^3 of solution can be prepared in a day.

6-7. SURFACE HEATING IN THE WINTER PLACEMENT OF COHESIVE EARTH

The need for heat treatment of contact zones in addition to chemical or salt treatment in the winter packing of cohesive soils, as experience in the construction of the Vilyuy and Khantayka dam has shown, is dictated by the fact that during leveling and rolling the temperature of the newly placed soil rapidly drops. Rapid cooling of the layer occurs not only from the surface but also from the previously packed layer underneath. The latter situation is much more hazardous, since the lower zone of the layer is under less compaction pressure and control of soil freezing there is difficult.

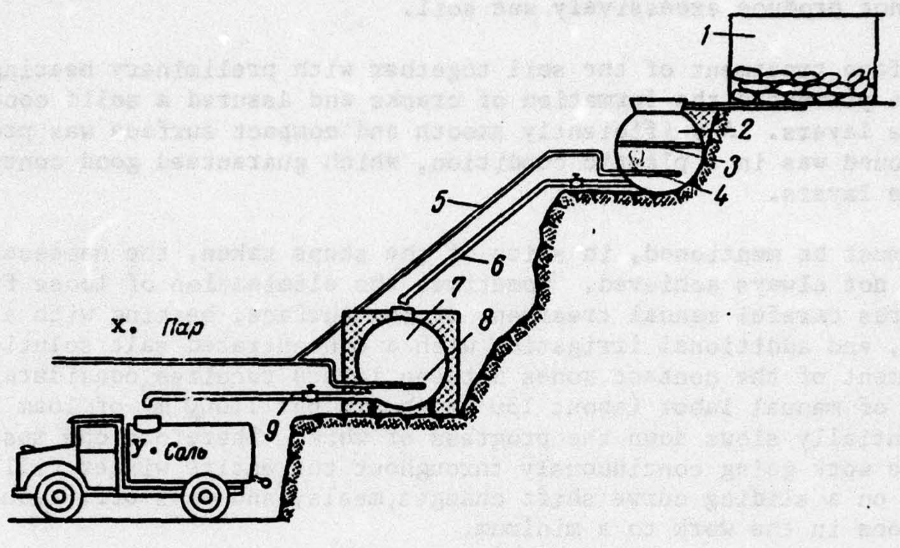

Figure 6-16. Diagram of an installation for preparing chloride solution.
1 - Storage shed for chloride salts; 2 - Upper tank; 3 - Screen; 4 - Perforated pipe; 5 - Steam pipe; 6 - Drain pipe; 7 - Lower distribution tank; 8 - Heating grid; 9 - Fill pipe.
Key: x - Steam; y - Salt.

During compaction a frozen crust forms on the surface of the working area. The thickness of this crust increases with time, and after a certain amount of time it becomes so thick that it begins to prevent further compaction of the soil below. The surface crust of the layer, which has frozen and lost its plasticity, is hardly compacted at all, while the soil underneath is still in the unfrozen state and could be compacted. If special steps are not taken to increase the plasticity of the freezing crust, a network of numerous cracks appears on the surface of a layer that has been laid and compacted (see Figure 6-17). Because of the lack of cohesion with the soil below, the surface crust, which is covered with cracks, easily breaks away along the interface of the frozen soil.

In order to avoid these phenomena, heat treatment of the contact zones is very effective in the winter packing of cohesive earth. An industrially produced turbojet device, type TM-59, was used for this purpose in building the dam for the Vilyuy Hydroelectric Power Station. This device is ordinarily used to clear airport runways of ice and snow in the winter.

The apparatus (see Figure 6-18) consists of a VK-1-A turbojet engine equipped with a long tube and a short funnel-shaped nozzle. The engine is mounted on a D-452 chassis in such a way that the jet pipe can be rotated

30° in the horizontal plane to either side of the central axis and the nozzle can be tipped 20° from the horizontal.

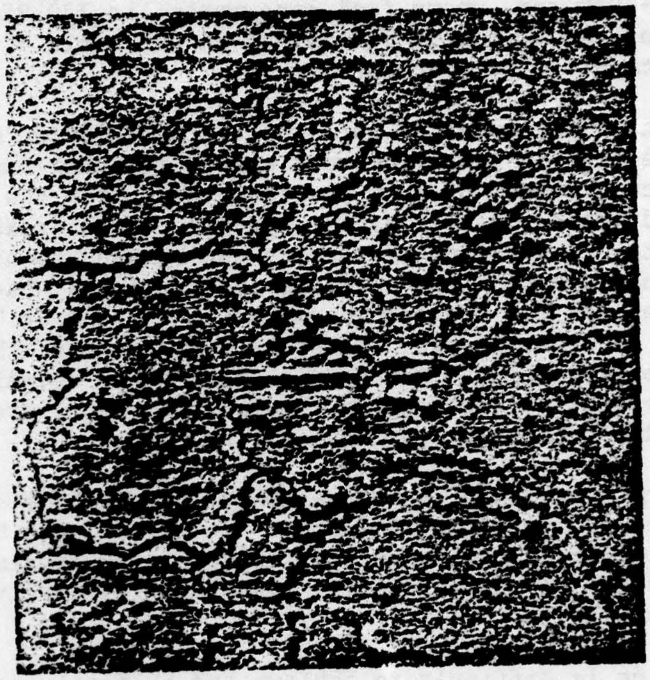

Figure 6-17. Cracks on the surface of a layer.

Figure 6-18. The TM-59 heating device.

The engine warms up at a speed of 2500 rpm. The maximum operating speed of the engine (9000 rpm) provides a gas-air mixture with a temperature of about +540°C with maximum jet speeds of 100 m/sec. The device is normally operated at 5000-7000 rpm, where fuel consumption is 750-950 kg/hr. The operating speed of the engine is determined on the basis of the nature of the surface treatment to be performed

During winter placement of soil the use of turbojet devices is particularly effective in preparing work areas, removing frozen lumps, uncompacted areas, accumulations of snow and ice, etc., from the surface of previously placed areas, as well as for heating frozen areas before they are treated with salt. In addition the device can be used to heat contact zones between embankments and concrete surfaces and previously packed areas. The device can be used for these purposes while running at "low speed" (300-4000 rpm). In order to avoid possible overdrying of the soil, since the temperature of the air-gas mixture is quite high, reaching 300-400°C, one must strictly control the heating time of individual areas and the movement of the device itself over the fill area.

After a short period of such treatment, the surface soil is heated to a shallow depth not exceeding 1-1.5 cm. But since such heat treatment of the surface is carried out immediately before salt treatment and the laying of new batches of unfrozen soil, even this relatively shallow heating gives positive results.

Deep heating is difficult to achieve and evidently not needed, since a considerable portion of the heat is lost to the atmosphere during prolonged treatment, and prolonged application of high temperatures dries out the soil too much, which has a bad effect on the attachment of the layers.

After the Vilyuy dam had been built, heat treatment of contact zones was used successfully in building the Khantayka Hydroelectric Power Station dam [44].

6-8. BORROW PITS. DIGGING AND STORING THE SOIL

Borrow pits for cohesive earth can be worked in the Far North only during the summer. Thawing of the ground begins in the middle of May and ends in September. The ground begins to freeze at the surface in the first half of October.

Borrow pit management is affected by the frozen condition of the earth in the pit, the diverse particle-size composition and moisture content of the earth from top to bottom, the short summer season, and the large scale of the earthwork involved.

Borrow materials for building dams are selected on the basis of engineering geology studies, and other conditions being equal, preference is given to deposits that are closest to the construction site.

Preparatory operations for working borrow pits include: cutting the trees, removing the topsoil, and digging ditches to carry away surface water. When the soil is very wet, additional engineering melioration steps are required.

The thickness of the active layer depends on such factors as the exposure of the tract, the moisture content and lithology of the earth, the presence of trees in the area, the presence of a moss-peat layer, etc. At the end of summer the active layer is rarely deeper than 1.2-1.5 m. Therefore in order to have the necessary amount of unfrozen soil in the winter, it must be prepared in the summer, and this requires the use of special techniques for working the borrow pits as well as a whole set of measures for the engineering melioration of the pits.

Work begins with the removal of snow from the borrow area and cutting the trees. This work is usually done with bulldozers, since the frozen wood at the beginning of spring is relatively easy to cut away from the roots and remove together with the brush and snow. The cleared borrow area is warmed by the spring sun long before the now cover has disappeared completely.

Topsoil removal begins when it has thawed to a depth of no more than 10-15 cm. Special attention must be given to this process to see that it is done before thawing has extended beyond the layer to be removed. Otherwise good mineral earth is removed with the topsoil, and this is most undesirable. In the Far North the useful borrow material, which is the weathering product of the bedrock, usually lies within the active layer and is no more than 0.6-1.2 m thick. Because of this, in order to obtain sufficient quantities of earth with the required particle-size composition, the area of the borrow pits is large.

Further working of the borrow pit and stocking of loam are also done with bulldozers as the ground thaws. In the warm days of spring and summer the earth usually thaws to a depth of 10-15 cm during the day. Bulldozers operating on the frozen layer beneath cut away the earth and move it into special piles (see Figure 6-19). Then it is loaded with shovels and transported to winter storage mounds or to the dam site, if placement is to be done in the summer. This relatively simple method of acquiring cohesive earth can be used in dry borrow pits, i.e., where the ice content of the earth in the pit is small. But such favorable conditions are very rare in the permafrost zone. The ice content of permanently frozen ground is typically high, and special engineering preparation of the borrow pits or engineering melioration is required.

We know that in permafrost regions the solar radiation reaching the earth during the spring (April-May) is only 10-15% less than the summer maximum, and the total radiation for each of these months amounts to about 13.5 kcal/cm^2 [67]. Timely removal of the snow cover permits full utilization of available heat for heating the surface layer by reducing reflection and saving the heat that would have been used to melt and evaporate the snow cover.

Figure 6-19. Digging loam.

Storage of earth in mounds for winter placement was first used in building the Vilyuy Hydroelectric Power Station [4], and later in building the Khantayka Hydroelectric Power Station [44]. Initially the mounds were built only with bulldozers. At that time the mounds were relatively low (up to 6-7 m), the gradient of the slopes was 1:3-1:4, and their specific surface was quite large (0.28 m^2/m^3). During the winter months such small mounds were subject to heavy freezing and at the end of winter no more than 40-50% of the earth remained unfrozen with a maximum temperature of no more than +4°C. In obtaining earth from a mound the top frozen crust had to be broken with explosives, and when the soil was mixed, its temperature dropped still further as it was being handled, so that soil with temperatures no higher than +0.5° to +0.7°C was delivered to the work site, and this was not high enough at all for fill work under winter conditions.

Experiments conducted in connection with the construction of the Vilyuy Hydroelectric Power Station showed that temperature drops during the digging, transportation, and compaction of such soil amounted to 6-10°C, depending on a whole set of factors. Therefore in order to assure that the soil delivered to the building site would have temperatures of +2° to +3°C, the minimum soil temperature in the mound would have to be at least +12° to +13°C. This is practically impossible to achieve when the soil is stored in small mounds. On the basis of results obtained with the winter placement of loam in the seal of the dam, it was decided to build winter storage mounds 16-18 m high and to concentrate up to 25,000 m^3 of earth in a single large mass.

Earth scooped up during the summer with bulldozers as it thawed was left in heaps for 2-3 weeks. During this time its temperature rose significantly and the moisture content dropped accordingly. Then the earth was loaded on trucks with shovels and moved from the drying piles to the winter storage mounds.

Based on the experience of Vilyuygesstroy, which was repeated at the Khantayka construction project, it is recommended that the mounds be trapezoidal in shape, 30-40 m wide at the top and 16-18 m high, with natural slopes of 1:1.4 (see Figure 6-20). The specific surface of such mounds is significantly less than that of small mounds, amounting to 0.12-0.13 m^2/m^3. Storage of soil in large mounds keeps the temperature higher and eliminates soil waste when it is dug in the winter.

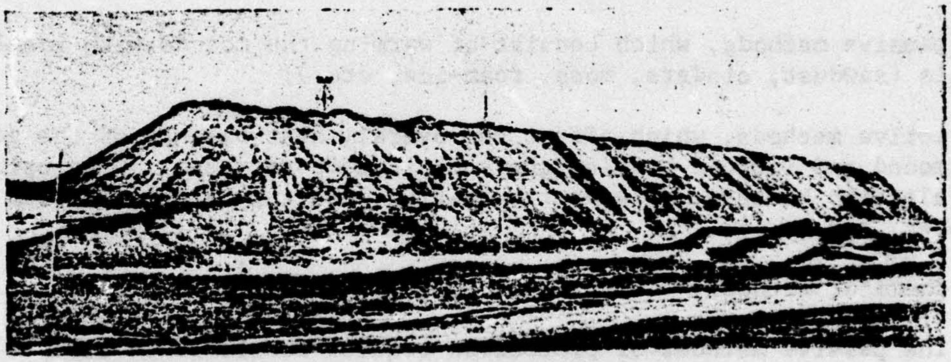

Figure 6-20. Winter soil storage mounds with a volume of 250,000 m^3.

The mound should be piled up in one direction to the full height at one time. In this way the density of the earth in the mound remains low, not exceeding 1.7 g/cm^3. The moisture content of earth stored in mounds equalizes, so that the moisture content of earth placed in the seal shows little variation, averaging 8-10%. When the mound is piled up to the full height at one time, the soil is also well mixed and its particle-size composition is evened out. Material was segregated as the mounds were built to remove boulders, since the technical specifications do not permit them in the seal.

The earth is usually transported from the borrow pits in dump trucks, and very rarely in tractor-drawn wagons and earthmovers. Even though the use of earthmovers in dam construction is very efficient because of their versatility (they dig, transport, and distribute the fill over the work area) as compared with other types of machinery, there has been very little experience with their use in constructing dams in the North. The reason for this lies in the very specific climatic and ground conditions there and the limited length of the warm season during which they can be used. When cohesive earth is used in dam construction during the winter, it is carried from the winter storage mounds in trucks with special heating arrangements to reduce heat loss in route. The truck bodies have double walls, and the engine exhaust gases pass between them. The top of the soil in the trucks is covered with special blankets.

With this type of transportation, as special studies have shown, heat losses en route at air temperatures as low as -40°C and over a distance of 5.5 km did not exceed 1.5-2.4°C [4].

6-9. WINTER STORAGE OF EARTH IN MOUNDS

a) Techniques of Winter Earth Storage

Building large mounds for the winter storage of soil does not provide complete protection from freezing. Additional steps must be taken to keep the depth of soil freezing at the surface to a minimum. Several protective methods are used for this purpose:

passive methods, which consist of warming the mounds with insulating materials (sawdust, cinders, moss, foam-ice, etc.);

active methods, which affect the overall heat balance of the earth in the mound and include heating the soil in temporary enclosures using steam, electric heaters, or electric heating with superficial or deep electrodes; and

chemical methods.

The passive methods of protection require considerable amounts of insulating material. When mounds warmed in this fashion are excavated, removal of the thermal insulation is a lot of trouble, the soil is contaminated, and as a result there is considerable waste. The construction of enclosures with various kinds of heating systems is no less complicated. Sometimes deep excavations, narrow stream conyons, isolated potholes, and the like are used to store soil for winter placement. In these cases portable enclosures are used to warm the working base when the soil is excavated. This system of thermal protection permits better conservation of the heat in the soil, but it is very expensive and requires a large quantity of scarce construction material.

The chemical method of protection also has its positive and negative aspects.

Based on the experience of Vilyuygesstroy, the best solution involves a combination of the passive, active, and chemical methods of protection, such as the salt treatment of the peripheral areas of a mound, electric heating, and thermal insulation of the surface of the mound by covering it with foam-ice.

b) Salt Treatment of the Peripheral Zones of Mounds

Earth fill for the peripheral zones of mounds was prepared in the following fashion during construction of the Vilyuy Hydroelectric Power Station, and later at the Khantayka project. Sodium chloride or calcium chloride was spread on earth to be treated in the borrow pit uniformly at a rate of 20-30 kg/m^2. After 2-3 weeks the earth was pushed into piles, from which it was delivered to the mounds and placed on the slopes in a layer 2-2.5 m thick.

The surface of a mound was salt-treated in place. This was accomplished by distributing salt uniformly over the surface and wetting it down. Small ridges were built to prevent loss of the brine. To promote more uniform salinization and increase the insulating properties, the surface of the mound was dug up to a depth of 3 m with a drag-line after the fill work was finished. Analysis of water extracts of samples taken from the surface of the mound showed that with this sort of treatment the salt was spread up to 3 m deep and the salt content ranged from 11.7-34 kg per cubic meter of soil. With this sort of salt treatment the peripheral zones of the mound did not freeze as deeply and the electrical conductivity was increased for subsequent use of electric heating techniques.

c) Covering the Mounds with Foam-Ice

In building the Vilyuy dam, loam was covered with a layer of foam-ice using a technique developed by the VNIIG. Artificially produced foam was applied to the surface of the mound. The negative temperatures caused it to freeze and form a protective layer which resembled natural snow in its mechanical and thermal properties. Investigations conducted at the construction site showed that a layer of foam-ice 0.5 m thick reduces the depth of soil freezing by 1 m.

The ice foam was produced with a relatively inexpensive foaming material (PO-6) and simple equipment installed in a portable enclosure.

The apparatus works as follows (see Figure 6-21). A 4% mixture of the foaming agent with water is pumped into an air mixer, where it is vigorously mixed. The foam produced is delivered through a rubber hose to the placement site. The air fed to the mixer is first heated in order to improve foam formation, to increase the stability and number of fubbles in the foam, and to prevent the formation of ice in the hoses at very low temperatures.[1]

The foam output of the apparatus with tenfold foam expansion is 200 m^3/hr; i.e. it can cover 400 m^2 of mound surface with a half-meter layer of foam-ice in one hour. We do not feel it desirable to increase

---

[1] The preliminary heating was suggested by V. S. Filippov, a Vilyuygesstroy engineer.

the output further because if it is placed too fast the lower layers of foam do not solidify and they might collapse.

The effectiveness of the use of ice foam depends on starting at the right time. Placement of ice-foam must begin only after temperatures have dropped permanently into the negative range. If work begins too early while there is still a possibility that temperatures will rise, the warmth might cause the foam to collapse. On the other hand, delay means significant heat loss from the surface of the unprotected soil and reduces the effectiveness of the ice-foam application. Application of the ice-foam should begin on the north side of the mound. This reduces the bad effect of any possible warm periods and protects the northern part of the mound from heat loss, where losses are most rapid.

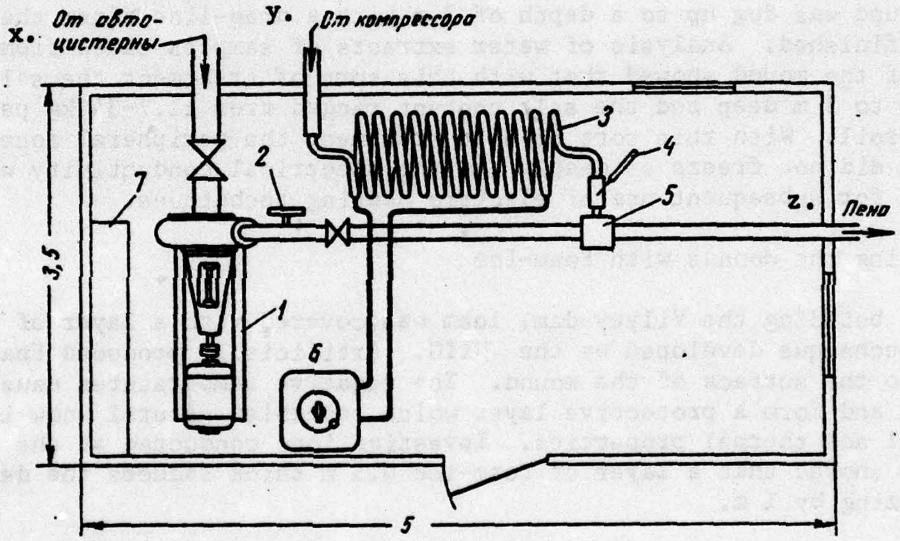

Figure 6-21. Apparatus for producing ice foam.
1 - Pump; 2 - Manometer; 3 - Air heater; 4 - Thermometer;
5 - Air mixer; 6 - Transformer; 7 - Switch panel.
Key: x - From tank truck; y - From compressor; z - Foam.

Freshly placed foam has a very low coefficient of thermal conductivity, and for this reason a thick layer of foam does not harden for a long time even when temperatures are quite low. Therefore the foam should be applied in layers no more than 10 cm thick and new layers should be added only after the earlier layer has hardened.

d) Electric Heating of Peripheral Zones of the Mounds

As Figure 6-22 shows, even when treated with salt at the surface the earth freezes to a depth of 2 m or more and frozen ground also reappears along the base of the mound.

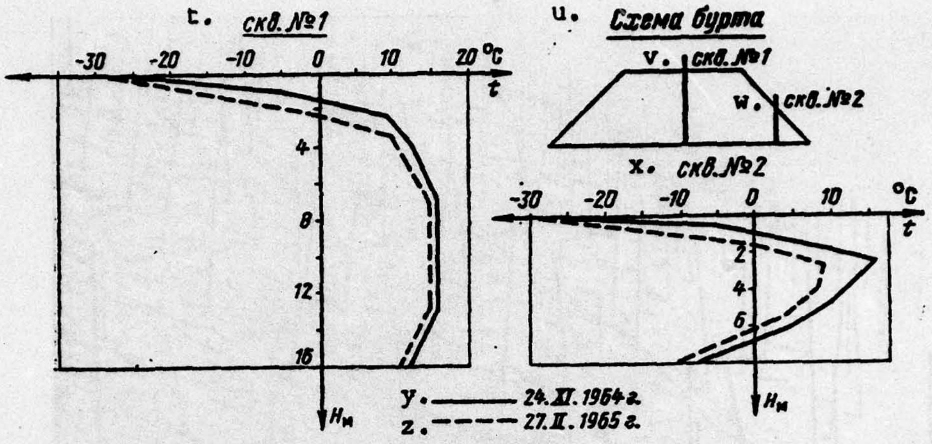

Figure 6-22. Temperature conditions in a winter storage mound of loam.
Key: t - Hole No. 1; u - Diagram of mound; v - Hole No. 1; w - Hole No. 2; x - Hole No. 2; y - November 24, 1964; z - February 27, 1965.

Electric heating improves the thermal regime of the mound as a whole, reduces its content of frozen lumps, and makes excavation of the mounds easier.

At the Vilyuy Hydroelectric Power Station 36x6 kV substations rated at 3150 kVA with voltages of 380, 1000 and 6000 V were used for electric heating of mounds. The electrical heating procedure was as follows. In the fall electrodes made of rebar stock 19 mm in diameter were inserted into the surface layer of the mound to a depth of 0.5-1 m (see Figure 6-23). By the middle of winter, i. e. when it was time to excavate the mound, the ground had frozen at the surface to a depth of about 2.5 m. Before excavating the mound voltage was applied to the electrodes. As heating began the current reached a level of 250 A or more, and then gradually dropped to 50 A. After the voltage was removed, the electrodes were driven in again to a depth of 40-50 cm and voltage was applied again. After the second insertion of the electrodes the current dropped smoothly below 50 A. The loam in the upper part of the mound thawed throughout, reaching temperatures of +8° to +12°C.

It was noted that after the current had dropped below 50 A further application of voltage to the area being heated was not effective because the heating process stopped. Formation of a layer of air 1-4 cm wide around the electrodes when they were driven into the mound and while the loam was thawing was unavoidable, and this reduced the heating effectiveness. The heating effectiveness was also reduced by a layer of baked loam 5-15 cm that formed where the electrode came in contact with the soil.

Excess amounts of salt represent a significant hazard in the electric heating of mounds, particularly local accumulations of salt in the soil. At the same time unsalted loams are difficult to thaw; they require greater expenditures of labor, more closely spaced electrodes, etc.

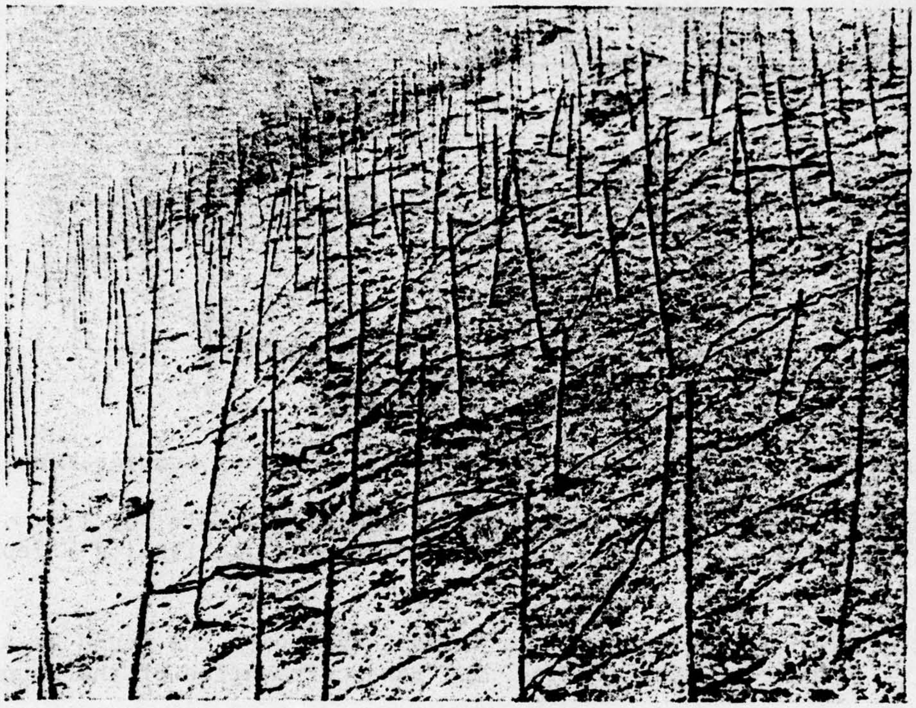

Figure 6-23. Preparation of a mound for electric heating.

Figure 6-24 shows a diagram of the arrangement of electrodes in salt-treated and untreated areas. When a voltage of 1000 V was used to treat unsalted loam, the electrode spacing was 0.7x0.5 m, and the volume of soil thawed at one time with this arrangement was 800-1000 m^3. Thawing of salinized loam was more efficient: 16-20 electrodes were inserted in a row, i.e. half as many as in the earlier case, and the electrode spacing was 1-1.2 m. The volume of loam thawed during the same period of time (26-30 hr) was 1400-1700 m^3.

The temperature of the thawing loam rises above zero 18-24 hours after heating starts. The process of thawing rises slowly in the temperature range from -6° to -2.5°C, and then the rate of heating increases and the current drops.

The mound is excavated after it has been thawed. When the electrodes are removed from one area, a second area is heated, and work proceeds in a third area to place the electrodes and prepare it for heating.

Thawing of 1 m^3 of soil and raising its temperature to +8°C takes about 40 kWh of electricity. But because of interruptions in the placement of the loam at temperatures below -50°C, when placement was halted it was necessary to maintain a voltage on areas already heated. This increased the electricity consumption per cubic meter by 5-10%, depending on the length of the interruptions.

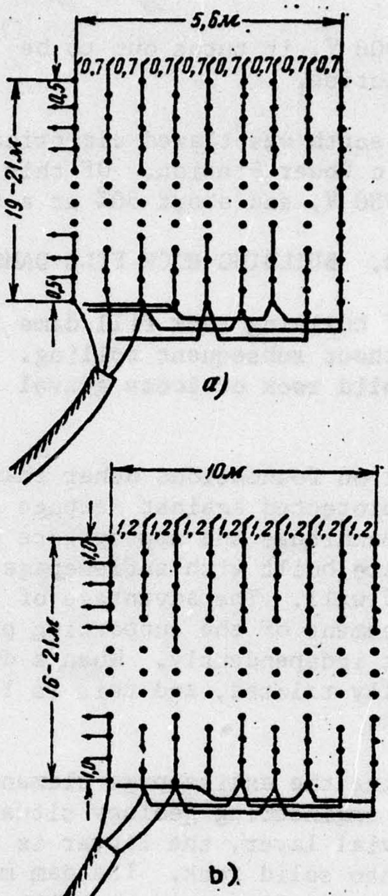

Figure 6-24. Diagram of electrode hookup.
a - Soil not treated with salt; b - Salt-treated soil.

Heating soil at a voltage of 380 V is less efficient than at 1000 V, since it requires the installation of a large number of transformers rated at 560 kVA. Almost twice as many rods have to be used, a large amount of flexible cable is needed, etc. The electric energy used for heating at 380 V is the same as at 1000 V, i.e. 40 kWh per cubic meter, but the time needed to heat the loam to a temperature of +8°C is 80-100 hr. In our opinion, therefore, it is most economical to heat the soil with a voltage of 1000 V.

During construction of the Vilyuy dam experiments were also conducted on heating loam at a voltage of 6000 V. It was found that when this voltage is used the air temperature should be always negative and there should be no accumulations of salt in the soil being heated. The presence of salts or positive temperatures even when solar radiation heats a layer of loam 1 mm thick causes short circuiting. In addition, use of 6000 V causes the current to spread some distance around the area being heated, making it impossible to work close by and to prepare adjacent areas. In spite of the attractiveness of using 6000 V, it turns out to be less effective than 1000 V and was not adopted in practice.

About 70,000 m^3 of earth was thawed electrically during the building of the Vilyuy Hydroelectric Power Station. Of this amount about 5% was heated at 6000 V, 15% at 380 V, and about 80% at a voltage of 1000 V.

6-10. BUILDING ROCK FILL DAMS

The basic method of building rock fill dams is the placement of rock in layers with or without subsequent rolling. The foundation of rock fill dams may be either solid rock or loose gravel and stone, gravel and sand, or sandy materials.

When dams are built on foundations other than solid rock, the foundation materials are protected against seepage and undercutting by appropriate antiseepage techniques and appropriate design of the subgrade contour. Rock fill dams are built with antiseepage elements in the form of seals and a core or cutoff wall. The advantage of rock fill dams with blanket seals is that placement of the supporting prism and the antiseepage element can be carried out independently. When a dam is built with a core, these processes are mutually related, and this is less convenient as far as scheduling is concerned.

The method of joining the antiseepage elements of a dam to the foundation depends on the engineering geology situation. When the rock is covered by a shallow alluvial layer, the latter is usually penetrated by a concrete or other tooth into solid rock. The dam may include sheeting, grouted or other curtains, and aprons may be built when there are blanket seals of loam, and in mixed dams.

Sharp changes in profile should not be permitted where the dam abuts the valley slopes, since this may lead to local deformations of the dam which will necessarily cause the blanket and core to fail.

For best compaction it is usually recommended that the body of a dam be filled in lifts of 3-5 m. Filling the body of a dam in thin layers involves the building of a large number of access roads, and this increases the total cost. There are a number of advantages to filling in large lifts, since this tends to speed up and reduce the cost of construction and provide better compaction of the fill.

The supporting prism of the Vilyuy rock fill dam was filled in lifts of 10, 15, and 25 m depending on operating convenience and the need to build access routes to the fill site along the steep slopes of the river valley. Within the construction excavation the rock was filled in layers 3.5 m thick. This was necessary because the dam in this area was to be built to a height of 50 m during the six winter months and water pressure would be applied when the reservoir was filled by the spring flood.

Usually when the fill is in large lifts, the rock fill is compacted by the passage of conveying equipment over the fill work area. The use of ordinary rollers is practically impossible when filling in lifts of 10-25 m. The use of extra heavy rollers to compact rock fill has hardly any detectable effect in comparison with the difficulties and complications this produces in planning the work.

Monitors are very often used abroad to compact rock fill. A jet of water delivering 3-4 m^3 per cubic meter of water at a pressure of 10-15 kg/cm^2 is directed against rock dumped on the slopes. This treatment of the rock fill promotes better sorting of the material, denser packing, and flushing out of silt and clay particles. But as experience has shown [15], monitors can be used to compact rock fill in the Far North only during the short summer season.

Different authors limit the quantity of rock fines in the fill to 5% (Ye. A. Zamarin and V. V. Fandeyev) or from 3-5% to 7% (M. M. Grishin). But there are no definitions as to what should be considered as rock fines, what particle sizes should be limited. As far as coarser material is concerned, recommendations are even more inconsistent. Grishin defines his specifications for rock in such fashion that the ratio of the largest size to the smallest should not be larger than 3-3.5, but the minimum weight of individual fragments should be no less than 80 kg, while in high dams the content of such rock should not be more than 20-30%, and the remainder should be larger.

Practice has shown that such fill is possible with very careful classification of fragmented rock in the quarry and the building of grading facilities, which is a very tedious operation. We recommend that the fill consist of rock whose size distribution results from explosions where the pattern of holes and placement of charges are controlled on the basis of experiment. But wide latitude in choosing these parameters is available only when the rock is obtained from quarries. When rock is obtained for the dam from excavations for other purposes, the nature of the explosions is limited by the need to preserve the contours of such excavations and protect the integrity of the rock in which the structures are built.

Experience in building the Vilyuy Hydroelectric Power Station has shown that a rock fill supporting prism for a dam can be built of rock (obtained with explosives from quarries or excavations for other purposes), if the physical and mechanical properties of such rock (strength and frost resistance) are suitable for placement in a dam. At the same time the specifications for rock quality in different zones should be different (see Figure 6-25). Thus rock without additional requirements as to frost resistance can be placed in zone I. The body of the dam in zone III is subject during use to the action of variable temperature and moisture conditions, and therefore the rock used in this zone should have maximum frost resistance. In the case of the Vilyuy dam, the rock placed in this zone according to the specifications had a compressive strength under live load of at least 600 kg/cm^2 after 150 freezing cycles. The content of fines (particles smaller than 100 mm) was allowed to be 15% or less, while the content of clay, loam, and soil could be no more than 5%.

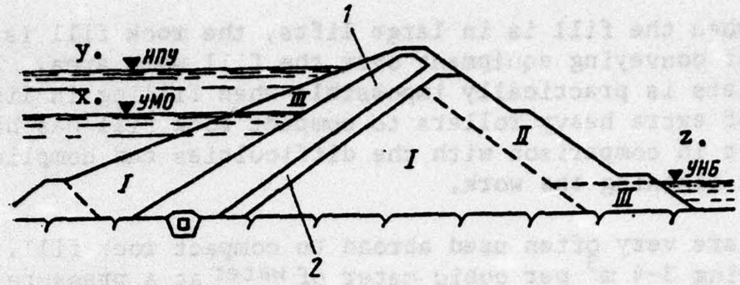

Figure 6-25. Rock placement zones.
1 - Blanket seal; 2 - Filters; I, II, III - Fill zones.
Key: x - LWE; y - NHL; z - LPE

The specifications for zoned fill of rock in the body of a dam muat be carefully observed as far as rock size is concerned: the larger rock should be placed in the downstream wedge of the supporting prism and smaller rock in the upstream zone and the zone below the blanket seal. This zonal distribution is achieved visually by technically qualified personnel supervising the work. In the case of the Vilyuy dam the top fill of the downstream slope consisted of about 70% rock measuring 80 cm or larger.

6 - 11. QUALITY CONTROL OF EARTH PLACEMENT

In building water-impounding structures of natural materials, and especially in building their antiseepage elements, the basic structural materials are usually loams or loamy sand materials. The reliability of structures built of such materials depends on the quality of their placement and above all on the quality of their compaction.

Successful construction depends not only on proper planning of construction operations but also on careful quality control of the work being done. The basic criterion governing the quality of compaction of earth being placed in the body of a dam is its density.

There are a number of methods for monitoring the compaction of earth in the body of an embankment, including the cutting ring method, the "voids" method, the radioisotope method, and other methods, each of which has its advantages and disadvantages.

The cutting ring method has come into wide use in building earth fill dams which are compacted in layers. In spite of its simplicity and ease of execution, it has significant disadvantages, since density data are obtained under laboratory conditions a long time after the sample is taken, and as a consequence operational control of density at the work site is reduced. In addition, the cutting ring method requires that pits be dug in the working area. This hinders the movement of machinery and makes it necessary to fill them later.

The well-known and widely used "voids" method is adequate for determining the bulk density of the soil skeleton [4].

When the bulk density of the earth does not meet technical specifications it is necessary to compact again and in some cases even to remove soil which has been placed and do the job again. Use of this method leads to significant delays and interruptions.

The method of determining bulk density with radioisotope density meters was used with good effect in building the dam for the Vilyuy Hydroelectric Power Station. This method made it possible to obtain needed data on the density of a layer even before it was placed. Frequency curves for the density and moisture content of earth placed on the blanket seal of the Vilyuy dam are presented in Figures 6-26 and 6-27.

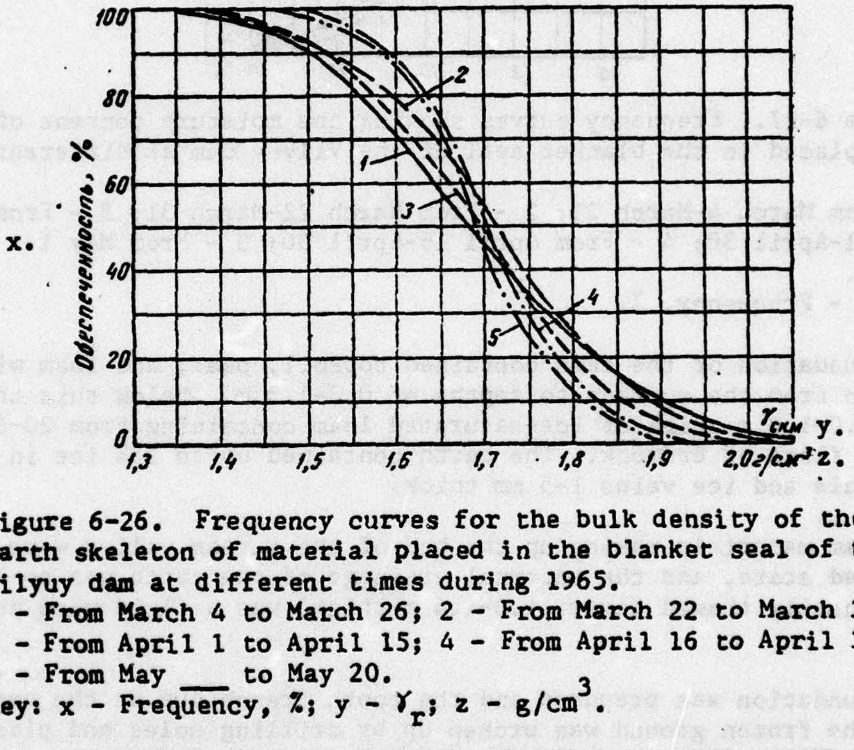

Figure 6-26. Frequency curves for the bulk density of the fine earth skeleton of material placed in the blanket seal of the Vilyuy dam at different times during 1965.
1 - From March 4 to March 26; 2 - From March 22 to March 31;
3 - From April 1 to April 15; 4 - From April 16 to April 30;
5 - From May ___ to May 20.
Key: x - Frequency, %; y - γ_r; z - g/cm^3.

6-12. TECHNIQUES USED IN BUILDING FROZEN DAMS WITH CUTOFF WALLS

We shall describe the construction of frozen dams with cutoff walls using the example of two dams built of frozen ground on a tributary of the Irelyakh River in the winter of 1961-1962 when air temperatures dropped to -40° to - 50°C.

The dams were designed as homogeneous earth dams and their construction under other conditions of engineering geology and climate should not have been difficult. But since the construction site was swampy and inaccessible for machinery during the summer, work could be undertaken only in the winter after the bogs had frozen.

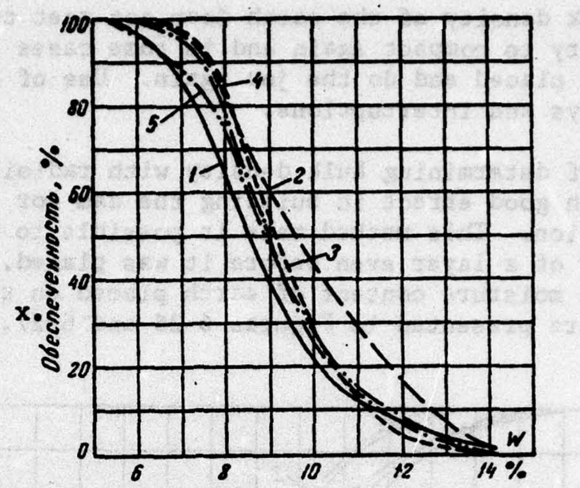

Figure 6-27. Frequency curves showing the moisture content of earth placed in the blanket seal of the Vilyuy dam at different times.
1 - From March 4-March 21; 2 - From March 22-March 31; 3 - From April 1-April 30; 4 - From April 16-April 30; 5 - From May 1-May 20.
Key: x - Frequency, %.

The foundation of the dams contained topsoil, peat, and loam with plant residues from the surface to depths of 0.8-1.3 m. Below this there was a layer 1.0-1.2 m thick of ice-saturated loam containing from 20-55% fragments and flags of bedrock. The earth contained up to 25% ice in the form of crystals and ice veins 1-5 mm thick.

The loam materials making up the bed of the stream valley were in a water-saturated state, and the internal drainage of the earth was so low that the seaonsally thawed layer (1.0-1.4 m thick) was a fluid mass during the summer.

The foundation was prepared and the tooth trench dug at the beginning of winter. The frozen ground was broken up by drilling holes and placing explosives in them. This method was effective only after the seasonally thawed layer had frozen down to the level of the permafrost. Detonating explosives at the beginning of winter in ground that was still unfrozen resulted in the formation of camouflets. Depending on the depth of freezing, the frozen crust here was broken up into large pieces which did not separate or (when large camouflets were formed) the gases simply escaped to the outside from the holes, enlarging the openings somewhat without breaking up the frozen crust at all.

The plan called for the placement of 11,500 m^3 of loam in the upper dam and 3100 m^3 in the lower dam. Thawing these amounts of loam and placing them in the body of the dams in the unfrozen state at air temperatures of -40° to -50°C was an extremely difficult task.

At the same time a set of diversionary structures had to be built before the spring flood in order to protect the mining structures from flooding by the high water, since mining at that time was being done at levels considerably lower than the level of the stream bed.

We proposed, developed, and implemented a method of building the dams out of frozen loams with a wood cutoff wall (see Figure 5-32).

Frozen loam was delivered to a preparation area, spread out, and broken up into pieces measuring 25-30 cm or less by multiple passes of a DET-250 caterpillar tractor. Then the broken loam was moved from the preparation area to the body of the dam, where it was spread in layers 30-40 cm thick over the work area and compacted with MAZ-205 dump trucks. A frozen loam density of 1.23-1.35 tons/m^3 in the body of the dam was produced in this fashion.

The cutoff wall was made of wood panels mounted on piles. Logs were installed in holes bored 2.0 m apart along the bottom of the tooth trench to support this cutoff along the axis of the dam. Double-layered panels (made of 40 and 25 mm boards) covered with two layers of tar paper were made up ahead of time in a warm room and the cutoff wall was assembled from these panels by attaching them vertically in 2-3 rows. The panels were mounted with an overlap of 15 cm.

In order to assure a good seal between the cutoff wall and the foundation, after the first row of panels was installed, the bottom of the tooth was filled with unfrozen loam to a depth of 0.5 m and carefully compacted with manual tampers.

The frozen loam was thawed after placement with area heaters.

6-13. INSTALLATION OF THE FREEZING SYSTEM

Installation of the freezing system in dams with an ice-earth antiseepage curtain consists of the following operations: boring holes and installation of freezing columns and air-distribution headers, fans, and the electricity supply system. The most laborious operations are the boring of the holes and the installation of the freezing columns in them. The holes may be bored and the freezing columns installed either from the crest of the dam after it has been built to the design elevation or before the dam is started. Both methods have their advantages and disadvantages.

In the first case the holes are usually dug with rope-impact equipment, such as the BU-20-2. The boring is done soon after the fill has reached the design level in particular areas of the dam. At the same time most of the boring work has to be done while air temperatures are negative using warm water. This means that some additional amount of heat is necessarily added to the body of the dam, and this, of course, is undersirable in building frozen dams.

When a hole has been dug to the proper depth the exterior pipe of the freezing column is installed in it with a cap on the lower end. Then the space around the pipe is backfilled with a slurry or specially prepared solution. This space should be filled carefully to eliminate all pockets of air.

The design depth of the holes should in every case be adjusted as the work is done in the light of the actual frozen ground conditions. These conditions often change as work progresses and the zero isotherm in the foundation may be significantly below the depth envisioned in the plan. Hence, in digging the holes for the frozen curtain of a dam on the Irelyakh River it was found that the actual size of the unfrozen zone beneath the river bed was significantly larger than indicated by survey data. The enlargement of the unfrozen zone might have been the result of a change in seepage conditions in the unfrozen zone below the river bed during the construction period. The presence of seepage flow during the creation of frozen curtains, of course, creates serious problems for freezing the earth during the period prior to contact between the individual ice-earth cylinders, as was observed, for example, in building the antiseepage frozen curtain in an excavation for the principal structures of the Gorkiy Hydroelectric Power Station [94]. The builders of the dam on the Irelyakh River had to take this experience into account. Therefore additional holes were bored on the downstream slope and an additional freezing system consisting of 22 columns was installed. Since these relatively shallow holes ran mainly in the bedrock, the boring went very fast and this auxiliary system was placed in operation in a relatively short time.

The air distribution headers of freezing systems usually are designed with a single pipe, but in the case of the dam on the Oyuur-Yureye River they embodied a two-pipe design (see Figure 6-28) on the assumption that such a design assures more even distribution of the coolant. In addition such a system is easier to protect in the summer.

But, as operating experience showed, the two-pipe system of air-distribution headers has a number of significant disadvantages [22]. It often becomes necessary during operation to remove the inside pipe of a freezing column for repair. In the two-pipe header system this is impossible without shutting down all the freezing columns. It is also more difficult to clear the space between the pipes when ice forms from condensation on the surface of the pipes. In addition, with the two-pipe system it is more difficult to monitor the operation of each individual freezing column. With the single-pipe system, however, it is possible to monitor the operation of each of the columns visually, and there is a signal flag on each column for this purpose.

Another advantage of the single-pipe system of headers is that this system makes it possible to monitor the progress of core freezing. If for some reason it is not possible to obtain direct information of the progress of freezing, an indirect method can be used. In the latter case observations are made on the amount of cold introduced into the core by measuring the amount of air forced into each column and the air temperature as it enters and leaves. This indirect method of observation on the progress of heat extraction from the core of the dam gives a basic idea of changes in temperature conditions within the body of the dam based on the heat balance, taking into account the moisture content of the earth.

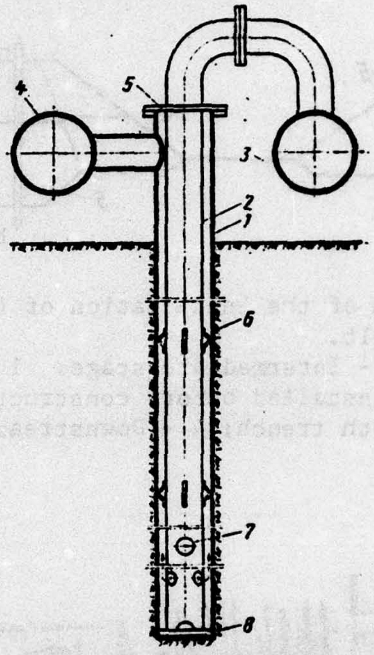

Figure 6-28. Freezing column of the dam on the Oyuur-Yurege River with a two-pipe air-distribution header system.
1 - Outside pipe; 2 - Inside pipe; 3 - Pressurized header;
4 - Exhaust header; 5 - Seal; 6 - Spacers; 7 - Openings;
8 - Cap.

In the one-pipe system the air-distribution header is a box channel made of light sheet metal, which is mounted on brackets attached to the pipes of the freezing columns. In the two-pipe system welded pipes usually 325 mm in diameter are mounted on blocks.

We suggested a different method for installing the cooling columns. In this method the outside pipes of the columns are installed before the dam is built, by boring holes either before or after the foundation is prepared (see Figure 6-29). It is also possible to bore from the bottom of the tooth trench before the core is built (see Figure 6-30).

When holes are bored from the crest of a dam to depth H after the dam has been raised to the design elevation according to the existing method, the spacing of the holes becomes irregular at the bottom because of unavoida deflection of the holes at deep levels. This interferes with smooth merging of the individual ice-earth cylinders developing around each freezing column into a single frozen curtain. "Windows" of unfrozen ground may therefore remain in the frozen curtain. Using the method suggested, the holes are bored and the freezing columns installed from the bottom of the tooth trench or rough elevations. In this case the boring depth is equal to H_1. This eliminates the need to bore a distance equal to H_2 ($H_2 = H - H_1$). The core and the supporting prisms of the dam body are raised after the freezing columns have already been fixed in their planned positions. The columns are extended as the core rises (see Figure 6-29).

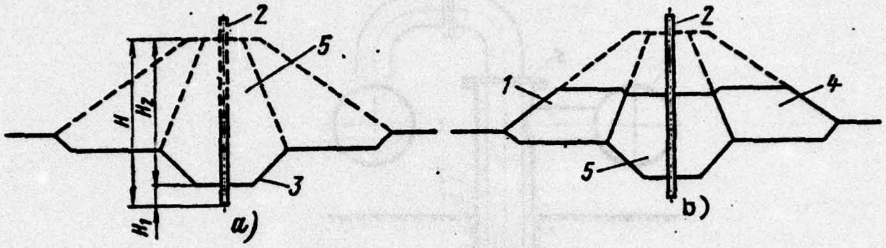

Figure 6-29. Diagram of the installation of freezing columns before the dam is built.
a - Initial stage; b - Intermediate stage. 1 - Upstream wedge; 2 - Freezing column installed before construction of the core begins; 3 - Bottom of the tooth trench; 4 - Downstream prism; 5 - Core.

Figure 6-30. Installing freezing column before building the tooth of the core.

Installing the freezing columns by this method has the following advantages: the amount of boring work done is reduced 80-90%; there is no need to use drive pipes nor go to the trouble of removing them later; the effort of backfilling the space around the pipe is eliminated; the quality of installation of the freezing columns is improved and the freezing system is placed in operation faster, since it can be started up immediately after the dam has been filled to the designed elevation; that is to say creation of the frozen antiseepage curtain in the body of the dam can begin immediately.

When this method of installing the freezing system is used, however, compaction of the earth around the columns and between them is made more difficult.

There is usually no problem with installing the fans and the electrical supply system. When snowfall is not excessive and blowing snow does not continue for long periods during the winter, the fans are mounted in the open air.

6-14. DISPOSAL OF WATER DURING CONSTRUCTION OF DAMS

During the construction of earth dams water may be carried away in one of the usual methods: along the river bed through a cut confined by parts of the dam near the bank (the Vilyuy Hydroelectric Power Station, the dams on the Irelyakh, the Oyuur-Yurege, Sytykan, and other rivers), through a drainage channel or tunnel (Vilyuy, Serebryanskaya, and Khantayka Hydroelectric Power Stations, the dams on the N'yukka Creek, etc.), and through gutters and pipes (the dams on the Irelyakh and Pavek Rivers, N'yukka Creek, etc.). At the same time, depending on the stage of construction, water may be disposed of in several ways as the same element is being built, for example, through a cut in the first stage, through a channel or pipe in the second stage, etc.

The use of any particular method of diversion depends on the size of the flow, the hydrological regime of the river, the geology and topography of the construction site, and the general plan and timing of the construction. At the same time the organization of the work and the method of construction are often determined on the basis of the hydrological regime of the river and the plan adopted for disposing of water during construction. Therefore all factors must be carefully taken into consideration in determining the method of unwatering.

As has already been mentioned, the hydrological regime of rivers in the permafrost zone is unusual: up to 85-90% of the annual flow occurs during the spring flood; discharge is typically low during the summer and fall, and is minimum in the winter.

Under particular geological, hydrological, and frozen-ground conditions unfrozen dams can be built without previous unwatering of the trench by filling directly into flowing water or by using hydraulic fill methods.

But the basic method of building dams in permafrost areas is still that of building them in the dry. In this case construction of at least the channel portion of the dam is done on a dry unwatered foundation area, by protecting the foundation site or accumulating the water in the upper pool.

Under ordinary conditions the method of accumulating the water in the upper pool has not been commonly used, but in the Far North, with the characteristic hydrological regime of the rivers there, this method is more widely used (at the Vilyuy Hydroelectric Power Station after filling of the construction trench, at the dam on the Sytykan River, and on the dam on N'yukka Creek).

Among the favorable factors for using this method we might mention low river discharge, the topographic conditions of the future reservoir, the presence of natural depressions in which the river flow can accumulate during construction of the dam at low elevations in the channel area, and the need for a relatively small amount of work to raise the dam within the channel cut zone, work which can be completed in a short time. The work should be planned in such a way that the dam rises ahead of the water level in the reservoir.

In all cases it appears necessary to take into account flood discharges. Only a correct approach to the problem of diversion can assure normal construction conditions.

For planning purposes the discharge during the construction period is assumed to be the maximum spring flood discharge as well as the rest of the summer flood, which may be quite high when the so-called "black water" coincides with the summer rain flood. The design discharge is assumed to occur with a frequency of 5-20%, depending on the capital class of the structure. The steps taken to dispose of discharge levels occurring with a frequency of less than 5% involves greater investment, while the assumption of calculated discharge levels with greater than 10% frequency involves definite risk with respect to disposing of the discharge without damage.

In building unfrozen dams, particularly when building them on a rock foundation, unwatering by diverting the river through a bypass channel or tunnel is most convenient from the viewpoint of construction operations. In this case, since the foundation site is accessible everywhere, construction work can continue simultaneously over the entire front, and this greatly simplifies the construction effort.

This method is not always feasible in building on permafrost which is not rock, especially when the dam is built so as to keep the foundation material in the frozen state. The reason for this is that when the water is passed through a bypass channel or tunnel, thawing of the foundation is unavoidable, with all the consequences that flow therefrom.

When topographic conditions of the dam valley permit, diversion is usually accomplished through the use of a channel-cut confined at the bank by dam fill (Irelyakh, Oyuur-Yurege, and Sytykan Rivers). In this case dam construction within the area of such a cut is possible only after it is closed with cofferdams and unwatered either by gutters or by pumping into the lower pool. This work is usually planned on the calendar for the period of minimum discharge.

When the flow is 10-15 m^3/sec, upstream and downstream cofferdams are built in the river channel, and the water held back by the upstream cofferdam is carried off through a trench or pipe into the lower pool across the construction area.

In building the dam for a temporary reservoir on the Irelyakh River (See Figure 5-1), which was built by the unfrozen method, it was necessary to handle two spring and summer floods. The spring flood of 1958, which had a 10% frequency level of 110 m^3/sec, was handled by a natural channel 35 mm wide. In order to prevent flooding of the by-wash excavation, during passage of the flood a barrier was installed at the intake area of the underwater channel, which served as a natural cofferdam.

The river channel was closed at the end of August 1958 and the reservoir began to fill. Since the permanent intake structure had not been completed, a temporary trough 6.0 m wide was built to pass the fall flood through the uncompleted by-wash.

During construction of the dam for a permanent reservoir on the Irelyakh River, the discharge during the first year of construction was passed through the natural channels. During the spring flood loam was placed in the tooth of the core on the left bank floodplain, the site having been prepared during the winter.

In order to prevent flooding of the tooth trench, the bottom of which was 6 m below the bottom of the river bed, cofferdams were built along the river channel and loam was placed in the tooth behind these cofferdams during the spring and summer. The foundation beneath the upstream wedge, which was also prepared during the winter, was protected from flooding by an embankment.

The spring and fall floods during the second construction year were passed through an opening after the dam had been raised to its full design elevation on both sides. After the fall flood during the second construction year was over, the river bed was closed with upstream and downstream cofferdams and the water present was pumped into the lower pool through a conduit. The channel portion of the dam within the opening was constructed on the unwatered foundation site after sediments had been stripped away.

In building the dam for the second-stage tailing pond on N'yukka Creek (see Figure 6-31), a trench was dug on the left-bank floodplain directly at the foot of the valley slope, and the flow was diverted after the stream bed was closed. This made it possible to strip the foundation and remove unsuitable materials in the dry. Subsequently, after this trench was closed, the creek flow was bypassed through two pipes 900 mm in diameter running beneath the portion of the dam which had been previously constructed on the left-bank floodplain. The creek flow had remained constant practically throughout the entire construction period, since it received the clarified water from the ore-dressing plant which was operating above the tailing pond.

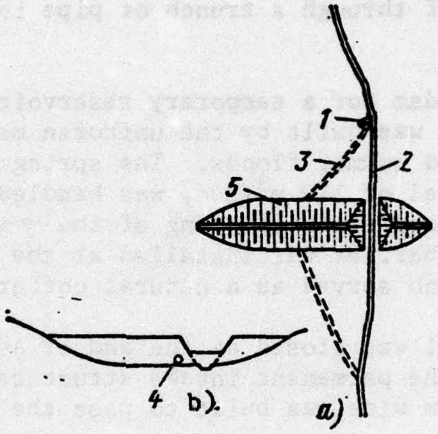

Figure 6-31. Diagram of the method used to handle during construction of the dam on N'yukka Creek.
a - Water passing through the cleft; b - Water passed through a conduit. 1 - Cofferdam; 2 - Construction trench; 3 - Creek bed; 4 - Conduit to handle discharge during the second stage; 5 - First stage of the dam.

After the dam has been built up in the area of the unwatering trench to the design elevation, the ends of the pipes were closed off and the inside cavity was filled with a slurry. During this period the creek flow was accumulated above the upstream cofferdam. When the water level had risen to the crest of the cofferdam, head pressure was applied to the tailing pond dam.

In building the dam on the Oyuur-Yurege and Sytykan Rivers, before the spring flood in the first year of construction the tooth trench was dug to its full extent and loam was placed in the tooth to the level of the river bottom, i.e. to a height of 3-4 m in the channel area. The waters of the spring and fall flood during the first year of construction were passed through the natural stream bed. During the spring flood the loam placed in the tooth was partially flooded. When the flood waters receded, work began from both banks on placement of more loam in the tooth and core. Fill work on the lateral prisms of the dam was not interrupted even during the spring flood period.

AD-A035 905   COLD REGIONS RESEARCH AND ENGINEERING LAB HANOVER N H   F/G 13/13
DAMS ON PERMAFROST (PLOTINY NA VECHNOY MERZLOTE),(U)
NOV 76 G F BIYANOV
UNCLASSIFIED   CRREL-TL-555   NL

3 OF 3
AD
A035905

END
DATE
FILMED
3-77

The openings in the dams on the Oyuur-Yurege (Figure 6-32) and Sytykan Rivers were closed after the fall floods in 1971 and 1972, respectively.

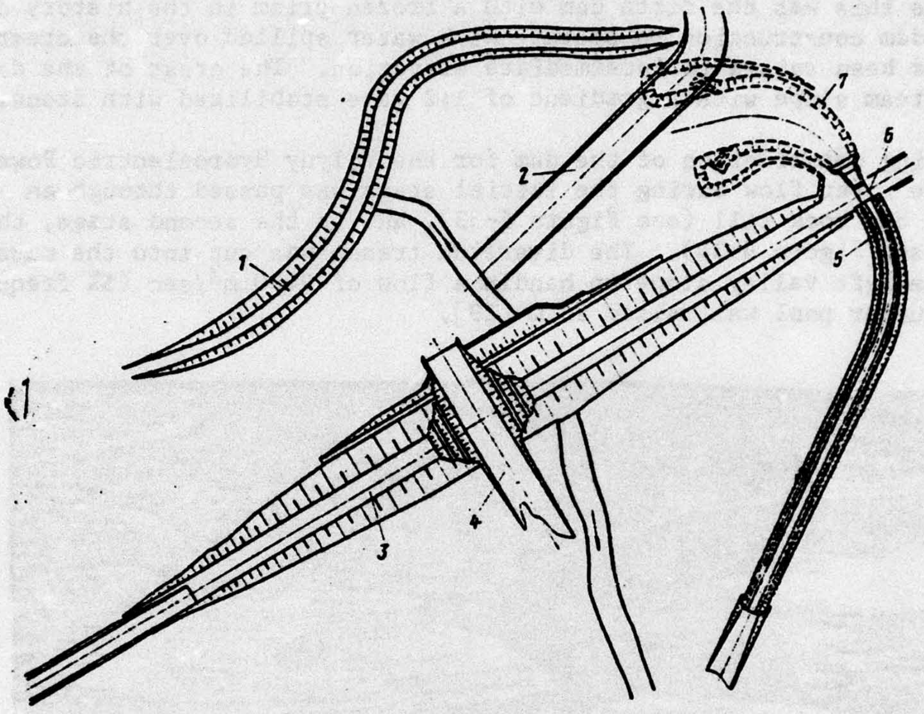

Figure 6-32. Diagram of unwatering of the construction site for the dam on the Oyuur-Yurege River.
1 - Temporary dam; 2 - Temporary channel; 3 - Dam fill; 4 - Opening to pass water during construction; 5 - Sluiceway channel; 6 - sluiceway; 7 - Flow-controlling dike.

It should be noted that, prior to the construction of both of these dams, temporary dams were built 0.5 km above them to be used for 2-3 years.

After the river was closed off and before the advent of heavy frost, the channel portion of the dam on the Sytykan River was raised to an intermediate elevation of 14 m and installation of the freezing system began. After freezing, the dam was placed under pressure in the spring of 1973 to create a reservoir with a volume of about 5,000,000 m^3. At this time excess flood waters were disposed of by spilling across the crest of the dam, since the level of the floor of the permanent spillway channel was 3 m above the crest of the dam (at the intermediate height).

After the spring flood of the second construction year had been taken care of, the dam was successfully operated for an entire year with a water head of about 10 m. The spring flood in the third construction year was passed through the permanent spillway channel, since the dam had been raised to its full design elevation and frozen.

Thus this was the fifth dam with a frozen prism in the history of practical dam construction to operate with water spilled over the crest when it has been raised to intermediate elevation. The crest of the dam and the downstream slope with a gradient of 1:2 were stabilized with stone.

During construction of the dam for the Vilyuy Hydroelectric Power Station the river flow during the initial stage was passed through an opening in the rock fill (see Figure 6-33), and in the second stage, through a trench (see Figure 6-34). The diversion trench was cut into the massive rock on the left valley slope to handle a flow of 9400 m^3/sec (5% frequency) where the upper pool was raised 25 m [19].

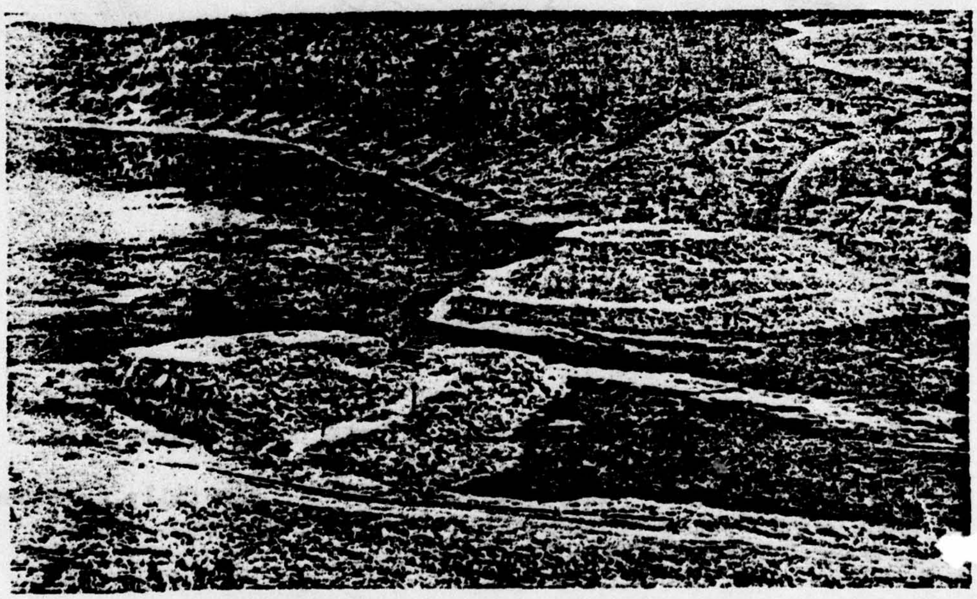

Figure 6-33. Water passing through an opening in the dam during the construction of the Vilyuy Hydroelectric Power Station in 1964.

The trench diversion system chosen cannot be considered successful for the Vilyuy conditions, since it was necessary to work on the dam in the trench area only during the winter and very fast. It would have been best to pass the high flood discharges through the main river channel or an opening in the dam (possibly also by spilling the water over the crest of the still-uncompleted dam), and to pass the summer low-water flow and the winter flow through water escape structures (a tunnel, canal, ditch, or conduit).

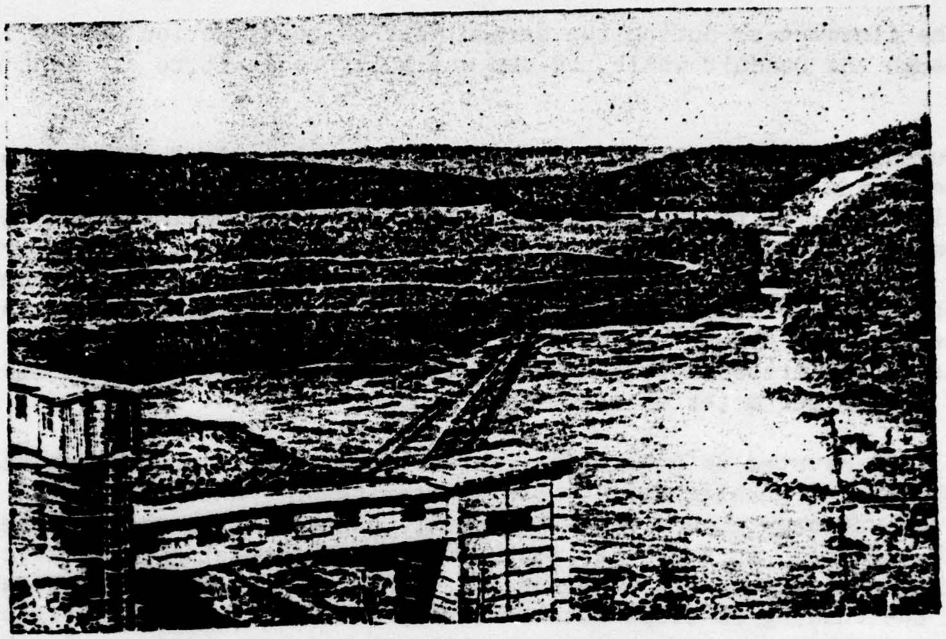

Figure 6-34. Bypassing of water through a trench during construction of the Vilyuy Hydroelectric Power Station. Concrete plant in the foreground.

Worth attention in this respect is the unwatering experience gained during construction of the Serebryanskaya and Khantayka Hydroelectric Power Stations. During construction of the Serebryanskaya Hydroelectric Power Station, discharges of 1120 m^3/sec were passed through a channel and conduit system (see Figure 6-35) where the head water rose 11-12 m at the entrance. The channel was not cut deep into the slope. Although this required building a reinforced concrete wall on the side toward the dam, the amount of work required to dig the channel and place earth in the body of the dam in this area was not great.

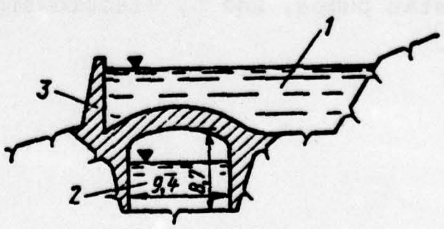

Figure 6-35. Unwatering of the construction site using a channel-conduit system during building of the Serebryanskaya Hydroelectric Power Station.
1 - Channel; 2 - Conduit; 3 - Reinforced concrete wall.

The floodwaters during the second year of construction were passed only through the conduit while the dam was built above it to a height of 35 m.

During construction of the Khantayka Hydroelectric Power Station a tunnel measuring 10x10 m in cross-section was constructed to handle fall-winter discharges up to 500 m^3/sec. Spring floods during the second construction year amounting to 6700 m^3/sec with a 5% frequency spilled over the crest of the dam, which had been built to full profile up to an intermediate elevation. In order to prevent erosion, the downstream slope was stabilized by shuttering with a concrete plate. Concrete was also placed partly in the cells of the cribwork (see Figure 5-15). The maximum specific discharge was 66.7 m^3/sec per meter of dam width [77].

Effort devoted to unwatering is relatively small in building dams on permafrost. A problem arises with unwatering only when working in the area of the unfrozen ground beneath a river channel, where ground water is typically present. At the same time, however, when working in excavations for the tooth of a core or blanket seal, if necessary steps are not taken to limit the thawing of slope materials with a high ice content, serious difficulties may be encountered which will make it impossible to do the job properly. In these cases one of the most important tasks is to keep the site dry, and successful building of the structure will depend largely on proper planning and functioning of the diversion facilities. Initial pumping of the construction site after the building of cofferdams and the removal of water which has seeped through the cofferdams and their foundations are accomplished with an open unwatering system involving the digging of trenches and drainage ditches. The sumps and piping must be heated in winter.

When the foundation sites to be protected are large in area and water-bearing materials are present in the foundation of the cofferdam and along the edges of the excavation with a water table close to the surface, an open unwatering system may be inadequate or may not be capable of fully drying the foundation materials. In such cases it may become necessary to remove water from below the surface with filter rods, less often with pipe wells equipped with artesian pumps, and by electroosmosis in relatively impermeable materials [7].

Chapter 7

WATER-ESCAPE STRUCTURES

7-1. LOCATION AND TYPES OF WATER-ESCAPE STRUCTURES

A hydraulic engineering installation containing dams made of local materials usually includes water-escape and water-intake structures of various designs for various purposes. Spillways usually are built at the surface, since they are used to release not only water but also ice from the reservoirs.

When dams are built in the permafrost zone, one usually avoids locating the water-escape and water-intake structures in the body of the dam, and they are ordinarily placed at one of the valley slopes. The reason for this is that dams made of natural materials are subject to considerable settlement during use, so that local deformations are possible both in the water-escape structures and in the dams themselves, so that the resistance to seepage and overall integrity of the dam are damaged.

Water-intake structures with pumping stations are placed alongside the dam to permit use of the water in the reservoir for economic purposes.

In some cases outlets are installed below the dam, in the form of metal or reinforced concrete pipes, to take water from the reservoir. Such outlets are equipped with a gate at the downstream end. Locating water outlets and intakes on the bottom makes it possible to utilize more fully the water resources of a reservoir. But the flow which can be accomodated by such outlets is usually not large and this function can be equally well met, in our opinion, by building siphons or pumping stations.

We are firmly convinced, moreover, that when building dams on a permanently frozen foundation, water-escape structures must not be built in the body of the dam. As far as the economy of such designs is concerned, it can hardly be compared with the reduced reliability of the dams themselves, even though to avoid uneven settlement of dams, efforts are made to install water outlets below the dams.

Nevertheless there are frequent cases of dam failure caused to some extent by the installation of water outlets in the foundation or body of dams. For this reason, the author feels that the recommendations of VSN-08-65 p. 24, which permit construction of bottom outlets under dams, should be considered erroneous.

Ordinarily open water-escape structures are built along the bank around the dam as self-regulating spillways or sluiceways with control gates at the crest and other well-known equipment to control the hydraulic relationships of the upper and lower pools and to absorb the energy of the flow. Automatic self-regulating water-escape structures with their crest at the elevation of the normal headwater level (NHL) of the reservoir are preferable from the functional viewpoint. But structures of this type mean expanding considerably the width of the water-escape facility, and this cannot always be accomplished because of topographic conditions.

When it is difficult to install the water-escape structure in frontal alignment with the dam, by-wash trenches are built which carry the water off to the side. Designers adopt this solution when building dams in narrow valleys with steep sides. In order to keep down the cost, the direction of spillway discharge of water-escape structures is usually positioned at right angles to the axis of the dam or aproximately so (see Figure 7-1). The water passes over the weir into a narrow canal, changes its direction almost 90° and then runs along the tail race.

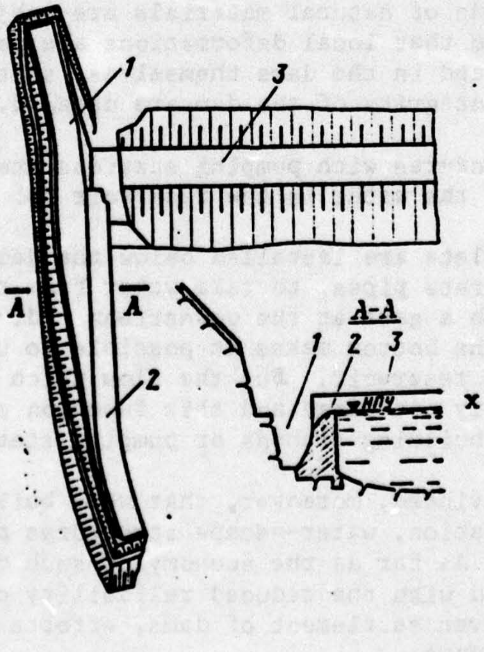

Figure 7-1. Trench-type water escape at the installation on the Myaundzha River.
1 - Escape trench; 2 - Concrete weir; 3 - Crest of dam. Key: x - NHL

The special characteristic of water-escape structures is that by nature they function intermittently, disposing of floodwaters during a short time period. At the same time a considerable amount of work is required to build them (see Table 7-1) and their cost is close to the cost of building the dam (see Table 7-2). For these reasons stringent specifications are applied to water-escape structures with respect to their design, workmanship, and temperature regime when built on permafrost.

Table 7-1

Work Involved in Building Water-Escape Structures

Operation	Volume, thousands of m^3		
	Irelyakh	Oyuur-Yurege	Sytykan
Excavation	197.0	38.0	254.0
Fill of noncohesive material	6.0	17.0	12.6
Placement of loam	1.0	--	14.0
Preparation with sand	4.5	--	2.7
Preparation with crushed rock	7.5	1.0	6.2
Facing channel with reinforced concrete plates	8.0	3.4	10.65
Concrete piers and piles of icebreakers	0.56	--	1.31

Table 7-2

Relative Cost of Dams and Water-Escape Structures

Installation	Relative cost, % of total construction cost		Ratio of relative cost of dam to relative cost of water escape
	dam	water escape	
Vilyuy	27.8	18.2	1.53
Khantayka	32.0	32.0	1.00
Kolyma	24.0	24.0	1.00
On Irelyakh River	52.5	46.5	1.13
On Oyuur-Yurege River	65.0	24.0	2.70
On Sutykan River	41.0	56.0	0.73

When selecting the alignment and structural components of an installation, one must give careful attention to the conditions under which the water-escape structures will be built, taking into account the use of simple designs which will permit the maximum possible degree of mechanization. The best alternative is adopted after studying and comparing a number of design, engineering, and operational alternatives.

As has already been mentioned, water-escape structures are built as regulated and open self-regulating channels. Gates are built at the head of regulated sluiceways. Lateral trench type water escapes, siphon-type escape-intakes, and bottom intake-outlets with gates located in a tower may also be used.

A water escape with an open self-regulating channel consists of an intake channel, a header, a discharge channel with associated structures, and a discharge canal. The discharge portion of the water-escape facility is linked to the lower pool via multiple cascades or a tail race, the construction of breakwater wells, spreaders, overshoots, etc.

The intake channel is usually built with a wide cross-section so that the approach speed will be slow and the bottom and sides will not require facing. When weak ground materials are involved, the tail race is usually replaced by a multiple cascade. The head structure of the water escape is usually made of concrete, and in controlled sluiceways it is equipped with working gates which can be repaired or replaced. In order to reduce the amount of excavation required, the tail race is curved in plan view and banked. Velocities in the tail race may reach 15-25 m/sec, so that it should be faced very carefully.

In multiple cascades energy is absorbed at every step. The dimensions of the multiple cascade are determined by hydraulic design calculations and tested with a model in the laboratory.

Siphon water escapes, being automatic in operation, pass water well and have good operating characteristics which permit free regulation of the discharge. Because they act automatically, the operation of the siphons is simple, although for normal operation during the winter it is necessary to warm them throughout their length.

Siphon devices made of metal pipe were used to carry away clarified water from the tailing ponds built on Novyy and N'yukka Creeks.

Siphon-type intake and discharge installations are desirable when building dams on permafrost in the Far North, because during operation they have less effect on the thermal regime of the dam and its foundation than any other type of water-escape structure.

7-2. EXAMPLES OF WATER-ESCAPE STRUCTURES

1. The water-escape structure of the temporary reservoir on the Irelyakh River was located where the dam joins the right bank and consists of intake and discharge channels and the sluiceway itself. Both channels are of practically the same length and were dug into the bed rock of the right valley slope. Figure 7-2 shows a longitudinal section through the axis of the sluiceway area.

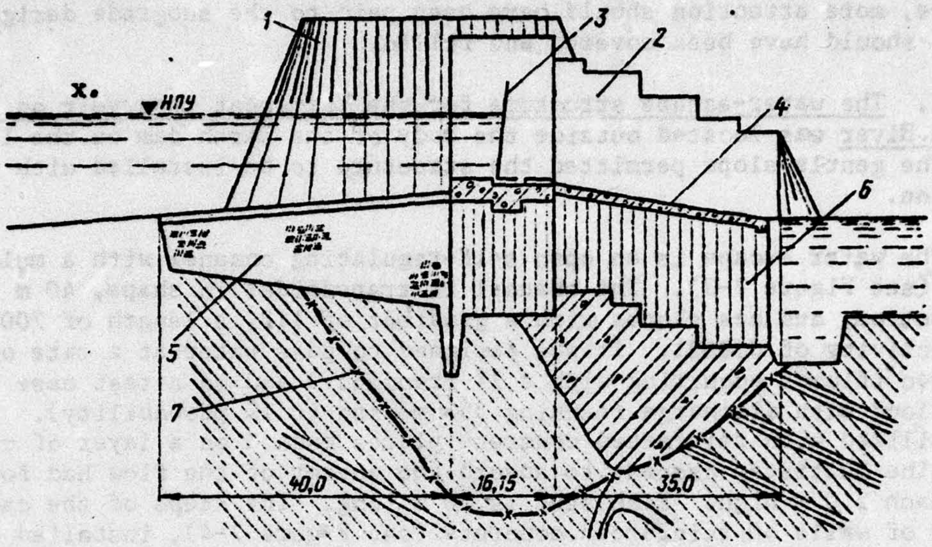

Figure 7-2. Sluiceway of the dam for a temporary reservoir on the Irelyakh River.
1 - Dam; 2 - Cribwork header; 3 - Gates; 4 - Concrete plate of the floor; 5 - Apron; 6 - Cribwork foundation; 7 - Location of zero isotherm under the sluiceway.
Key: x - NHL.

The water-escape structure was originally planned to be self-regulating. The threshold at the NHL level of the reservoir has a cribwork header 24 m wide and 7.5 m long feeding into the wooden sluice of the tail race. The tail race was planned as a frame structure 100 m long on a pile foundation. An ice-retaining structure was installed in the reservoir ahead of the water escape consisting of clusters of seven piles, each 30 cm in diameter.

A few days after the water in the reservoir had reached the design elevation, seepage appeared in the foundation of the water escape as a result of design deficiencies in the junction between the cribwork header of the water escape and the foundation. When the first spring flood occurred (1959), the water escape was practically entirely destroyed (see Paragraph 8-4).

Repairs consisted of planning and installing a reinforced concrete plate on the floor of the structure. An apparatus to raise the water level was built on the concrete plate consisting of girders, poles, and panels, which made it possible to raise the normal headwater level by 2.15 m. In addition, an apron of loam faced with reinforced concrete plates was installed in front of the header in the intake channel.

This water-escape structure was planned using designs employed under ordinary conditions. Since permafrost was present in the foundation of the structure, more attention should have been paid to the subgrade design; fissures should have been covered and filled.

2. The water-escape structure for the permanent reservoir on the Irelyakh River was located outside the body of the earth dam on the left bank. The gentle slope permitted the structure to be installed with minimum excavation.

The water escape is an open self-regulating channel with a multiple cascade (see Figure 7-3). The channel is trapezoidal in shape, 40 m wide at the bottom, and has slopes with a gradient of 1:2, a length of 700 m, and a declivity of 0.0021. It was designed to pass water at a rate of 166 m^3/sec (floods occurring with a 2% frequency) and in a test case to pass a flood with discharge reaching 356 m^3/sec (0.1% probability). It was stabilized with reinforced concrete plates bedded on a layer of crushed stone. The multistep cascade to absorb the energy of the flow had four steps, each 3.0 m high. Each step is 30 m long. The steps of the cascade are made of walls of reinforced concrete (see Figure 7-4), installed on a grid of reinforced concrete piles sunk 5.0 m into the frozen ground (see Figure 7-5).

Pockets were provided in front of the walls on the upstream side. These were filled with crushed stone, and perforated pipes of asbestos cement were installed in them. Drain pipes were also installed in the concrete walls to drain water from the pockets and permeable material was placed below the walls of the steps. The bottom and slopes of the channel are lined for a distance of 10 m below the walls with heavy reinforced concrete plates 1.0 m thick, which serve as aprons.

The plan did not provide for the passage of ice through the spillway. It was assumed that an ice cover would form on the reservoir, and only individual pieces of ice torn away from the ice cover could reach the spillway. An ice-retaining wall consisting of six massive reinforced concrete piers built on pilings was constructed to hold back the ice floes at the entrance to the water-escape channel. The piles, with a 35x30 cm cross-section, were sunk 8 m into the frozen ground. An automobile road was run across the piers. A longitudinal frozen curtain of freezing columns similar to that used in the dam was built along the right bank of the channel, i.e. on the dam side.

A frozen curtain was installed beneath the water-escape channel in line with the axis of the dam. It was produced by five metal pipes running horizontally in the loam tooth. Cold air was circulated through these pipes in the winter.

A deficiency of this water-escape structure is the fact that the frozen curtain was built only in the intake region (along the axis of the dam), while the design allowed the foundation to thaw along the rest of the channel. The channel was designed with filters and drains but means were not provided to prevent seepage of warm water into the foundation materials.

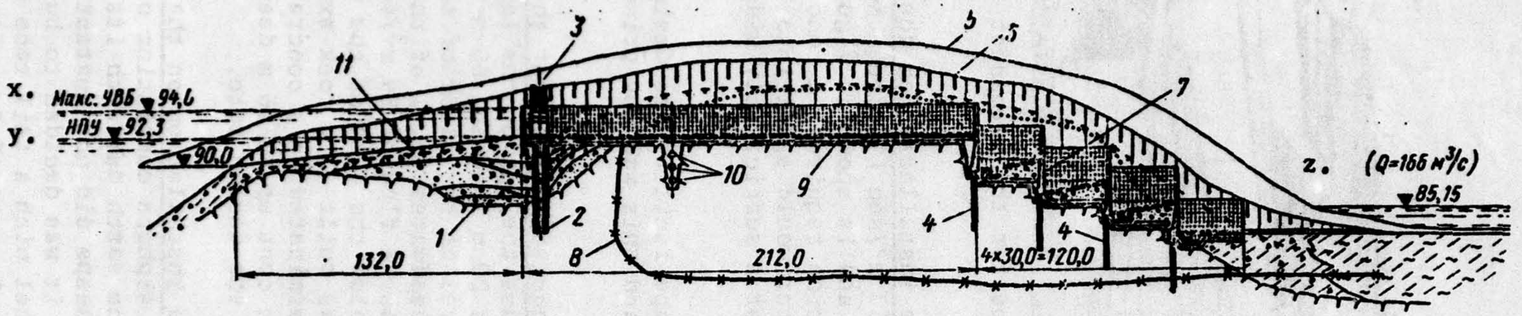

Figure 7-3. Water escape structure of the dam for a permanent reservoir on the Irelyakh River.
1 - Surface of bedrock; 2 - Piles; 3 - Automobile bridge; 4 - Piles; 5 - Natural ground level along the left side of the channel; 6 - The same, along the right side of the channel; 7 - Cascade walls; 8 - Zero isotherm; 9 - Concrete facing of the bottom and sides of the channel; 10 - Freezing system for the channel; 11 - Diluvial loam.
Key: c - Maximum level of upper pool; y - NHL; z - (Q = 166 m^3/sec).

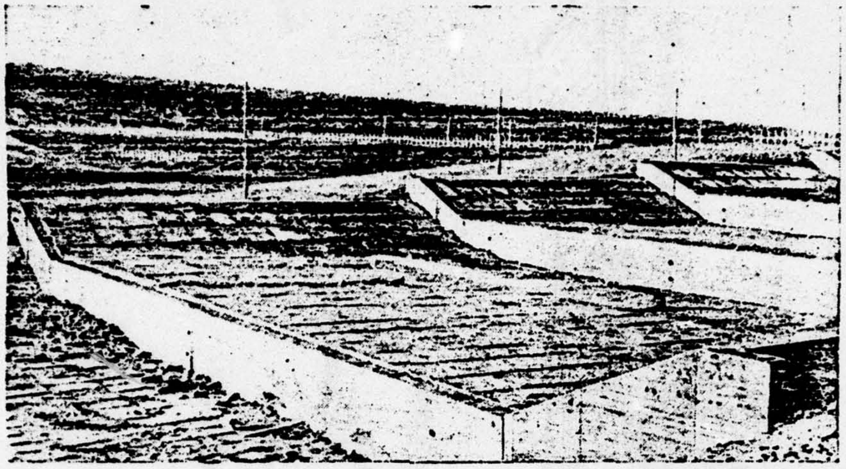

Figure 7-4. Permanent dam water escape for the permanent reservoir on the Irelyakh River.

3. <u>The water-escape structure for the installation on the Myaundzha River</u> was planned to handle a discharge of 338 m^3/sec (1% frequency). It was located on the left bank in massive rock and is shown in Figure. 7-1. In order to reduce the amount of rock excavation required, a concrete weir was run parallel to the slope so that the water would spill into a trench faced with concrete. This design provided for a sufficiently wide spillway with minimum rock excavation.

In contrast to the dam, the water-escape facility was designed to allow thawing of the foundation during use, and this was the principal design defect.

4. <u>The water-escape structure with concrete header for the installation on Ponneuregen Creek</u> was built to bypass the dam on the left slope of the valley. The intake channel, which was 20 m long and 60 m wide was built with a reverse slope of 0.005. The level of the floor of the water-escape structure was 3.6 m below the normal headwater level of the reservoir. The water-escape structure was designed to pass a flow of 89 m^3/sec (0.5 % frequency). The spillway width was 15 m. Below the head of the water escape a tail race 8 m wide and 220 m long was built in a rock excavation and faced with concrete. The water escape terminated in a concrete overhang with a reverse slope of 15°. An erosion cone up to 16 m deep is formed beyond the overhang when water is flowing at the design rate.

5. <u>The water-escape structure for the installation on the Oyuur-Yurege River</u> (Figure 7-6) was originally planned as siphons consisting of five pipes 1.0 m in diameter installed in the body of the earth dam with its frozen curtain. But since this design for the water-escape did not guarantee that the ice-earth frozen curtain would be preserved, it was decided to build the water escape as an open self-regulating channel with a tail race on the left bank. The discharge channel, which was 410 m long and 7.0 m wide at the bottom was built in rock (diabase) and semiconsolidated sedimentary materials with sloping sides (1:1.5) and a bottom slope of 0.025.

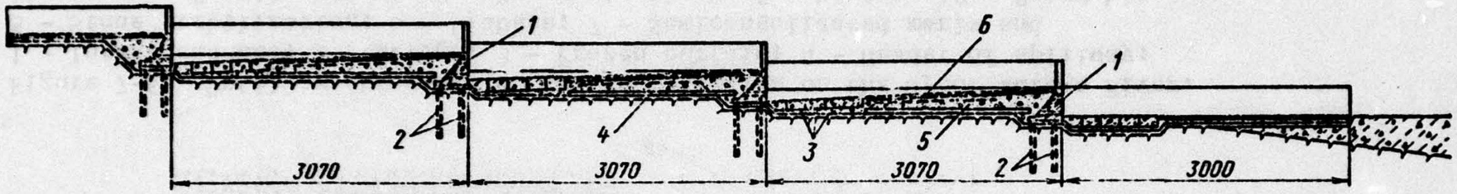

Figure 7-5. Cascade walls of the water escape of the dam for a permanent reservoir on the Irelyakh River.
1 - Concrete wall; 2 - Piles; 3 - Massive concrete; 4 - Drain pipes; 5 - Ground drained; 6 - Reinforced concrete plates.

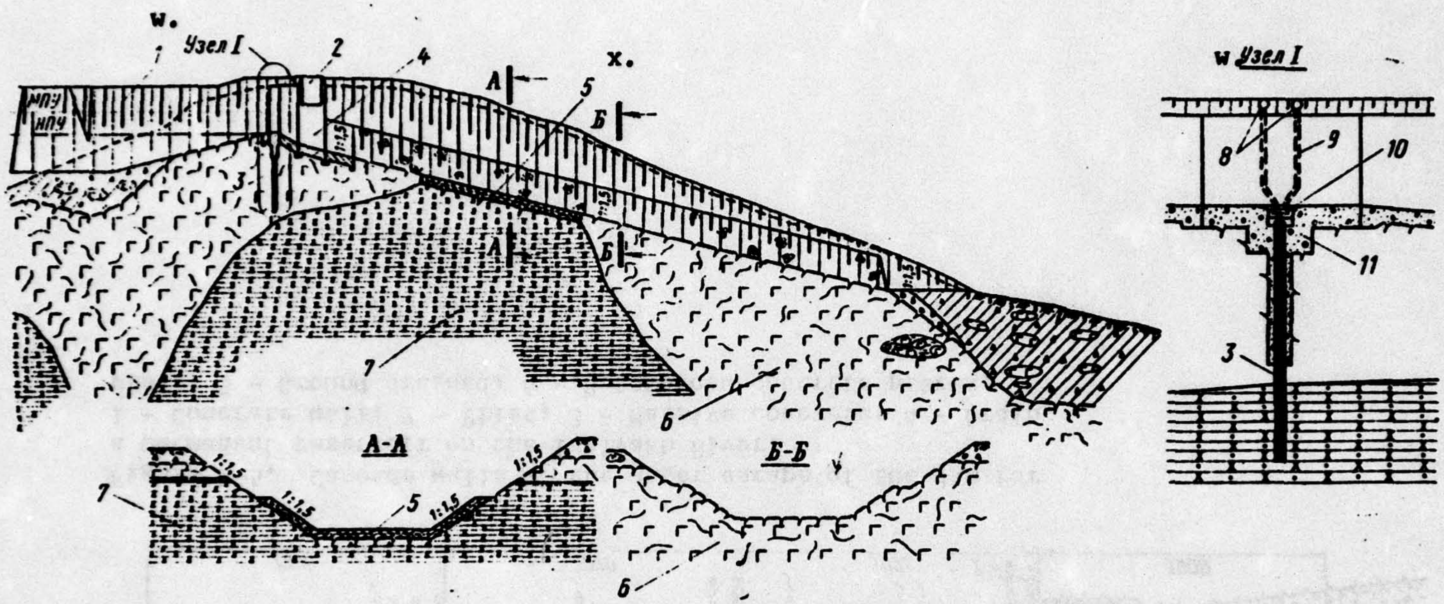

Figure 7-6. Spillway structure of the installation on the Oyuur-Yurege River.
1 - Intake channel; 2 - Bridge; 3 - Frozen curtain; 4 - Header of spillway; 5 - Stone stabilization; 6 - Diabase; 7 - Semiconsolidated marls and limestones; 8 - Air header; 9 - Removable riser of a header; 10 - Removable cover; 11 - Concrete plate with cutoff wall.
Key: w - Detail I; x - B-B.

The bottom of the channel is horizontal for the last 50 m. The sides and bottom of the channel were stabilized with riprap in the sedimentary rock area. The concrete tail race was built as a rectangular trough. A concrete cutoff tooth 5 m deep was installed below the floor of the concrete header and was cut like a spur into the massive rock on the left bank. The header of the water-escape structure was built so as to keep the foundation in the frozen state.

A freezing system was installed along the axis of the tooth. It is a continuation of the dam freezing system. The freezing columns run through the concrete cutoff wall and are buried 3 m into the rock below. The curtain extends a distance of 8 m below the floor. This system extends into the left rock bank of the water-escape structure 7.5 m from the edge of the header.

The pressurized and exhaust air headers run at the level of the dam crest. The pipes of the freezing columns are connected to these headers by removable riser pipes. These are taken off when a flood is passing through the water-escape structure. The heads of the freezing column pipes are kept during the summer in recesses built into the concrete floor of the spillway.

The structure was designed to pass a flow of 35 m^3/sec with a 1.4 m head of water at the floor. Ice was not expected to pass through the spillway. Ice is held in the reservoir by the directional dikes of the intake channel. These run at an angle of 90^o to the axis of the spillway structure.

The freezing system below the floor of the spillway was designed for this dam after taking into account the design deficiencies associated with the dam on the Irelyakh River. A deep concrete cutoff wall and vertical freezing columns assure preservation of the foundation in the frozen state.

6. Water-escape structure of the installation on the Sytykan River is located on the left valley slope of the river in the form of an open channel passing through semiconsolidated materials (see Figure 7-7).

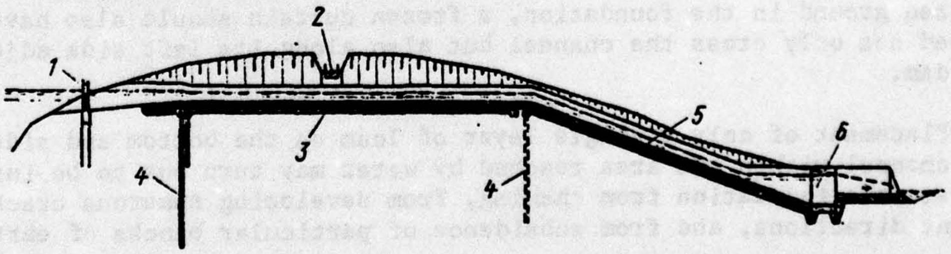

Figure 7-7. Water escape structure of the installation of the Sytykan River.
1 - Icebreaker piers and bridge; 2 - Automobile road; 3 - Loam platform; 4 - Frozen curtain; 5 - Tail race; 6 - Breakwater.

The water-escape structure was designed to handle a flow of 318 m^3/sec with a 0.5% frequency. The structure consists of an intake area, where an ice-retaining structure is located, the channel itself with a transition region, a tail race, and a breakwater. The total length of the water-escape structure is 720 m. The channel, which is 40 m wide at the bottom and 280 m long has a curvilinear outline in plan view with a back-to-front slope of 0.003. The bottom and sides of the channel within the area reached by water are faced with concrete plates 0.2 m thick bedded on a layer of crushed stone 0.2 m thick. Below the concrete facing of the channel there is a layer of loam 0.7 m thick.

After a straight transitional area, the canal feeds into a tail race 230 m long and 18 m wide, which has a slope of 0.077 and is faced with reinforced concrete plates 0.5 m thick on a layer of crushed stone 0.2 m thick. The slope of the channel banks is 1:2. The seams between the plates are packed with a tarry mixture. The tail race terminates in a splash wall.

The ice-retaining wall, which is 120 m long at the front, consists of 7.5x1.2 m piers spaced 6 m apart. A service bridge runs along the piers. The piers are built on piles sunk 7 m into the bedrock.

The water-escape structure of the installation on the Sytykan River was planned to operate without removing the foundation material in the frozen state, but means were provided to limit the growth of the unfrozen zone by installing an earth-concrete wall with a frozen curtain below the structure. Steps were taken to prevent the development of a thawed zone (if formed) in the foundation of the dam, as did occur under the dam on the Myaundzha River and as is tending to form under the dam on the Irelyakh River.

Operating experience with the water-escape structures described above leads us to conclude that it would have been better for this frozen-type dam if a water escape structure had been planned which would operate with the foundation and the portion of the water-escape channel adjacent to the dam maintained in a frozen state. Moreover, since the materials along the path of the canal was fissured, with cracks often as wide as 10-15 cm and filled with ice, two or three deep antiseepage grout curtains should have been installed after thawing the frozen foundation. In order to preserve the frozen ground in the foundation, a frozen curtain should also have been installed not only cross the channel but also along its left side adjacent to the dam.

Placement of only a single layer of loam on the bottom and sides of the channel within the area reached by water may turn out to be insufficient to protect the foundation from thawing, from developing numerous cracks in different directions, and from subsidence of particular blocks of earth.

7-3. THE TEMPERATURE REGIME OF WATER-ESCAPE STRUCTURES

A dam and its water-escape structure frequently and without justification will be built to operate under different temperature conditions.

The problem of choosing the temperature conditions for building water-escape structures is still far from solved. Further study is needed.

As has already been mentioned above, water-escape structures usually are located along one of the valley slopes and above the dam. Depending on the engineering geology and topography of the area, the positioning of the dam and that of the water-escape structure are interrelated. Under these conditions, the temperature regimes for the dam and the water-escape structure cannot be selected without careful study and prediction of their temperature regime during the operating period. It should be kept in mind that, because of the close mutual positioning of these structures, heat exchange necessarily takes place between them, and this may lead to an emergency situation.

The opinion expressed by some specialists to the effect that it is not necessary to build all the structures in an installation according to the same temperature principle should, in our opinion, be considered incorrect. Even under the most favorable frozen ground conditions (which practically eliminates conditions in the permafrost zone), an opinion of this sort requires the most careful examination.

It should be remembered that about half of the total cost of an installation goes into the water-escape structure (see Figure 7-2), and the consequences of damage to it may be very serious, even if the dam itself is not damaged in the process. Damage to a water-escape structure necessarily affects the operational reliability of the entire installation. Moreover, from the operational viewpoint the operating conditions of the water-escape structure are more severe than for the dam. In frozen dams the reliable functioning of the frozen curtain usually determines and guarantees the temperature stability of the body of the dam. The problem with the operation of the water-escape structure lies in the fact that the working surface of this structure is completely open, so that it is subject to the influence of external factors, including seasonal temperature variations, the annual amplitude of which in these regions may reach 100°C. Operating conditions are most severe for the self-regulating water-escape structures.

The bottom and sides of the channel freeze up in the winter. In some years it is also possible that unfrozen zones will persist under the spillways. When floodwaters are passing through a channel, the surface of which may previously have already been warmed up to positive temperatures, the water has a thermal as well as a mechanical effect on the structural elements of the channel. At the start of the flood the water temperature usually is from 0 to +2°C. Its temperature gradually rises, and as the high water recedes, it may have a temperature as high as +6° to +8°C or even more. During a period of abundant summer rain the water spilling through the channel may be deep and its temperature even higher (+15° to +18°C).

This means that conditions of heat and mass exchange within a water-escape channel may be much more severe than in the dam itself, the temperature regime of which, as has been already stated, is governed by the reliable operation of the freezing system. Ignoring these characteristics may lead to radical changes in the temperature regime of the water-escape structure and the foundation materials, with the threat that the thawed zone will spread toward the dam, causing all sorts of problems.

Special attention should be given to these problems in building dams and related structures on permafrost, since the introduction of heat into the body and foundation of the structures may lead to a change in their temperature regime causing a loss of bearing capacity in the foundation, seepage, and loss of overall structural integrity. Therefore all structures of an installation usually should be built to operate under the same temperature conditions; i.e. in building a dam of the frozen type the water-escape structures also should be built in such a way that both the foundation and the other elements are maintained in the frozen state during operation.

Disregard of these requirements of engineering permafrostology will lead to serious consequences, causing damage to the structure which will be very costly to repair.

Examples of how disregard of these requirements results in structural damage are seen in the installations on the Myaundzha River in Magadan Oblast and on the Irelyakh River in the Yakut ASSR [12, 33].

The concrete water-escape structure of the installation on the Myaundzha River was built in 1952-1954. The earth dam with its frozen core creates a reservoir of 4.5 million m^3. On the left bank the dam abuts a rocky shoreline. From the surface to a considerable depth the rock consists of heavily fissured basalts and andesites which have been weathered to the state of gravel. Where the dam joined the rock a concrete tooth was built, as well as a wall joining the dam to a concrete weir. The design of the water-escape structure, which was built without regard for the temperature regime during operation, is described in Section 7-2.

During the first year of operation of the installation, when the pressure on the structure did not exceed 50% of the design pressure, a thawed zone formed in the highly fissured material under the concrete crest of the weir. Then the thawed zone spread under the foundation of the adjacent abutment, and at the end of the second year of operation, positive ground temperatures were reported at the juncture between the earth dam and the abutment (see Figure 5-40). The seepage that started in the foundation of the left-bank part of the dam assumed threatening proportions. In August 1955 the seepage reached a level of 3.5 thousand m^3/day, in January 1956 5.5 thousand m^3, and at the end of winter 8.1 thousand m^3/day. The thawed depth in the foundation under the left-bank junction of the dam reached 35 m. In spite of heavy operation of the air and brine frozen curtain system, the thawed zone continued its progress under the dam.

In 1961-1962 work was done to grout the foundation of the concrete weir and the left-bank portion of the earth dam. This made it possible to freeze the core and the foundation throughout its length, and thus to protect the dam to a considerable extent.

Subsequently the dam was maintained in a frozen state by two cooling systems: one system operated year-round with forced circulation of a calcium chloride brine coolant through the freezing columns; the second operated seasonally, cooling by circulation of cold air.

Construction of this installation was based on two principles for building on permafrost: the earth dam was built on the basis of the frozen principle with a frozen impervious core and the foundation maintained in the frozen state, while the concrete weir was built on the basis of the thawing principle which allows thawing of the foundation materials.

With further operation, thawing of the foundation of the weir caused by heat accumulated in the reservoir and the appearance of seepage through the fissured rock promoted further deep thawing of the foundation (to 70-75 m) and large losses of water from the reservoir.

A water escape channel with a stepped cascade was built on the Irelyakh River in 1962-1964 (see Figure 7-2).

A single dam of the frozen type 20 m high impounds a reservoir of 12,000,000 m^3. The ice-earth cutoff wall in the loam core is maintained by freezing through circulation of cold air in freezing columns during the winter.

An air-frozen curtain was also built along the right side of the water-escape channel, the cooling system being an extension of the frost curtain in the dam. Its purpose it to protect the right side of the channel from the warming action of the water passing through it. This step based on the design assumptions was to protect the foundation and core of the dam from thawing, since the water-escape structure had been planned in a way that would permit thawing of the frozen ground in its foundation during operation. The freezing columns installed along the right side of the canal and in the core of the dam were of the same type.

In the course of building the structures at the installation we suggested extending the frozen curtain of the dam under the water-escape structure and joining it to the bedrock on the left bank. This suggestion was adopted, but the freezing system benearth the channel was planned to consist not of vertical columns but of five pipes running horizontally in the loam tooth (Figure 7-8) through which cold air was circulated. This freezing system design, as became evident later, was inadequate to counteract the warming effect of the seepage flow. After four years of operation it was found that the materials in the foundation of the water-escape channel had thawed to a depth of 15-16 m.

The pattern of change in the temperature regime of the canal foundation depends on the design of the water-escape structure and the nature of the bedding of bedrock in its foundation.

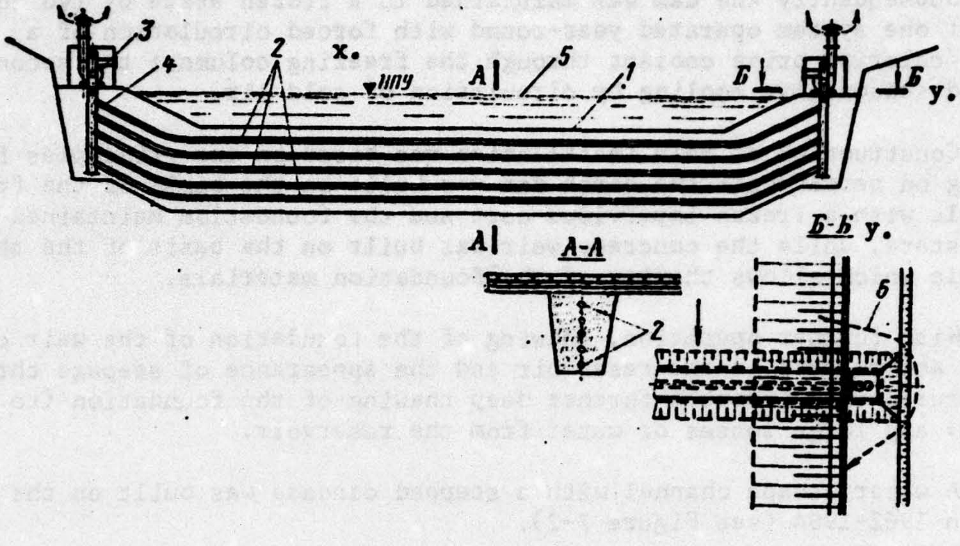

Figure 7-8. Freezing system beneath the water-escape channel of the permanent reservoir on the Irelyakh River.
1 - Bottom of channel; 2 - Freezing pipes; 3 - Fan; 4 - Header;
5 - Wat[illegible] level; 6 - Seepage around the system.
Key: x - NHL; y - B-B.

As has already been stated, the open water-escape channel, which is 40 m wide at the bottom, was built around the dam on the left bank. The channel runs along the second terrace above the floodplain. The bedrock along the route consists of frequent interlayers of argillaceous limestones, sheets of dolomite, calcareous sandstones, marls, and marly clays. The intake sector of the channel has a reverse slope and runs within a layer of diluvial loam, never cutting through it and never revealing the bedrock. Later on, the horizontal sector of the channel and the cascade sector cut into the bedrock to depths of as much as 8-10 m (see Figure 7-3).

As planned, the bottom and sides of the channel were faced with prefabricated reinforced concrete plates installed on a layer of crushed rock and sand 0.4 m thick without antiseepage elements.

The cascade walls were built as retaining walls on a pile foundation. Spaces behind the walls were filled with noncohesive materials; and drain pipes were installed in the concrete walls of the cascades to carry off water. Permeable materials were also placed at the foot of the cascade walls. Thus the construction of the cascade walls and the facing of the bottom and sides of the channel permit seepage of water over the entire length of the channel, so that the design of the channel does not tend to prevent thawing of the permafrost in the foundation.

The loam cutoff wall 3.7 m deep built across the canal, which is artificially cooled, cannot block the path of the seepage flow, since its crest is located beneath the layer of crushed rock on which the reinforced concrete facing plates rest.

The structure of the bedrock is also very interesting. When the trench was dug in the bottom of the channel to build the frozen curtain in the sides of the channel, the semiconsolidated rock was found to be very heavily fissured. The cracks extend both horizontally and vertically and are up to 1[illegible]20 mm wide.

Since the fissured rock in the bottom and sides of the channel were not sealed with any sort of impervious layer, nothing prevented the seepage flow, as it moved through the layer of crushed rock under the plates, from penetrating as time passed into the fissured bedrock of the bottom and sides of the channel around the end of the horizontal curtain. This movement was also favored by the southern exposure of the left bank of the canal, which promoted deep warming of the ground.

If one examines carefully the manner in which the thawed zone is spreading beneath the water-escape channel, one can clearly see the beginning of a thawed zone right in the area of contact between the frozen curtain and the left bank of the channel (see Fibure 5-41).

Hence it is quite probable that uncovering the natural fissured bedrock and the formation of new cracks as a result of blasting to cut the trench for the frozen curtain in the left bank were primarily responsible for the seepage of water around the curtain.

The seepage which was started in this fashion continued to grow, involving a large region, so that the frozen curtain of the channel was unable to halt the progressive thawing of the frozen ground.

The same fate overtook the frozen curtain along the right bank of the channel: the final sector (up to 40% of the total length) was curtained. A thawed zone up to 15 m thick formed under the curtain (see Figure 7-9). The frozen curtain in the right bank of the channel did its job well only in the initial sector. If timely steps had not been taken to reduce seepage in the foundation of the channel and particularly at the right bank, the thawed zone might have grown rapdily toward the left junction of the dam.

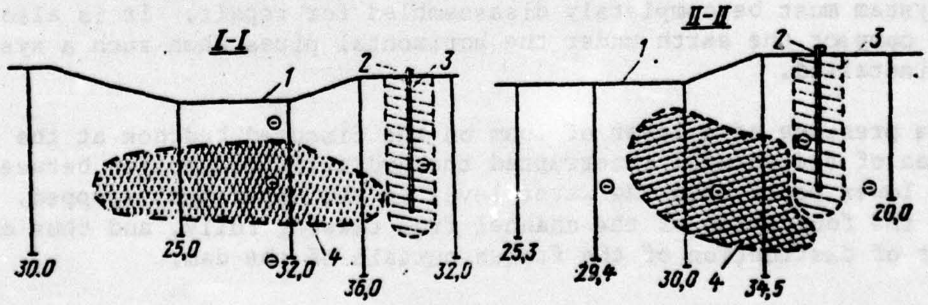

Figure 7-9. Temperature field in the foundation of the water-escape channel of a permanent reservoir on the Irelyakh River (sections as shown in Figure 5-41.).
1 - Bottom of channel; 2 - Freezing columns; 3 - Frozen curtain along right bank of channel; 4 - Thawed zone.

It should also be noted that some deformation of the cascade walls was observed as a result of thawing of the foundation in the region of the stepped cascade. The reason for this evidently was that a portion of the pile foundation was in a positive temperature range over its entire depth. During operation of the spillway, as materials with a high ice content thawed, the plate covering the bottom of the canal settled in particular areas.

Data gathered in the middle of 1968 from eight specially bored thermal holes showed the presence of a thawed zone in the foundation of the spillway channel from 6-15 mm thick with temperatures up to +21°C. A question naturally arose as to whether or not the thawed zone beneath the channel ran completely through the permafrost, whether a permanent seepage flow existed here, and whether there was a hydraulic connection between the upper and lower pools. Were we to expect an increase in seepage and a threatened loss of water from the reservoir? Does the thawed zone that has formed threaten the frozen curtain along the right bank of the channel and the dam itself?

The main cause for thawing in the foundation and under the sides of the water-escape channel evidently was the seepage of water during the warm season from the reservoir through fissures in the semiconsolidated rock that made up the foundation and left bank of the canal.

The design of the water-escape channel did not provide for protection of the permafrost in its foundation from thawing. According to the plan, water seepage is permitted through a sand and crushed-rock filter beneath the reinforced-concrete plate facing the channel. This design was adopted on the assumption that seepage would be insignificant from the viewpoint of water loss from the reservoir, and its warming effect could not threaten degradation of the frozen ground in the foundation and sides of the channel.

It should also be noted that one of the reasons for deep thawing of the foundation was the improper use of the channel.

The design of the freezing system with its horizontal pipe headers should be considered unfortunate, since it cannot be monitored during operation and the system must be completely disassembled for repair. It is also difficult to compact the earth under the horizontal pipes when such a system is being installed.

The presence of a layer of loam on the fissured bedrock at the entrance area of the channel interrupted the hydraulic connection between the upper and lower pools when the water level in the upper pool dropped. It protected the foundation of the channel from thawing fully, and thus avoided the threat of destruction of the frozen curtain of the dam.

In order to remove the threat of thawing of the dam foundation, steps were taken to restore the temperature regime in the material surrounding the channel by grouting the thawed zones, building a frozen curtain in the channel and along its right side, and by taking steps to prevent seepage from the channel into the fissured rock of the foundation by plugging the joints between the facing plates with a grouting solution and thus creating an impervious layer.

Operating experience with the water-escape channel discussed here also has shown that, when building water-escape channels on frozen fissured ground by the unfrozen method, one should cover the region of the channel reached by water with an impervious layer of loam or other materials which will prevent seepage of water into the foundation during periods of high water.

As was shown earlier, in both of the installations discussed, basic rules for construction on permafrost were broken: the dams were built by the frozen method in which the foundation of the body of the dam was to be maintained in the frozen state, while the water-escape structures directly abutting the dams were designed to permit thawing of the foundation materials during operation. The construction of a thin frozen cutoff in the channel for the dam on the Irelyakh River, while permitting seepage and thawing of foundation materials in the rest of the channel area, turned out to be unsuccessful.

It should also be stated that when building frozen curtains in frozen fissured materials, the number of rows of freezing columns should be increased for better freezing results. The number of rows of holes and the distance between them should be determined on the basis of the nature of fissures in the rock.

Problems of grouting or the creation of other types of impervious seals in fissured permafrost should also be investigated when the structure is intended to be built so as to keep the foundation and body of the dam in the frozen state.

Chapter 8

ACCIDENTS AND DAMAGE TO DAMS AND WATER-ESCAPE STRUCTURES

8-1. General

Failure of dams and related structures built on permafrost may be caused by deficiencies in planning the structures and in survey data, failure to observe engineering guidelines during construction, the use of low-grade materials, and, of course, improper operation of the structures. When building dams in the Far North the strength and reliability of structures planned and built with the most careful observance of all standards of construction practice depend chiefly on temperature conditions. Failure to take this factor into account, or errors in predicting the temperature regime of structures during construction and operation after the reservoir has been filled, may be the cause of failure in some structures.

During the early era of water-related construction engineering, a lack of knowledge about the special requirements for construction on permafrost was the cause of most failures. With the passage of time, water-related construction on permafrost has become more and more reliable, as is indicated by the decline in number of accidents. Thus, in the period from 1940-1950 there were six failures, while in the Sixties there were only two.

This pattern resulted from the accumulation of experience, from study, and from generalizations based on them in both planning and construction. At the present time we have the "Temporary Instructions for the Planning of Earth and Rock-Fill Dams in the Far North" (VSN-08-65), compiled by the VNIIG.

The fundamental problems of dam construction on permafrost are treated properly in VSN-08-65, we feel, but they are in need of review in the light of experience accumulated during the last ten years both in domestic and foreign dam construction on permafrost.

Enormous quantities of water are impounded in reservoirs created by dams (for example, 36,000,000,000 m^3 by the Vilyuy dam, 12,000,000 m^3 by the Irelyakh dam, and 35,000,000 m^3 by the Sytykan dam). Failure of such structures could cause significant damage.

In contrast to accidents involving industrial, transportation, and other structures, the loss of which in many cases is equivalent to the cost of restoring the damage to the structure itself, accidents involving water-impoundment structures usually lead to destruction and damage to other structures located in the valley below the dam, so that the loss due to accidents involving water-impoundment structures sometimes exceeds by several times the cost of the structure itself.

In view of the importance of such structures as dams, the specialist charged with the responsibility of planning and building a dam must above all carefully study cases of dam failure under conditions similar to those in which the new structure will function. The most important of these conditions are the engineering geology and frozen-ground conditions of the dam foundation, the hydrological regime of the river, and the quality of the structural materials.

Understanding the conditions under which spring floods occur in a river both during construction and during the period of operation is a problem of primary importance. Floods on rivers within a small catchment area are more sudden than on rivers with large basins. When slopes are steep and consist of permafrost, surface runoff is very rapid, being shed almost entirely by the permafrost into the river, so that floods on such rivers show a rapid rise in discharge and water level, very often reaching peaks within 1.5-2 days.

In taking up our discussion of cases of damage to dams and water-escape structures it must be mentioned that we shall be limited to examples from domestic experience in dam building on permafrost, since published information from foreign sources in this area is not available.

8-2. ACCIDENTS WITH UNFROZEN DAMS

1. The dam on the Kadychkan River in Magadan Oblast (see Figure 5-7). Construction of the dam was started in 1960, and after an interruption in January 1962, work was resumed only in 1966. Work was completed in 1968, and the structure was placed in operation in September of the same year.

A washout occurred on July 10, 1970 at 7:30 in the morning, and after four and one-half hours the water level in the reservoir had dropped 4 m. The reservoir was completely emptied. A tunnel formed in the body of the dam in the region where the washout occurred. An arch 4-5 m thick remained for five days and then collapsed. The upper part of the dam was frozen. Cracks up to 1.2 m wide and 2.5 m deep appeared at the crest of the dam. When the crest of the dam was excavated, the top of the frozen ground was found at a depth of 1.8 m, and the level of the top of the core was found at a depth of 2.0 m.

Serious deviations from the plan were permitted during construction of the dam, so that during the first year of operation following filling of the reservoir, heavy seepage began through the body and foundation of the dam. Boring from the crest and determination of the permeability characteristics of the material in the body of the dam showed that the material did not correspond to the design specifications. The base of the tooth was not backfilled to the design level over a considerable area. The permeability coefficient of the material along the length of the dam varied from 250 to 1,000 m/day, indicating that the material in the body of the dam was not uniform.

The cut for the tooth of the dam at the right bank deviated in a major way from the plan: the length and depth did not agree with the design specifications (see Figure 8-1).

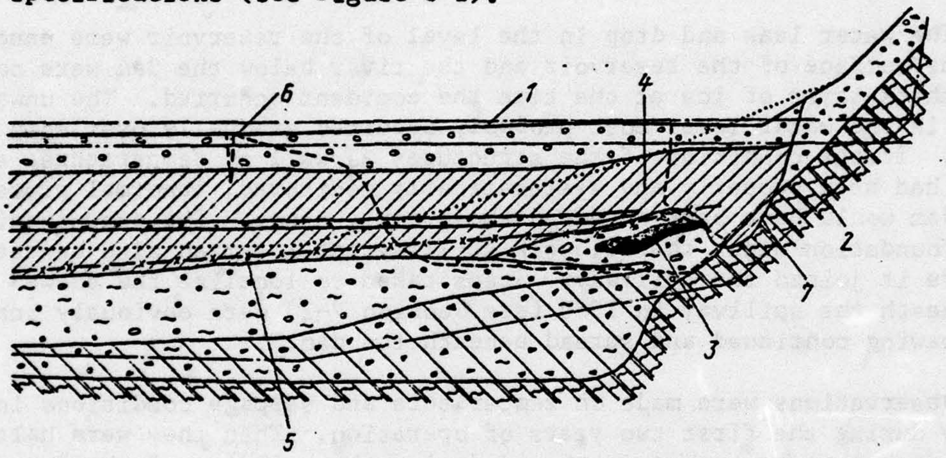

Figure 8-1. Right-bank junction of the dam on the Kadychkan River. 1 - Crest of the dam; 2 - Outline of tooth as planned; 3 - Lens of ice; 4 - Washout in the body of the dam on July 14, 1970; 5 - Top of permafrost; 6 - Crack; 7 - Actual outline of base of tooth.

Investigation revealed the edges of an ice lens below the tooth of the dam. The ice lens was 20-30 cm thick and covered with a layer of diluvial material. This ice lens was not detected during the survey stage, evidently because the survey holes were spaced too widely. Surveys of the completed dam in February 1968 revealed the presence of a buried ice lens up to 1.0 m thick below the actual level of the tooth, but still within its design outline. Thus, if the tooth had been built as planned, this lens might have been completely removed.

No observations were made on the condition of the dam during the period of operation, since the plan did not provide for the installation of monitoring equipment in the dam.

The cause of the failure was a washout that developed in the body of the dam at the right-bank junction. Buried ice was found in the foundation of the dam during its construction. This later thawed as a result of seepage under pressure. The voids that formed were not filled by material from the body of the dam because it was frozen, and an arch formed over the site of the washout.

2. Temporary dam on the Irelyakh River (see Figure 5-1). A destructive leak of water from the temporary reservoir on the Irelyakh River occurred in November 1964. Water was retained only behind a cofferdam built during the construction period at the entrance area to the intake channel, which was not removed when it should have been. This situation was responsible for the site and etiology of the water leak. Further investigation revealed the presence of a cavity in the foundation of the dam where it joined the left cribwork abutment of the water-escape structure.

The water leak and drop in the level of the reservoir were unnoticed, since the surface of the reservoir and the river below the dam were covered with a thick layer of ice at the time the accident occurred. The unwanted decline in the water level most probably occurred gradually over some period of time. If the condition of the structures as well as temperatures and seepage had been properly and systematically monitored, abnormal processes in the dam would have been detected at an early stage. The deep thawing of the foundation under the cribwork spillway later spread also beneath the dam where it joined the spillway. Steps taken to localize the thawed zone beneath the spillway in 1960 (see Section 7-2) were obviously inadequate, since thawing continued and spread beneath the dam.

Observations were made on temperature and seepage conditions in the dam only during the first two years of operation. Then they were halted, and the measuring equipment destroyed during the building of the dam was not replaced.

Location of the spillway outside the dam might have prevented its foundation from thawing. The interface between the relatively rigid elements of the spillway (wood or concrete) and the pliant body of an earth dam is always a weak point. When the body of the dam settles, seepage paths may concentrate along this interface. It is practically impossible to mechanize the placement and compaction of earth along the cribwork of a spillway and in the cells of the cribwork, while manual compaction is slow and, more importantly, difficult to control.

The bedding of the rock beneath the water-escape structure at the intake canal showed fissures: steeply dipping layers (50-60° were present with strike in the direction of flow, and the fissures were not always filled with ice or rock-weathering products. No steps were taken to close the fissures with grouting or other materials and there was no obstacle to seepage through these rocks around the joint with the right abutment.

If the water-escape structure had been located outside the dam, these fissures would not have been uncovered, and the layer of diluvial loams covering the bedrock would have been a reliable obstacle to seepage at the dam junction.

3. Dam on the Tatta River (Yakut ASSR). This earth dam, filled with material excavated for the water-escape channel, was built in 1953. A bypass with a floodgate was constructed to carry away water along the left bank. The floodgate with an opening 15 m wide was installed 40 m from the head of the bypass canal. The floor of the sluiceway was located at the low-water level.

During the first high water with a design head of 1.5 m, the sluiceway, canal, and dam were destroyed. Erosion began at the left-bank abutment, then spread beneath the apron.

The cause of the failure was thawing of icy ground and uneven settlement of the foundation. When a trench was cut for the tooth, layers of ice 15-20 cm thick were found at a depth of 3 m, but necessary steps to remove the hazardous materials from the foundation were not taken. After the reservoir was flooded, seepage developed as such materials thawed. As a result of the heat exchange that took place, thawing intensified, with the appearance of uneven subsidence. The failure resulted from lack of attention to problems of engineering geology and errors in planning and construction.

There are several more examples of failures of dams built for agricultural purposes in the Yakut SSR. These include the dams built on the Suola River (1948), the Myla River (1953-1954), and the Tatta River (1954) [46]. The basic reason for the failure of these dams were low-quality construction and design, the latter done without serious attention to problems of engineering geology. Insufficient attention was given to the quality of the junction between the abutments and the body of the dam. Frozen ground, which cannot be compacted, was used as fill for the body of the dams. Ice-saturated materials were left in the dam foundations. Thawing of such materials necessarily leads to uneven settlement, the appearance of seepage, the development of leaks, and the failure of the impoundment.

4. The earth dam on Stake 89 Creek (City of Noril'sk). The 5.5 m high dam impounded a head of 4.75 m and was built to regulate the flow of a chain of lakes and convert them into a reservoir. The dam was built in the dry behind a cofferdam, and carefully. The foundation of the dam contained loamy sand and loam with ice inclusions. In the autumn a year after cperation began, seepage flow containing earth was discovered at the right-bank junction, and a day after the seepage started, the dam was breached at this location. The reason for the failure was the thawing of the ice inclusions and the development of seepage at the interface between the frozen ground of the foundation and the unfrozen ground of the body of the dam. Further thawing of the icy foundation material as a result of the seepage flow led to a loss of bearing capacity [27].

5. Earth dam on Kvadratnyy Creek (see Figure 3-5). The dam was built in 1941. The foundation of the dam contained permanently frozen loamy sands and sands with ice veins. The dam lasted one month and was destroyed as a result of melting of ice includions in the foundation materials and at the joints with the banks.

6. Earth dam with a concrete cutoff wall on the Right Magdagacha River (see Figure 3-4). The dam was built in 1932 to create a reservoir for domestic and drinking-water purposes. The dam was 550 m long, 7.3 m high, and impounded a head of 4 m. It was built of gravelly loam. After the reservoir was filled, permanently frozen icy fissured porphyrites in the foundation began to thaw. The seepage that developed promoted further thawing and uneven settlement of the foundation, so that the concrete cutoff walls split. In 1935 the dam was destroyed.

Development of seepage in the body of a dam and the foundation is to be expected in unfrozen dams. Therefore the planning should provide appropriate means to prevent piping (inverted filters, drains, etc.). The structures should be flexible and be able to accept uneven settlement of the foundation without loss of overall integrity and resistance to seepage.

8-3. ACCIDENTS WITH FROZEN DAMS

The dam on the Pevek River (see Figures 5-50 and 5-51) was built to create a reservoir for household and drinking-water purposes. Seepage was discovered on August 6, 1970, when a well in the bottom intake valve control room on the downstream slope of the dam was flooded. At 9:15 in the morning on August 12 a sinkhole 6 m wide and more than 1 m deep appeared in the dike connecting the water intake well with the dam, and at 4:20 in the afternoon seepage began to intensify in the vicinity of the well. At midnight on August 13, 1970, a tunnel washout developed in the body of the dam. It rapidly increased in size and reached a cross-section of about 40 m^2 (see Figure 8-2). The water level in the reservoir dropped 6.35 m, Other structures below the dam were also destroyed.

The cause of the failure was seepage of water from the reservoir through the body of the dam, carrying earth with it along the trench containing the intake pipes. This seepage-piping phenomenon could have occurred as a result of the absence of a frozen curtain at the left junction.

This occurred as a result of carelessness during construction and planning deficiencies. During construction, the trench in the bedrock to accommodate the pipes for the bottom water intake was made 3.5 m wide because of construction conditions, but concrete placement strictly followed the plans; i.e. it did not grip the rocky walls of the trench. The space that was left was filled with sandy material.

There are also other indications that the trench was not properly closed off. The plans called for backfilling the trench with earth in 15-20 cm layers and compacting these layers with pneumatic hammers. Actually the trench was filled partly with frozen ground. In addition, an instrumentation cable was laid along the top of the concrete jacket on the water-intake pipes before backfilling with earth, and in order to protect the cable from damage during the backfill operations, it was covered with boards. Spaces remained under the boards, of course, and these became the primary paths for seepage of water which carried heat into the frozen body of the dam.

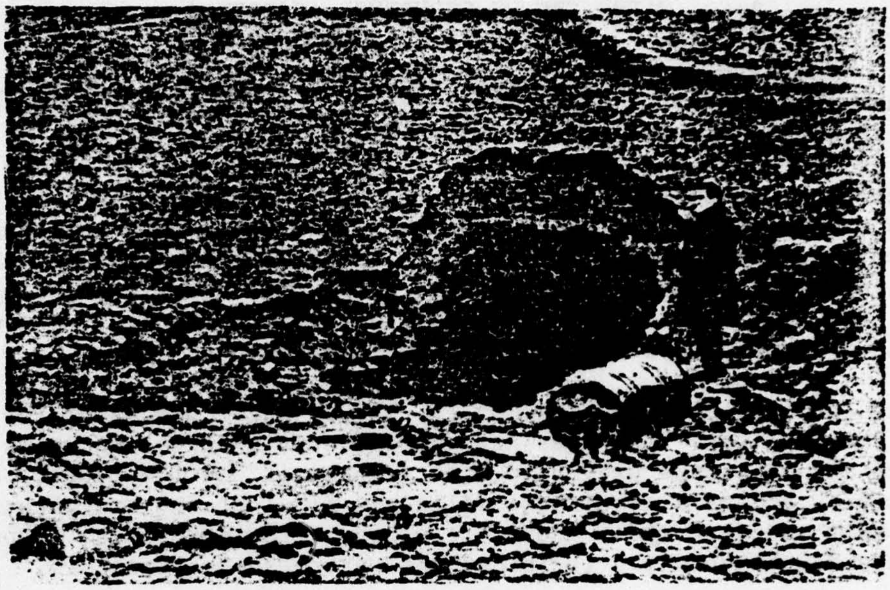

Figure 8-2. Washout in the dam on the Pevek River.

Under these conditions the subsequent course of events leading the degradation of the frozen body of the dam and the development of seepage-piping processes becomes clear.

The frozen curtain in the junction with the left bank as well as in the region of the bottom water intake was doomed because of the perforating openings (under the boards between the cable and the concrete). The seepage that developed and grew, carrying with it the sandy-clay backfill material from the bottom water intake trench, led to the failure.

Seepage and piping of earth from the body of the dam where there was no frozen curtain might have been avoided if there was at least a minimum supporting prism with the usual inverted filter on the downstream slope of the dam (in the region of the bottom water-intake trench).

This example indicates the necessity of building inverted filters along the downstream edge of a frozen core. The presence of such a filter where there were defects in the frozen curtain should have prevented the erosion of earth from the body of the dam.

Defects in planning were the principal cause of the failure. This example emphasizes the inadvisability of building bottom water intakes in dams on permafrost, and more espeically those built as frozen dams. It confirms the error in the recommendations of VSN-08-65 which permit construction of such water intakes.

8-4. WATER-ESCAPE STRUCTURE ACCIDENTS

1. Water-escape structure for the temporary dam on the Irelyakh River. The discharge of the first flood which passed through the finished spillway was more than 50% higher than the design discharge.

According to the original plan, the spillway was to be a cribwork header 24 m wide and 7.5 m long with a tail race 100 m long built on a pile foundation. At the entrance to the short intake channel ahead of the spillway, an ice-retaining structure was built which consisted of clusters of piles, but in place of the permanent tail race a temporary one was built with a stepped cascade 20 m long and 6 m wide.

The spring flood began on May 19, 1959. Large pieces of ice began to enter the spillway. The presence of two stream-directing walls in the channel of the tail race prevented free passage of the ice floes. An ice jam developed, and as a result the walls and floor of the trough were stripped away and destroyed. Severe erosion of the downstream slope and face of the dam began where it joined the water-escape structure.

The water level in the reservoir exceeded the catastrophic design level by 0.5 m. Seepage started through the crushed rock fill on the crest of the dam above the cribwork cutoff wall, and as the water continued to rise in the reservoir, it began to spill over the top of the dam. A situation had developed which threatened the dam. Under these conditions the right-bank abutment of the spillway was torn away. Three hours later the level in the upper pool dropped 21 cm, and after two days it had dropped 1.44 m.

In planning the reapirs to the spillway, better discharge figures were used and the design was changed to increase the size of the passage for the water.

The principal causes for the failure of the water-escape structure during the first year of operation were as follows:

a) Insufficient information about the hydrological regime of the river, so that incorrect determinations were made as to the discharge, the size of the water passages, the backed-up water levels, and the elevation of the dam crest.

b) Improper design of the ice-retaining structure and incorrect placement of this structure. The ice-retaining structure was located in a curved drop zone where it fed into a steeper section, so that current speeds approaching it were too high. In rebuilding the installation the axis of the ice-retaining structure was set back 10 m into the upper pool and it was built in the form of concrete piers rather than clusters of piles.

c) Improper design of the tail race trough, and the presence of too much resistance in the trough in the form of stream-directing partitions.

d) Weak design of the connection between the cribwork header of the spillway and the foundation, so that the subgrade antiseepage outline of the structure was inadequate. The seepage flow that developed when the reservoir was filled started rapid degradation of the frozen ground under the spillway, which subsequently intensified because of improper construction of the junction between the dam and the spillway abutment.

e) In preparing the foundation for the head of the spillway and the intake canal, fissured bedrock was found on the right bank. No provisions were made in the plan to provide grouting or other methods of sealing. There was nothing to prevent seepage through these fissured rocks around the junction between the right abutment of the spillway and the bank.

In our opinion, the proper solution would have been to create a grout curtain under the spillway after thawing the semiconsolidated fissured foundation materials.

2. Spillway for the installation on the Kazachka River. A wooden spillway was built around the body of the dam on the right bank. The wooden spillway was located 50 m from the tooth on piles sunk 5 m into the foundation material. In the beginning and middle sections of the trough of the spillway, wooden cutoff walls were built and sunk 3 m below the bottom of the trough. The trenches for the cutoff walls were filled hydraulically with loamy sand. The discharge channel was traversed by lenses and layers of ice more than 3 m thick. The plan called for removal of the ice and icy earth in the sides of the channel for a distance of 150 m; the ice was not removed from the foundation of the channel.

The foundation of the dam contained cobbly-gravelly material with a high content of ice, silty clayey loamy sand and fine sand underlain by frozen loam. Thick lenses of fossil ice were found at the left and right-bank junctions.

The central portion of the unfrozen dam was built of fine sand and loamy sand, while the lateral prisms were made of sandy-gravelly earth and stones.

In September 1965 the spillway was destroyed by a flood resulting from fall rains (Figure 8-3). The plan did not call for complete transsection of the large ice inclusions with vertical antiseepage structures, and construction operations deviated from the plan. Specifically, excessively wet loamy sand material was substituted for compact loam in the foundation of the wooden spillway floor and the piles were not sunk deep enough.

When the trench for the spillway canal was exposed, ice wedges and lenses were revealed where the spillway adjoined the banks and in adjacent areas of the intake and discharge channels along outcrops on the valley walls. Later on their position was quite easy to detect from the terraced depressions and fissures in the upper surface of the tundra, and from eroded areas and cavities where the spillway joined the bank and beyond along both sides of the canal.

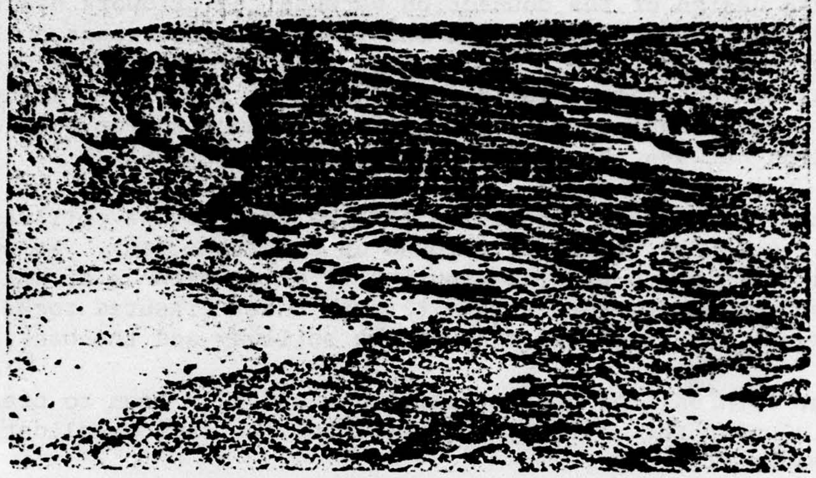

Figure 8-3. Spillway on the Kazachka River.

The design of the spillway trough as given in the plan was not appropriate for the frozen-ground conditions in the area where the reservoir was built. Subsidence of the ice-saturated materials in the foundation of the spillway was unavoidable, since the structures were planned without provision for freezing unfrozen zones and preserving the frozen ground. Vertical antiseepage structures, even when sunk deep into the icy frozen ground of the foundation and banks and with careful packing of cohesive earth in the voids, cannot guarantee structural stability if ice inclusions are not completely intercepted, since installing a cutoff wall in the frozen ground necessarily disturbs its structure and thermal regime. Even the most careful compaction of ground used to backfill voids creates the conditions necessary for later melting of ice which is not cut off by the wall. Concentrated bypass seepage will certainly appear, and this in turn speeds up the process of ice melting, forming thermokarst, which is seen as depressions and fissures, and later on, erosion cavities.

As a result of uneven settlement, improper control of the density of the earth used to backfill the trench after installation of the cutoff, and the impossibility of assuring tight contact between the thin wooden floor and the earth beneath, voids and uncompacted areas could have formed which, together with the cavities developed as a result of the ice melting in the banks, became foci of concentrated seepage.

In view of the substantial uneven settlement of the foundation earth beneath the floor of the spillway and the melting of ice lenses in the floodplain earth of the Kazachka River, the plan for building the abutments and antiseepage walls should have called for careful compaction of the foundation earth and obligatory control of the density of the bulkheads and the density of the backfill.

Locating the wooden spillway 50 m from the antiseepage tooth, which was a continuation of the tooth for the central portion of the dam profile, laid the groundwork for heavy melting of ice in the soil of the bottom and sides of the intake channel in the area from the tooth to the spillway floor. Provisions were not made in this area to control seepage from the intake channel to the foundation materials, especially in the region where the wooden spillway floor adjoined the slopes of the channel. Installation of an apron ahead of the spillway floor connected to the bottom and sides of the channel undoubtedly would have prevented heavy seepage around the spillway and under its floor.

These examples of failures of water-escape structures in installations built on permafrost indicate that they occurred chiefly because of inadequate design, improper engineering solutions, incorrect location of the water-escape facilities with respect to the dam, and deviations and gross disregard for the technical specifications during construction.

A frequent cause of failure is the incorrect evaluation of engineering geology and frozen ground conditions in the foundation of structures, so that appropriate provisions are not made in their planning.

Sometimes failure of water-escape structures and dams occur as a result of improper sizing of the channels due to incorrect determination of discharge for design purposes. In this case errors are frequently due to inadequate original data for determining hydrological characteristics, since in most cases sufficient observational data is lacking for areas where dams are built in the Far North, especially in the case of small rivers and their tributaries.

BIBLIOGRAPHY

1 Adamovich, A. N. *et al*. A dam of local materials with an injection core. In: Trudy IV soveshchaniya-seminara po obmenu opytom stroitel'stva v surovykh klimaticheskikh usloviyakh (Transactions of the Fourth Seminar Meeting to Exchange Experience in Construction under Severe Climatic Conditions). Vorkuta, 1966.

2. Alpatkin, M. T. Mekhanizatsiya zemlyanykh rabot v usloviyakh mnogoletney merzloty (Mechanization of earthwork under permafrost conditions). Stroizdat, 1965.

3. Baranov, I. Ya. Printsipy geokriologicheskogo (merzlotnogo) rayonirovaniya oblasti mnogoletnemerzlykh gornykh porod (Principles for subdividing on the basis of geocryology (frozen ground) the region in which the ground remains frozen through the summer). Moscow, "Nauka", 1965.

4. Batenchuk, Ye. N., G. F. Biyanov, L. N. Toropov, and Yu. N. Myznikov. Zimnyaya ukladka svyaznykh gruntov na Kraynem Severe (Winter placement of cohesive earth in the Far North). Moscow, "Energiya", 1968.

5. Baulin, V. V. *et al*. Geokriologicheskiye usloviya Zapadno-Sibirskoy nizmennosti (Geocryological conditions in the West Siberian Lowland). Moscow, "Nauka", 1967.

6. Belopukhova, Ye. B. Ice-segregation phenomena in the seasonally frozen layer of the Irelyakh River valley. Merzlotnyye issledovaniya 1961, Vyp. 1. (Izd. Moskov. Gos. Univ.).

7. Biyanov, G. F. Construction of a water-management installation under permafrost conditions. In: Trudy soveshchaniya-seminara po obmenu opytom stroitel'stva na vechnomerzlykh gruntakh (Transactions of the Seminar Meeting to Exchange Experience in Construction on Permafrost). Magadan, 1964.

8. Biyanov, G. F. Experience in the construction of a water-management installation under permafrost conditions. *Gidrotekhnicheskoye stroitel'stvo*, 1965, No. 10.

9. Biyanov, G. F. Experience in the construction of low-pressure dams of frozen loam under conditions in the Far North. Kolyma, 1965, No. 4.

10. Biyanov, G. F. Experience in the construction and operation of low-pressure dams on permafrost. In: Trudy IV soveshchaniya-seminara po obmenu opytom stroitel'stva v surovykh klimaticheskikh usloviyakh (Transactions of the Fourth Seminar Meeting to Exchange Experience in Construction under Severe Climatic Conditions). Vorkuta, 1966.

11. Biyanov, G. F. Dam construction. In: Trudy V soveshchaniya-seminara po obmenu opytom stroitel'stva v surovykh klimaticheskikh usloviyakh (Transactions of the Fifth Seminar Meeting to Exchange Experience in Construction under Severe Climatic Conditions. Tyumen', 1968.

12. Biyanov, G. F. Temperature and seepage conditions in a by-wash and dam made of local materials under severe climatic conditions. In: Trudy VI soveshchaniya-seminara po obmenu opytom stroitel'stva v surovykh klimaticheskikh usloviyakh (Transactions of the Sixth Seminar Meeting to Exchange Experience in Construction under Severe Climatic Conditions). Krasnoyarsk, 1970.

13. Biyanov, G. F. Experience in construction and operation of low-head dams on permafrost. Gidrotekhnicheskoye stroitel'stvo, 1970, No. 9.

14. Biyanov, G. F. Dam construction in the Canadian North. Gidrotekhnicheskoye stroitel'stvo, 1970, No. 1.

15. Biyanov, G. F. Construction of the rock fill dam for the Vilyuy Hydroelectric Power Station. Gidrotekhnicheskoye stroitel'stvo, 1970, No. 1.

16. Biyanov, G. F., and L. N. Toropov. Experience with the use of turbojet devices in hydraulic construction. Gidrotekhnicheskoye stroitel'stvo, 1970, No. 3.

17. Biyanov, G. F. Vodoponizheniye pri stroitel'stve gidroelektrostantsiy (Lowering the water table during construction of hydroelectric power stations). Moscow, Gosenergoizdat, 1958.

18. Biyanov, G. F. Compaction of highly porous soil materials. Na stroykakh Rossii, 1965, No. 6.

19. Biyanov, G. F. Unwatering and blocking the river during building of the Vilyuy Hydroelectric Power Station. Gidrotekhnicheskoye stroitel'stvo, 1966, No. 2.

20. Biyanov, G. F. Construction of tailing pond dams on permafrost. Paper presented at the Conference on Hydraulic Engineering in the North. Krasnoyarsk, 1972.

21. Biyanov, G. F. and V. N. Lazutin. Construction of a water-use facility under permafrost conditions. Paper presented at the Conference on Hydraulic Engineering in the North. Krasnoyarsk, 1972.

22. Biyanov, G. F. Experience in the construction of dams on permafrost in Yakutia. In: Trudy II Mezhdunarodnoy konferentsii po merzlotovedeniyu (Transactions of the Second International Conference on Permafrostology). Vyp. 7. Yakutsk, 1973.

23. Bliznyak, Ye. V. Planning and building dams under permafrost conditions. Gidrotekhnicheskoye stroitel'stvo, 1937, No. 9.

24. Bogdanov, N. S. Vechnaya merzlota i soorvzheniya na ney (Permafrost and construction on it) St. Petersburg, 1912.

25. Bogoslovskiy, P. A. Construction of earth dams in permafrost regions. Trudy Gor'kovskogo inzhenerno-stroitel'nogo instituta, 1958, vyp. 29.

26. Bogoslovskiy, P. A., and G. I. Kuznetsov. Dams made of local materials and foundations for water-engineering structures in permafrost regions (review paper). In: Trudy VI soveshchaniya seminara po obmenu opytom stroitel'stva v surovykh klimaticheskikh usloviyakh (Transactions of the Sixth Seminar Meeting to Exchange Experience in Construction under Severe Climatic Conditions). Vol. VIII, Vyp. 1. Krasnoyarsk, 1970.

27. Borisov, G. A., and G. Ya. Shamshura. Experience in the planning, construction, and operation of earth dams in Noril'sk. In: VII Mezhduvedomstvennoye soveshchaniye po merzlotovedeniyu. Materialy po inzhenernomu merzlotovedeniyu (Eighth Interdepartmental Meeting on Permafrostology: Materials on engineering permafrostology). Izd. Akad. nauk. SSSR, 1959.

28. Birulya, V. I. Raschet i kontrol' uplotneniya gruntov (Calculation and control of compaction of soil materials). Moscow, Avtotransizdat, 1958.

29. Brakhina, N. F. Prediction of change in the depth of seasonal thawing as a result of working loam borrow pits to get fill for the blanket seal of the dam for the Vilyuy Hydroelectric Power Station. Merzlotnyye issledovaniya, 1964, vyp. IV. (Izd. Moskov. Gos. Univ.).

30. Brusse, A. G., V. D. Glebov, and B. V. Detkov. Polyethylene seal in the cofferdam for the right-bank dam of the Ust'-Khantayka Hydroelectric Power Station. Gidrotekhnicheskoye stroitel'stvo, 1971, No. 11.

31. Bykov, N. I., and P. N. Kapterev. Vechnaya merzlota i stroitel'stvo na ney (Permafrost and construction on it). Moscow, Transzheldorizdat, 1940.

32. Vasil'yev, A. F., and A. F. Pavlov. Building the earth dam for the Upper Tuloma hydraulic installation. *Energeticheskoye stroitel'stvo*, 1965., No. 1.

33. Vedernikov, A. Ye. Frost processes in the body and foundation of the dam on the Myaundzha River. Trudy VNII-I, 1963, Vol. XXII, Magadan. (All-Union Scientific Research Institute of Gold and Rare Metals.)

34. Sumgin, M. I. (Ed.). Vodosnabzheniye zheleznykh dorog v rayone vechnoy merzloty (Water supply for railroads in the permafrost region). Moscow, Transzheldorizdat, 1939.

35. Volodenkov, P. I. Building earthworks by the method of earth fill in water. *Energeticheskoye stroitel'stvo*, 1967, No. 1.

36. Vyalov, S. S. Reologicheskiye svoystva i nesushchaya sposobnost' merzlykh gruntov (Rheological properties and bearing capacity of frozen ground). Moscow, Izd. Akad. Nauk SSSR, 1959.

37. Temporary instructions for planning earth and rock-fill dams in the Far North. VSN-08-65. Leningrad, "Energiya," 1965.

38. Geologiya i neftegazonosnost' Zapadno-Siberskoy nizmennosti- novoy neftyanoy bazy SSSR (The geology and oil and gas fields of the western Siberian lowland -- the new oil lands of the USSR) Novosibirsk, Izd. Sib. Otd. Akad. Nauk SSSR, 1963.

39. Goldtman, V. G. The method of thawing permafrost before construction. *Promyshlennoye stroitel'stvo*, 1959, No. 10.

40. Gorbunov, B. P., and L. I. Kudrenko. The electrical method of thawing and compacting permafrost in foundations prior to construction. *Osnovaniya, fundamenty i mekhanika gruntov*, 1961, No. 4.

41. Dalmatov, B. I., and V. S. Lastochkin. Iskusstvennoye zasoleniye gruntov v stroitel'stve (Artificial salinization of earth in construction). Leningrad, Stroyizdat, 1966.

42. Dostovalov, B. N., and V. A. Kudryavtsev. Obshcheye merzlotovedeniye (General permafrostology). Izd. Moskov. Gos. Univ., 1967.

43. Yevdokimov-Rokotovskiy, M. I. Postroyka i ekspluatatsiya inzhenernykh sooruzheniy v vechnoy merzlote (Construction and operation of engineered structures in permafrost). Tomsk, 1931.

44. Zhilenas, S. V., Yu. N. Myznikov, and N. A. Ten. Winter placement of moraine earth materials in the dam for the Ust'-Khantayka installation. *Gidrotekhnicheskoye stroitel'stvo*, 1973, No. 3.

45. Zhukov, V. F. Predpostroyechnoye protaivaniye mnogoletnomerzlykh osnovaniy fundamentov (Preconstruction thawing of permafrost foundations). Moscow, Izd. Akad. Nauk SSSR, 1958.

46. Zhukov, V. F. and V. D. Ponomarev. Preconstruction thawing of permafrost. In: Trudy II Mezhdunarodnoy konferentsii po merzlotovedeniyu (Transactions of the Second International Conference on Permafrostology). Vyp. 7, Yakutsk, 1973.

47. Zlatkin, M. A. Opyt stroitel'stva plotin v Yakutii (Dam construction experience in Yakutia). Izd. Akad. Nauk SSSR, 1959.

48. Instruction for thawing frozen ground with water injection rods. Magadan, 1962 (VNII_I).

49. Kalabin, A. I. Permafrost and the hydrogeology of northeastern USSR. Trudy VNII-I, 1960, Vol. XVIII, Magadan.

50. Kamenskiy, R. M. The thermal regime of the foundation and blanket seal of the Vilyuy Hydroelectric Power Station dam. In: Trudy II Mezhdunarodnoy konferentsii po merzlotovedeniyu (Trnasactions of the Second International Conference on Permafrostology). Vyp. 7. Yakutsk, 1973.

51. Kronik, Ya. A., S. B. Ukhov, and N. A. Tsytovich. Artificial salt treatment of soil materials to control frost heaving. Osnovaniya, fundamenty i mekhanika gruntov, 1969, No. 1.

52. Krylov, M. M. Zamorozhennyye plotiny (Frozen dams). Merzlotovedeniye, 1946, Vol. 1, No. 1.

53. Kuryachiy, A. M. Thawing with water in construction on permafrost. Kolyma, Magadan, 1959, No. 12.

54. Leshkov, V. G. Spravochnik drazhnika (Dredger's handbook). Moscow, "Nedra", 1968.

55. Lofitskiy, V. N. Zimniye zemlyanyye raboty v gidroenergeticheskom stroitel'stve (Winter earth work in hydroelectric power plant construction). Moscow, Gosenergoizdat, 1961.

56. Lyskanov, G. A. Opyt stroitel'stva plotiny merzlogo tipa (Experience with dam construction of the frozen type). Yakutskoye knizhnoye izd., 1964.

57. Lavrov, P. A. Earth work under the conditions of the Far North. Promyshlennoye stroitel'stvo, 1960, No. 9:41-43.

58. L'vov, A. V. Poiski i ispytaniya vodoistochnikov vodosnabzheniya na zapadnoy chasti Amurskoy zheleznoy dorogi v usloviyakh "vechnoy" merzloty pochvy (Exploration and testing of sources of water supply along the western sector of the Amur Railway under "permafrost" conditions). Irkutsk, 1916.

59. Mel'nikov, P. I. Merzlotno-geologicheskiye usloviya vozvedeniya grazhdanskikh i promyshlennykh zadaniy na territorii tsentral'noy Yakutii i opyt stroitel'stva (Frozen-ground geological conditions for building civilian and industrial buildings in central Yakutia and construction experience). Moscow, Izd. Akad. Nauk SSSR, 1951.

60. Mel'nikov, P. I. Patterns of distribution and development of frozen soil and rock in the Lena River basin. In: Materialy po obshchemu merzlotovedeniyu (Materials on general permafrostology). Moscow, Izd. Akad. Nauk SSSR, 1959.

61. Moiseyev, S. N. Stroitel'stvo Irkutskoy GES na r. Angare (Construction of the Irkutsk Hydroelectric Power Station on the Angara River). Moscow, Gosenergoizdat, 1959.

62. Moiseyev, S. N. Kamenno-zemlyanyye i kamennonabrosnyye plotiny (Earth-rock fill and rock fill dams). Moscow, "Energiya," 1970. 224 p.

63. Moiseyev, S. N. Rock and cobble-gravel fill work in the body of a dam during winter. Gidrotekhnicheskoye stroitel'stvo, 1966, No. 3.

64. Myznikov, Yu. N. Filling of cofferdams and preparing loam for building the Vilyuy Hydroelectric Power Station. Ekspress-informatsiya, 1964, No. 161 (Orgenergostroy).

65. Myznikov, Yu. N., S. V. Zhilenas, and N. A. Ten. Construction of the right-bank dam for the Ust'-Khantayka Hydroelectric Power Station. Gidrotekhnicheskoye stroitel'stvo, 1973, No. 4.

66. Nersesova, Z. A. Physical phenomena and processes in freezing, frozen, and thawing ground. In: Osnovy geokriologii (merzlotovedeniya). (Foundations of geocryology [permafrostology]). Vol. 1. Moscow, Izd. Akad. Nauk SSSR, 1959.

67. Nersesova, Z. A., and N. A. Tsytovich. Unfrozen water in frozen ground. Paper presented to the International Conference on Permafrostology. Moscow, Izd. Akad. Nauk SSSR, 1963.

68. Nekrasov, I. A. Taliki rechnykh dolin i zakonomernosti ikh rasprostraneniya (Unfrozen zones in river valleys and patterns of their distribution). Moscow, "Nauka," 1967.

69. Nikolayev, N. I. The problem of the origin of mantle loams. Izv. Akad. Nauk SSSR, Seriya geogr., 1959, No. 4.

70. Osnovy geokriologii (merzlotovedeniya). Ch. i, Obshchaya geokriologiya (Fundamentals of geocryology (permafrostology), Part 1, General geocryology). Moscow, Izd. Akad. Nauk SSSR, 1959.

71. Osnovy geokriologii (merzlotovedeniya). Ch. 2, Inzhenernaya geokriologiya (Fundamentals of geocryology (permafrostology), Part 2, Engineering geocryology). Moscow, Izd. Akad. Nauk SSSR, 1959.

72. Pavlov, A. V. Teploobmen promerzayushchikh i protaivayushchikh gruntov s atmosferoy (Heat exchange between freezing and thawing ground and the atmosphere). Moscow, "Nauka," 1965.

73. Popov, A. I. Merzlotnyye yavleniya v zemnoy kore (Frost phenomena in the earth's crust). Moscow, Izd. Moskov. Gos. Univ., 1967.

74. Porkhayev, G. V. Some data on the percolation coefficient of thawing ground. In: Issledovaniya po fizike i mekhanike merzlykh gruntov (Research on the physics and mechanics of frozen ground). Vyp. 4. Moscow, Izd. Akad. Nauk SSSR, 1961.

75. Puskov, V. I. The effects of artificial salt treatment on the thermophysical properties of soil materials. In: Materialy VIII Vsesoyuznogo mezhduvedomstvennogo soveshchaniya po merzlotovedeniyu (Proceedings of the Eighth All-Union Interdepartmental Conference on Permafrostology). Vyp. 4. Yakutsk, 1966.

76. Plotnikov, V. M., and A. I. Golyshev. The Ust'-Khantayka underground hydroelectric power station. Gidrotekhnicheskoye stroitel'stvo, 1971, No. 9.

77. Plotnikov, V. M., and A. G. Brusse. Disposal of flood waters of the Khantayka River during construction over the crest of the rock fill dam of the Ust'-Khantayka Hydroelectric Power Station. Gidrotekhnicheskoye stroitel'stvo, 1971, No. 11.

78. Pchelkin, N. A. Characteristics of water-diversion structures under severe climatic conditions. In: Trudy IV soveshchaniya-seminara po obmenu opytom stroitel'stva v surovykh klimaticheskikh usloviyakh (Transactions of the Fourth Seminar Meeting to Exchange Experience in Construction under Severe Climatic Conditions). Vorkuta, 1966.

79. Saltykov, N. I. Osnovaniya i fundamenty v rayonakh rasprostraneniya mnogoletnemerzlykh gruntov (Foundations in regions where the ground remains frozen for many years). Moscow, Izd. Akad. Nauk SSSR, 1959.

80. Sever Dal'nego Vostoka (The Far Eastern North). Moscow, Nauka, 1970.

81. Semenov, N. G. Dam on the Irelyskh River built on weak materials in a permafrost region. Trudy Lengidroproyekta, 1964, Vyp. 1.

81.a. SNiP II-B. 6-66. Osnovaniya i fundamenty zdaniy i sooruzheniy na vechnomerzlykh gruntakh. Normy proyektirovaniya (Foundations for buildings and structures on permafrost. Planning standards).

82. Solodkin, S. N. Problems of water supply for residential settlements and industrial enterprises in Magadan Oblast. In: Trudy soveshchaniya-seminara po obmenu opytom stroitel'stva v surovykh klimaticheskikh usloviyakh (Transactions of the Seminar Meeting to Exchange Experience in Construction under Severe Climatic Conditions). Magadan, 1964.

83. Srednyaya Sibir' (Central Siberia). "Nauka," Moscow, 1964.

84. Sloyev, L. N. Water diversion facilities of the lagoon irrigation system in central Yakutia. In: Trudy IV soveshchaniya-seminara po obmenu opytom stroitel'stva v surovykh klimaticheskikh usloviyakh (Transactions of the Fourth Seminar Meeting to Exchange Experience in Construction under Severe Climatic Conditions). Vorkuta, 1966.

85. Sizov, V. M. Stroitel'nyye raboty v zimnikh usloviyakh (Construction operations under winter conditions). Moscow, Gosstroyizdat, 1961.

86. Smit, Kh. R. Calcium chloride treatment to control frost effects. In: Merzlotnyye yavleniya v gruntakh (Frost phenomena in the ground). Moscow, Izd. inostr. lit., 1955.

87. Sumgin, M. I., S. P. Kochurin, N. I. Tolstikhin, and V. F. Tumel'. Obshcheye merztolovedeniye (General permafrostology). Moscow, Izd. Akad. Nauk SSSR, 1940.

88. Sumgin, M. I. Vechnaya merzlota pochvy v predelakh SSSR (Soil permafrost within the confines of the USSR). Izd. Akad. Nauk SSSR, 1937.

89. Timofeychuk, V. S. Deformations of earth dams of the unfrozen type on permafrost. Kolyma, 1960, No. 8.

90. Toropov, L. N. Construction of a dam with a frozen curtain on the Irelyakh River. Energeticheskoye stroitel'stvo, 1970, No. 9.

91. Trupak, N. G. Zamorazhivaniye gruntov v stroitel'stve (Ground freezing in construction). Moscow, Stroyizdat, 1970.

92. Trupak, N. G. Construction of earth dams on permafrost. Gidrotekhnicheskoye stroitel'stvo, 1970. No. 9.

93. Ukhanov, S. B. Artificial salt treatment of loams for construction in winter. Izv. vuzov, Stroitel'stvo i arkhitektura, 1959, No. 1.

94. Khakimov, Kh. R. Voprosy teorii i praktiki iskusstvennogo zamorazhivaniya gruntov (Theoretical and practical problems of ground freezing). Moscow, Izd. Akad. Nauk SSSR, 1957.

95. Khukhlayev, G. A. Construction of the earth-rockfill dam for the Serebryanskaya Hydroelectric Power Station above the Arctic Circle. Gidrotekhnicheskoye stroitel'stvo, 1969, No. 8.

96. Khrustalev, L. N. Percolation properties of the seasonally thawing loam materials in the Vorkuta region. In: Materialy k osnovam ucheniya o merzlykh zonakh zemnoy kory (Basic materials for the study of the frost-affected zones of the earth's crust). Vyp. VII. Moscow, Izd. Akad. Nauk SSSR, 1961.

97. Tsvetkova, S. G. Experience with the construction of dams in permafrost regions. Ibid. Vyp. VI. 1960.

98. Tsytovich, N. A. Contribution to the theory of the equilibrium state of water in frozen ground. Izv. Akad. Nauk SSSR, Ser. Geogr., 1945, No. 5-6.

99. Tsytovich, N. A. Mekhanika gruntov (Soil mechanics). Moscow, Stroyizdat, 1963.

100. Tsytovich, N. A. Osnovaniya i fundamenty na merzlykh gruntakh (Foundations on frozen ground). Moscow, Izd. Akad. Nauk SSSR, 1958.

101. Tsytovich, N. A., N. V. Ukhova, and S. B. Ukhov. Prognoz temperaturnoy ustoychivosti plotiny iz mestnykh materialov na vechnomerzlykh gruntov (Prediction of the temperature stability of a dam built of local materials on permafrost). Leningrad, Stroyizdat, 1972.

102. Tsytovich, N. A. Mekhanika merzlykh gruntov (Mechanics of frozen ground). Moscow, Vysshaya shkola, 1973.

103. Chernyshev, M. Ya. Vodosnabzheniye v vechnoy merzlote (Water supply in permafrost). Moscow, Transzheldorizdat, 1933.

104. Chistopol'skiy, S. D. Theory of the seepage-point method of thawing permafrost. Trudy VNII-I, 1961, Vol. XIX, Magadan.

105. Chistyakov, G. Ye. Vodnyye resursy rek Yakutii (The water resources of the rivers of Yakutia). Moscow, Nauka, 1964.

106. Engineering News Record, 1948, v. 140, No. 6, p. 70.

107. MacDonald, D. H. Kelsey Generating Station dam and dykes. Engineering Journals, October, 1960.

108. Johnston, G. H. Dykes on permafrost, Kelsey Generating Station, Manitoba. Ottawa, August, 1968.

TABLE OF CONTENTS