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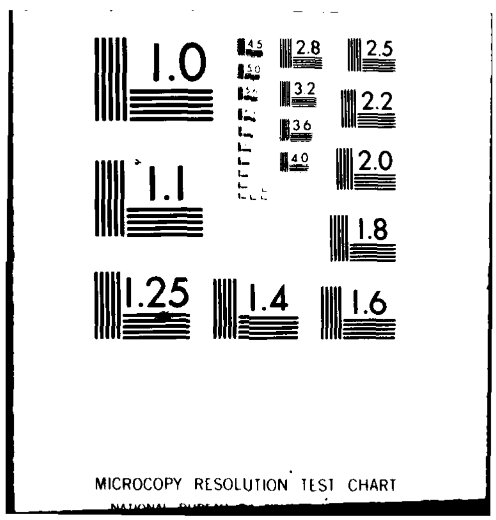
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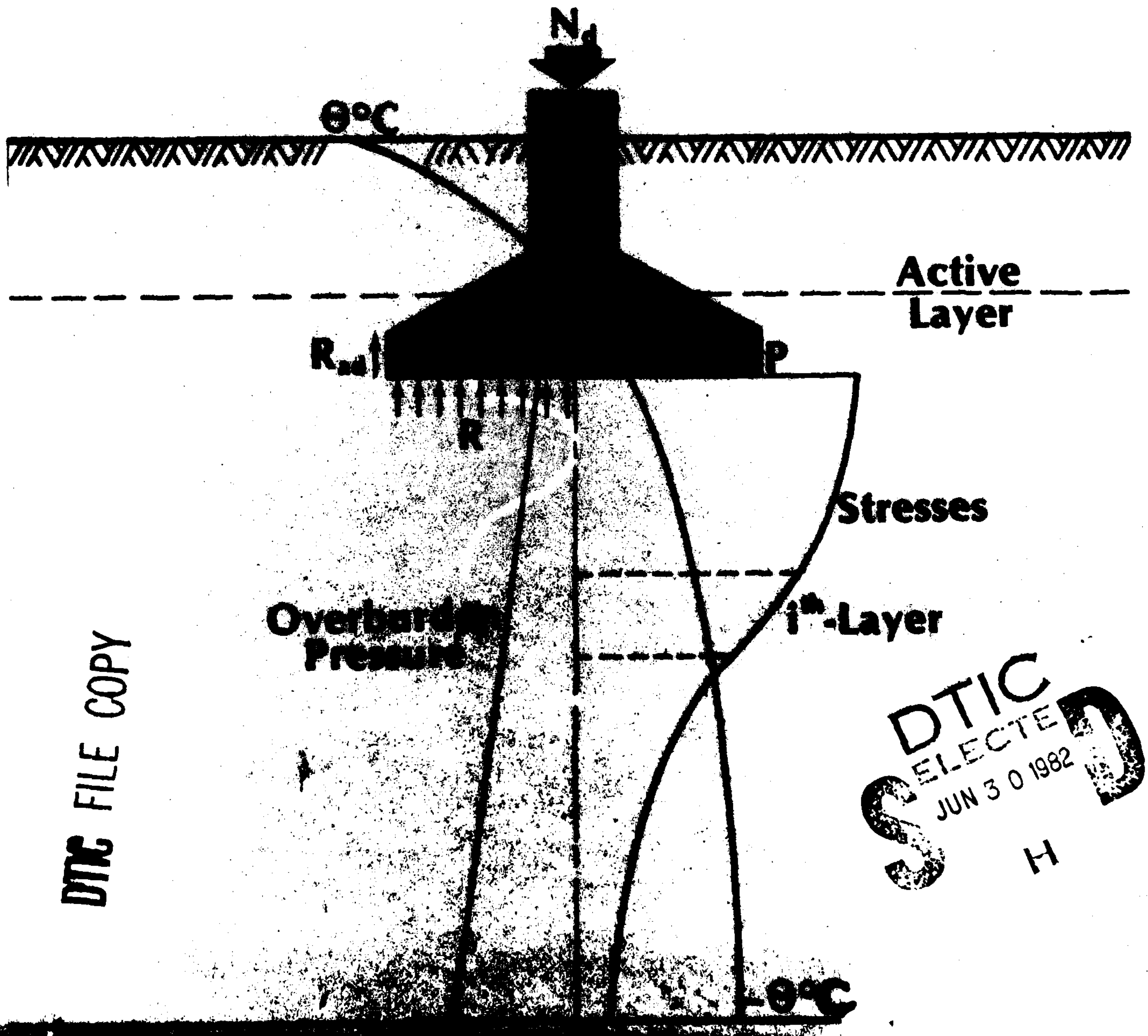
CRREL REPORT 82-14

US Army Corps of Engineers
Cold Regions Research & Engineering Laboratory

Comparative analysis of the USSR Construction Codes and the US Army Technical Manual for design of foundations on permafrost



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CRREL Report 82-14

May 1982

Comparative analysis of the USSR Construction Codes and the US Army Technical Manual for design of foundations on permafrost

Anatoly M. Fish

Prepared for
U.S. ARMY RESEARCH OFFICE

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A comparative study was made of design criteria and analytical methods for footings and pile foundations on permafrost employed in U.S.S.R. Design Code SNiP II-18-76 (1977) and U.S. Army Cold Regions Research and Engineering Laboratory Special Report 80-34, developed in the early 1970's by the U.S. Army Corps of Engineers and published in 1980. The absence of adequate constitutive equations for frozen soils and of rigorous solutions of the boundary problems has made it necessary to incorporate (explicitly or implicitly) various safety factors in the foundation analyses. From the review it is concluded that the principal difference between these practices is in the assessment and application of appropriate values of safety factors, which leads to a substantial discrepancy in the dimensions and cost of footings and pile foundations in permafrost.

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PREFACE

This report was prepared by Anatoly M. Fish, Research Associate, Massachusetts Institute of Technology. Funding was provided by contract no. DAAG 29-77-0016 between the Civil Engineering Department of the Massachusetts Institute of Technology and the U.S. Army Research Office.

This report was technically reviewed by T.C. Johnson, F.H. Sayles, W.F. Quinn and E.F. Lobacz, all from the U.S. Army Cold Regions Research and Engineering Laboratory. Special appreciation is extended to T.C. Johnson and Dr. A. Assur, without whose constructive participation this work could not have been written in such a form. Many colleagues at CRREL are also acknowledged for their careful editing and critical review of the manuscript.

CONTENTS

ILLUSTRATIONS

TABLE

CONVERSION FACTORS: U.S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

These conversion factors include all the significant digits given in the conversion tables in the ASTM Metric Practice Guide (E 380), which has been approved for use by the Department of Defense. Converted values should be rounded to have the same precision as the original (see E 380).

Multiply	*By*	*To obtain*
inch	0.0254*	metre
$inch^2$	0.00064516*	$metre^2$
foot	0.3048*	metre
pound	0.4535924	kilogram
pound-force	4.448222	newton
ton	0.02916667	kilogram
degrees Fahrenheit	$t_{°C} = (t_{°C} - 32)/1.8$	degrees Celsius

*Exact

COMPARATIVE ANALYSIS OF THE USSR CONSTRUCTION CODES AND THE US TECHNICAL MANUAL FOR DESIGN OF FOUNDATIONS ON PERMAFROST

Anatoly M. Fish

INTRODUCTION

This paper is principally a discussion, from the viewpoint of frozen soil mechanics, of foundation analytical and design methods employed in the U.S.S.R. Design Code SNiP II-18-76[3] (1977) and U.S. Army Cold Regions Research and Engineering Laboratory Special Report (SR) 80-34[22] (1980)†. Thermophysical problems of frozen soils as well as the design of foundations on thawing soils are not considered in this review. In the present work, the author does not pretend to undertake a detailed and comprehensive critical review of the theoretical bases of analyses used in the U.S. and U.S.S.R. design practices. It is obvious that any accepted theory or analytical method, no less than any standard test procedure for determining soil properties, has both positive and negative features, and can be the subject of separate discussion. The author participated in the preparation of the former edition of the SNiP[4], a Design Manual[5] and other documents. As a consequence of this familiarity with the details of the development of standards governing Soviet design practice, some of the commentary will extend beyond a mere exposition of the contents of the current version of the SNiP. The purpose of the author's review is to identify both the strong and weak aspects of the latest edition of the Soviet SNiP and of SR 80-34 as a whole, and to discuss needed research to make design of foundations on permafrost more reliable and economical.

U.S.S.R. SYSTEM OF STANDARDS

In the U.S.S.R. all design and construction of buildings and structures, and also the testing of construction materials and soils, are carried out according to the requirements contained in the following documents:

1. Construction Standards and Codes (SNiPs).
2. Instructions, Recommendations, Guidelines and Manuals for design, analysis, and construction of specific types of projects.
3. State Standards (GOSTs) on the requirements as to the quality of construction materials, equipment, methods and procedures of tests, etc.

These documents are prepared by the leading design and research institutes and are called "standard documents." Official editions of SNiPs, GOSTs, etc. are issued by the government departments and have the force of law; compliance is obligatory. Minor variance is permitted only with special written permission of the institute that prepared the document, and only in those cases where there is sound technical and economic justification.

Publication of the standard documents is planned and financed by the State Committee for Construction (GOSSTROY), and by various ministries and government departments. The main standard documents on construction are SNiPs. There are about 200 chapters of SNiP and hundreds of the items mentioned in 2 and 3 above. According to existing practice the SNiPs are revised about once every 10 years. Materials which for some reason were not included in a chapter or were developed during the interval between two editions are published in Guidelines, Instructions, or Recommendations. Some of these are issued by GOSSTROY or the other government departments, and serve as a supplement to the appropriate chapter of SNiP. Besides these there are many semiofficial documents such as Guidances and Handbooks. As a rule, after publication of a revised document the previous one loses its legal status. Therefore, each year GOSSTROY publishes a special list of the documents that are currently in effect[1,2].

†Special Report 80-34 was prepared with the final objective of publication as an official engineering manual: Department of the Army Technical Manual TM 5-852-4 and Department of the Air Force Manual 88-19, Chapter 4, Arctic and Subarctic Construction, Foundations for Structures.

U.S.S.R. DESIGN CODE SNiP II-18-76 (1977) SUBSOILS AND FOUNDATIONS ON PERMAFROST

The main standard document on design of foundations on permafrost is SNiP II-18-76[3], which is prepared by NIIOSP (Scientific Research Institute of Foundations and Underground Structures), Moscow, with the participation of design and research institutes such as LENZNIIEP (Leningrad), FUNDAMENTPROYEKT (Moscow), TSNIIIS (Moscow) and PROMSTROYPROYEKT (Krasnoyarsk). The current version went into effect on 1 January 1978. The chapter was developed mainly on the basis of six references[4-9] and covers foundation design for civil and industrial buildings. It does not cover design of special structures such as roads, railroads, bridges (except some specifics of foundation design), tunnels, or dams. The requirements of this chapter also do not apply to subsoil and foundation design for hydrotechnical structures, airfield pavements, machines with dynamic loads, and buildings and structures in areas of subsidence caused by coal mining, etc. Design of these structures is covered by the regulations in special chapters of the SNiP[10].

SNiP II-18-76 consists of the following sections:

1. General regulations.
2. Classification of soils.
3. Basic regulations of foundation design.
4. Analysis of subsoils and foundations.
5. Specifics of design of foundations on ice-rich soils and ground ice.
6. Specifics of design of foundations on saline soils.
7. Specifics of design of foundations on frozen organic soils and peat.
8. Specifics of design of foundations in seismic permafrost regions.
9. Specifics of design of foundations for bridges and culverts.

Appendix 1. Design values of thermophysical characteristics of unfrozen and frozen soils.
Appendix 2. Temperature regime of winter-ventilated crawl spaces.
Appendix 3. Depth of thawed ground under buildings and structures.
Appendix 4. Depth of seasonal freeze-thaw of soil.
Appendix 5. Analysis of stability and strength of foundations on frost-heaving soil.
Appendix 6. Design values of strength characteristics of frozen soils.
Appendix 7. Analysis of foundation settlements on ice-rich soils and ground ice.

Since the American reader is not familiar with their content, the various sections will be considered briefly.

General regulations

In this section the following basic requirements are formulated:

1. Scope of SNiP II-18-76.
2. Meeting the State Standards and Codes.
3. Scope of geocryological surveys, the amount of in situ data needed and laboratory testing of soils.
4. Monitoring of subsoil conditions during construction and use of a structure.

Only general requirements for the scope of engineering surveys of frozen soils are included in this SNiP. Their scope corresponds approximately to Figure 4-1 of SR 80-34[22]. More detailed information on this question is contained in State Standards and Special Instructions[10,11].

The most important regulations of the section concern the following:

1. Design of structures, taking into account possible changes in the temperature-moisture regime of subsoils during the period of their use.
2. Compilation (before construction begins) of special programs of observations of foundation and subsoil conditions during construction and use of the structure.
3. Development of measures for environmental protection.

One can conclude that meeting the first two regulations involves conventional cold regions engineering practice and causes no extraordinary difficulties†. But meeting the third would require development of a method for estimating the *economic* consequences of changes in the temperature regime of the frozen ground. Special technical solutions, necessitating scientific research in some cases, would be required to find ways to localize and limit these changes. Nowadays such solutions can be found with confidence only in cases where the frozen state of the subsoil is preserved. For thawing soils such solutions have not been developed. It is quite obvious that attempts to satisfy this requirement in the absence of an adequate technological base might lead to very serious deficiencies in design. Apparently, this requirement was included in the SNiP mainly to call the attention of designers to the problem of environmental protection.

Classification of soils

The basic classification of frozen soils according

†Shaded text is author's comments on either the SNiP or the SR and may contain information not found in either document.

to their granulometric composition adopted in the SNiP does not differ substantially from that of unfrozen soils[9]. However, frozen soils are further classified according to:

1. Cryogenic structure.
2. Degree of ice-cementing and rheological properties.
3. Ice content.
4. Organic content.
5. Salt content.

Frozen soils are subdivided according to cryogenic structure into two categories: "massive" (fused) structure and reticulate-layered (laminar-cellular) structure.

According to the degree of ice-cementing and rheological properties, frozen soils are divided into three groups:

1. *Solidly frozen*—soils cemented by ice with a compressibility coefficient of $a < 0.001$ cm^2/kgf and various temperatures (depending on granulometric composition) lower than -1.5°C.
2. *Plastic frozen*†—sandy and clay soils cemented by ice with $a > 0.001$ cm^2/kgf, degree of saturation $G > 0.8$ and with temperature (depending on granulometric composition) from 0 to -1.5°C.
3. *Loosely frozen*—coarse-grained soils and sands not cemented by ice, with water content $w < 0.03$.

Special design provisions apply if the frozen soil belongs to one of the following classes:

1. *Ice-rich soils*—when the fraction of ice inclusions $i_i > 0.4$.
2. *Peaty frozen soils*—when the organic content $g \geq 3\%$ of the mineral parts for sandy soils and 5% for clayey soils.
3. *Saline frozen soils*—when the salt content (depending upon granulometric composition) is greater than 10% of the dry soil weight.

Peaty and saline frozen soils can also be classified as solidly frozen or plastic frozen soils, depending upon their temperature and organic and salt content. The importance of such a classification will be shown later; here we merely note that the classification is the basis for selection of 1) the scope of field and laboratory tests of soil, 2) the effective types of foundations, 3) the methods of foundation analysis.

Relationships[12] among the simplest physical characteristics of frozen soils and some of their properties are also given in this section.

†Plastic is a conventional term used in the SNiP to refer to frozen soils that undergo substantial deformation under moderate loads.

Basic regulations for foundation design

This section contains recommendations for selecting the method of using frozen ground as a subsoil for structures and technical instructions for foundation design, depending on the construction method chosen.

Two principles of design for building on frozen ground are stated:

I. Preservation of the frozen state of the subsoil.
II. Preconstruction thawing of the subsoil or allowing subsoil thawing during construction or use of a structure.

For subsoils that are kept in the frozen state (principle I), recommendations are made for preserving the subsoil thermal regime or decreasing the temperature of plastic frozen soils by providing winter-ventilated air (crawl) spaces, pipes and channels, thermopiles, etc. Precast concrete pile foundations are proposed as the main type of foundation for construction under principle I. Methods for pile installation are selected according to the type of frozen soil.

Thaw prior to, during, or after construction is permitted either when the building is founded on bedrock or when deformation of thawing ground does not exceed the values given by SNiP II-15-74[9]. To reduce the expected settlement or damage due to thawing of soils the following measures are recommended:

1. Improvements in the construction properties of soils by preconstruction thaw, compaction and stabilization of thawing soils, etc.
2. Increasing the general rigidity of a structure to permit only uniform settlements as a unit, or increasing the flexibility of a structure to permit deformations without structural damage.

It is emphasized that the principle (basis of design) must be selected from technical and economic comparison of designs, considerations of the future serviceability of the structures if large deformations are expected, and measures connected with site work, grading, drainage, and protection of the environment.

Analysis of subsoils and foundations

In general, the analysis of subsoils and foundations involves separate consideration of two topics: bearing capacity of footings and piles, and foundation settlements.

Design criteria

Subsoil analyses and foundation designs are based on various assumptions. Safety factors are used to decrease the uncertainty of the analyses. SNiP II-18-76 does not include a separate section on safety factors in design. Information on this question is scattered in various SNiPs. However, since the construction cost depends substantially upon the safety factors that are chosen, this problem is worthy of discussion in the framework of the present analysis.

One of the most important requirements of SNiP II-18-76[3], compared with the previous edition[4], is the increased reliability (more conservative design) of structures on frozen ground that is achieved by incorporating the following safety factors:

1. Overload safety coefficients.
2. Soil safety factors.
3. Reliability coefficients.

Overload safety coefficients are used in analyzing the bearing capacity, stability, and settlements of foundations. Their absolute values, which are tabulated in a special chapter of the SNiP[13], depend on the statistical probability of the coincidence of various live and dead loads.

Soil safety factors are selected on the basis of statistical analysis of the results of field and laboratory soil tests. In all analyses of subsoils, the *design* values of soil characteristics A are used, which are determined by the formula

$$A = \frac{A_n}{k_s} \tag{1}$$

where A_n is a *nominal*† soil characteristic, i.e., cohesion c_n, a friction angle ϕ_n, bulk unit weight γ_n, etc.; k_s is a soil safety factor.

Soil safety factors are selected in accordance with 1) the variability of the characteristics of the soil, 2) the number of tests, and 3) the applicable value of the probability confidence limit, which is 0.85 in analysis of bearing capacity and deformations, 0.9 for foundations of bridges and 0.99 in certain special cases.

Reliability coefficients, standardized by the SNiP, are values established for each design principle, category (class) of structure, and method of foundation analysis. They are used for analysis of foundation bearing capacity or stability with respect to frost heave, but not for settlement analysis.

The introduction of these safety factors is the most important change in the new SNiP, making it possible to apply statistics in foundation design. Thus, according to SNiP II-18-76, analysis of bearing capacity consists of satisfying the criterion

$$N_d < Q^* \tag{2}$$

where $N_d = N_n \cdot k_{ov}$ = design load
N_n = nominal load: dead and live loads
k_{ov} = overload safety coefficient
$Q^* = Q/k_r$ = design (allowable) bearing capacity
Q = ultimate (nominal) bearing capacity of subsoil
k_r = reliability coefficient.

The value of k_{ov} generally varies from 1 to 2, but it can be 0.9, for example, in the analysis of foundation stability with respect to frost heave. The values of k_r are between 1.1 and 1.2 for all types of foundations except those for bridges. For bridge pile foundations k_r varies from 1.4 to 1.75, depending upon the number of piles in the foundation.

Bearing capacity analyses are performed for all types of frozen soils, including solidly frozen, loosely frozen, and plastic frozen soils, ice-rich soils, and ground ice.

Analysis of deformations consists of satisfying two criteria:

1. Average design bearing pressure p under the foundation must not exceed the allowable bearing pressure q^*, i.e.,

$$p \leq q^* \tag{3}$$

where $p = N_d/F$, $q^* = Q^*/F$, and F is the foundation area.

2. Foundation deformation S (total settlement, tilt, differential movement, etc.) must not exceed the permissible ultimate value S^*, i.e.,

$$S \leq S^* . \tag{4}$$

Settlement analyses are performed only for foundations on plastic frozen soils, ice-rich soils and ground ice. The accepted values of S^* are the same as the maximum deformations customarily allowed for structures on unfrozen soils[9] without any alteration to account for the severe climatic conditions of northern regions.

The SNiP requires the use of soil safety factors in all types of foundation analyses and the use of reliability coefficients in analyses of bearing capacities and stabilities. The SNiP does not require that a special safety coefficient be applied in the analysis of foundation settlements.††

In considering the possible need for such a safety factor, it is noted that numerous data on damaging settlements and, in some cases, failure of structures

†A nominal characteristic, according to the SNiP's terminology, means an average value of a property obtained from field or laboratory tests.

††However, it will be shown that the analytical methods of the SNiP (and SR80-34 as well) contain certain safety factors implicitly.

have been accumulated[12]. In many cases the consequences of foundation settlements have been so severe that the cost of reconstruction far exceeded the initial construction cost. It appears that the analysis of foundation settlements should take into account the economic consequences of excessive settlements, which may entail large costs for repair and in extreme cases complete reconstruction. It would be very useful to introduce an economic safety factor in the analysis of settlements. Thus eq 4 would become

$$S < \frac{S^*}{k_{ec}} \qquad (5)$$

where $k_{ec} > 1$ = an economic safety factor.

Such a safety factor could consist of a number of coefficients ($k_{ec} = k_1 \times k_2 \times \ldots\ldots k_n$), each reflecting the dependence of the repair or reconstruction cost upon the consequences of large deformations of structures on permafrost. They could be tabulated according to the type of structure, the regional and climatic factors affecting the costs of construction materials, labor, transportation, etc. As construction experience accumulates, the coefficients could be revised as necessary.

It is well known that selecting the most economical type of foundation is a complicated problem, requiring a comparison of various foundation types and detailed cost estimates. Introduction of an economic safety factor would simplify the design procedure, particularly in the case of thawing soils.

Bearing capacity analysis of footings and pile foundations

The nominal bearing capacity Q of centrically loaded footings or single piles is estimated by the following formula:

$$Q = k_{uc}\left(RF + \sum_{i=1}^{m} R_{ad(i)}\,F_{(i)}\right) \qquad (6)$$

where

k_{uc} = uniformity (nonhomogeneity) coefficient = 1.4, depending upon soil temperature, type of foundation, etc.

F = cross-sectional area of pile or footing

$F_{(i)}$ = shear area

R = long term resistance of frozen soils to normal pressure

$R_{ad(i)}$ = frozen soil long term adfreezing strength

m = number of soil layers.

The bearing capacity of a footing on frozen soils depends mainly on the soil resistance R to the applied normal pressure. In lieu of the tabulated values, R can also be obtained from laboratory tests. In the latter case the R values are calculated by the Berezantsev solution for a square foundation on an ideally cohesive medium (angle of internal friction $\phi = 0$):

$$R = 5.7c + \gamma h \qquad (7)$$

where

c = design value of the long term cohesive strength, determined from unconfined compression tests (assuming after Mohr that $c \approx 0.5\,\sigma$, where σ is the long term compressive strength), or by the ball plunger testing method

γ = design bulk unit weight of the frozen soil

h = foundation depth.

The solution is based on the assumed complete formation of a zone of plastic equilibrium under the foundation.

The design values of R and $R_{ad(i)}$ tabulated in the SNiP depend on the soil type, temperature, and salt, peat, and ice contents. In addition, the tabulated values of $R_{ad(i)}$ depend on the type of adfreezing surface. The subsoil temperature is estimated by conducting a thermal analysis. Formulas and the thermophysical characteristics of soil necessary for such an analysis are given in the SNiP. For short term and repeated loading, the tabulated values of R are increased by up to 100%, depending on the duration of the load. For repeated loads of 24 hr or more in duration, the correction factor is 1.05.

The SNiP proposes a very interesting method of temperature corrections for determining pile foundation bearing capacity from the results of field experiments. The nominal bearing capacity Q of a single pile is determined by the formula:

$$Q = k_t\,\frac{P_n}{k_s} \qquad (8)$$

where

P_n = nominal bearing capacity of a single pile obtained from field tests

k_s = soil safety factor

$k_t = Q_d/Q_{ex}$ = temperature correction coefficient.

Q_d and Q_{ex} are theoretical values of pile bearing capacity estimated by eq 6 for the design temperature of the structure subsoil (Q_d) and for the test temperature (Q_{ex}).

For instance, if the tests were performed in the period of maximum ground temperature (summer) and the design temperature of the structure's subsoil, determined by the thermal analysis, was lower, the bearing capacity of the pile P_n obtained from the field tests can be raised. The reverse is also true. The method for determining P_n does not differ substantially from that given in SR 80-34.

The bearing capacity of piles in permafrost in the case of partial thawing of frozen ground is determined by taking into account the negative friction of the soils caused by settlement of the thawing layer (provided drainage takes place). The magnitudes of friction forces are taken to be equal to those of unfrozen soils[9].

One concludes that the following are basic premises of the SNiP in regard to bearing capacity analysis:

1. **Frozen soil is considered as an ideally cohesive medium ($c \neq o$ and $\phi = o$).**
2. **Frozen soils are assumed to possess long term strength whose value differs insignificantly from their 24-hr strength.**

In reference to point 1, it is not clear why, for the same soil, the friction angle need not be taken into account when the soil is frozen, even though, according to the SNiP[9], analysis of the thawed or thawing soil bearing capacity must take into account both cohesion and friction of the soil, no matter how small they are. The assumption that the frozen soil's 24-hr strength differs only slightly from the long term strength is not confirmed by the experimental data[14]. Apparently this simplifying assumption is introduced in an attempt to facilitate the design and the test procedure.

From an engineering viewpoint such assumptions are possible. However, the requirements of the SNiP concerning the selection of the most economical design are not strictly fulfilled by this procedure.

Settlement analysis of foundations on plastic frozen soils

As noted above the SNiP requires that settlements be estimated only for foundations on *plastic frozen* soils, tacitly implying that deformation of *solidly frozen soils* may be neglected.

The analysis of settlements of footings on plastic frozen soils can be divided into two stages: determination of the allowable bearing pressure on the subsoil q^*, and calculation of foundation settlement S.

The allowable bearing pressure on the subsoil is determined by an analysis of bearing capacity. Settlements of footings are calculated[9] by the commonly known semi-empirical method of summing the vertical "elastic" deformation of individual layers of the subsoil along the center line of the foundation (Fig. 1). The latter are calculated from the solutions of the boundary problem of the linear theory of elasticity. In the general case the ultimate foundation settlement is estimated by the formula:

$$S = \beta \sum_{i=1}^{m} \frac{p_i h_i}{E_i} \qquad (9)$$

where

$p_i = \omega_i(p - p_o)$, an average pressure in the i^{th} layer of the subsoil along the central axis of a foundation

p = total average design unit load under the foundation

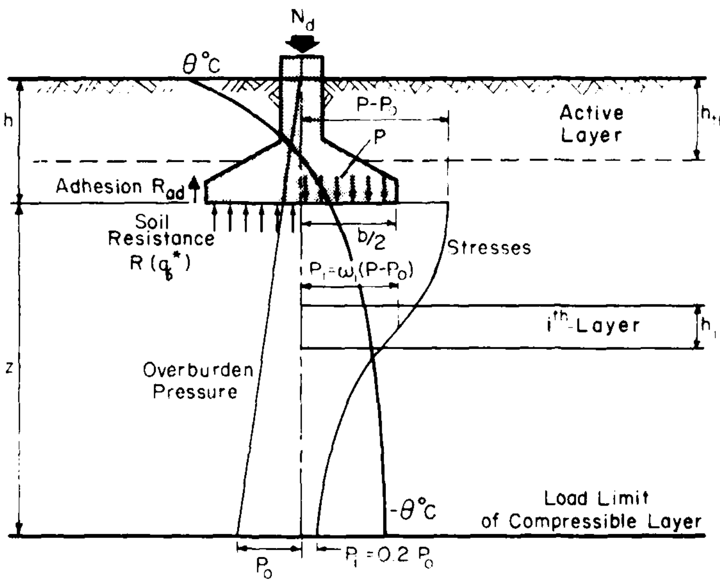


Figure 1. Diagram for analysis of foundations.

$p_o = \gamma h$, overburden pressure

ω_i = stress distribution coefficient in the i^{th} layer of the subsoil depending upon the depth of the layer and the foundation shape, etc.

h_i = height of the i^{th} layer

$\beta = 0.8$, a dimensionless coefficient

E_i = deformation modulus of i^{th} layer

m = number of layers.

Either the deformation modulus E_i, determined in situ, or the laboratory determined compressibility coefficient a_i for each layer is used in the analysis, taking into account neither the time factor nor the annual temperature change of frozen soil. The SNiP also allows the settlements to be calculated on the basis of the solution of the nonlinear problem.

Such an approach to the problem of the analysis of foundation settlements gives rise to the following questions:

1. **From the viewpoint of the mechanics of solids, there is no difference between solidly frozen and plastic frozen soils. Both deform if the load exceeds a certain level. Therefore, it is not clear why plastic frozen soil is considered to be deformable (compressible) and solidly frozen soil not deformable, since both eventually fail, accompanied by large deformations.**
2. **It is extremely difficult to maintain the temperature regime of a subsoil when it is close to 0°C, thereby fitting the definition of frozen soil. It is probable that a plastic frozen soil will change either to the unfrozen or to the solidly frozen state. For solidly frozen soil, no settlement analysis is required[3]. For plastic frozen soil, the SNiP governing frozen soil requires that a settlement analysis be carried out in accordance with the SNiP governing unfrozen soil[9], but the methods stipulated in the latter fail to account for the specifics of frozen soil.**
3. **It is not clear how to determine the soil deformation moduli E_i for different layers from protracted field plate tests while maintaining the temperature regime at various depths. Determination of the compressibility coefficients a_i also engenders considerable difficulties.**

The SNiP recommends determining pile foundation settlement by using data from field tests of piles under static loads. However, the SNiP does not recommend a specific method of calculation.

Settlement analysis of foundations on ground ice and ice-rich soils

This section is the most important accomplishment of the SNiP. For the first time in construction practice an official regulation allows the construction of large buildings and structures on subsoils containing ice-rich soils and ground ice.

Settlements of foundations on ground ice and ice-rich soils are estimated by the formula

$$S = S_1 + S_3 \tag{10}$$

where

S_1 = immediate settlement due to the soil (ice) consolidation as a result of changes in its air porosity

S_3 = settlement due to the viscous flow of ice.

It is assumed that settlement S_1 occurs rapidly after application of the load. Its absolute value is insignificant; however, the formula for estimation of S_1 is given in SNiP. The settlement S_3 is estimated on the basis of an approximate solution of the boundary problem of a nonlinear visco-elastic half space with a variable temperature regime[6,16] i.e.,

$$S_3(t) = \frac{1}{2}\,(p-p_o)^n\; bt\,K \times \sum_{i=1}^{m} (k_{\theta i} + k_{\theta i+1})(\omega_{i+1} - \omega_i) \tag{11}$$

where

p = total average design unit load under the foundation

p_o = overburden pressure

n = strain hardening parameter

b = foundation width

t = duration of use of structure (structure's lifetime)

K = parameter of deformability, the reciprocal of the "viscosity" coefficient of ice determined by the linear segment of the rheological curve

$k_{\theta i}, k_{\theta i+1}$ = temperature distribution coefficients in the i^{th} layer of the subsoil, depending upon the subsoil temperature and the depth of the layer

ω_i, ω_{i+1} = stress distribution coefficient in the i^{th} layer of the subsoil, depending upon the foundation shape, the layer depth and the value of parameter n

m = number of layers.

Graphs of the values of $k_{\theta i}$, $k_{\theta i+1}$, ω; and ω_{i+1} are given in two works[6,16]. For the case when the parameter $n = 1$, these values are also given in tables in the SNiP[3]. Equation 11 is recommended by SNiP 11-18-76 only for settlement analysis of foundations on ground ice, but it can also be applied to settlement analyses for foundations on ice-rich soils[6].

The method of settlement analysis for foundations on ice-rich soils employed in the current issue of the SNiP is based on the same assumptions as those stated above for ground ice, but in somewhat simplified form. The SNiP gives the formulas for determining the design allowable load q^* on ice-rich soils and ground ice, depending on the foundation shape, the ratio of the foundation's sides, and the temperature.

Note that the settlements computed by the provisions of the SNiP will be substantially overestimated; the reasons for this will be examined further. It is very convenient for the following consideration to neglect the variation of temperature and mechanical properties of the subsoil with depth, as well as the temperature change with time, and represent eq 11 in the simple form[16]:

$$S_3(t) = (p-p_o)^n \, b \, t \, \frac{K\omega}{1+|\theta|}. \tag{12}$$

Equations 11 and 12 make it possible to estimate the degree or the rate of settlement at any time during use of the structure. However, the solution has one serious defect: difficulties in determining the deformability parameter K. An error in its determination may strongly affect the results of the settlement estimation.

Equations 11 and 12 were obtained on the basis of an approximate solution of the nonlinear elastic theory problem using the ice flow law in the secondary creep stage[16,18]

$$\dot{\epsilon} = \frac{K'\sigma^n}{(1+|\theta|)^\alpha} \tag{13}$$

where

$\dot{\epsilon}$ = creep strain rate in uniaxial compression

σ = applied compressive stress

n = strain hardening parameter

α = empirical parameter accepted to be unity

θ = temperature in °C without the sign

$K' = 3^{-[(n+1)/2]} K$

K = deformability parameter.

The Soviet Manual[17] recommends determining the deformability parameter K and the strain hardening parameter n from short term uniaxial compression creep tests. It is assumed that the primary creep stage ends if the increments of deformation do not exceed 0.005 mm/hr (Fig. 2). In practice, this condition can be reached within several hours after the start of the test. On the other hand, according to data of Votyakov[19] the settlement rates of experimental plates on ice were not constant, even after the first four months following loading. Experiments[18] show that the primary creep stage of ice continues up to one year and probably longer.

In the general case, ice flow in the primary creep stage may be approximated[16,18,20] by the equation

$$\dot{\epsilon}\,(t) = \frac{\lambda K'\sigma^n t^{-\delta}}{(1+|\theta|)^\alpha} \tag{14}$$

where λ = time hardening parameter ($\delta = 1 - \lambda$).

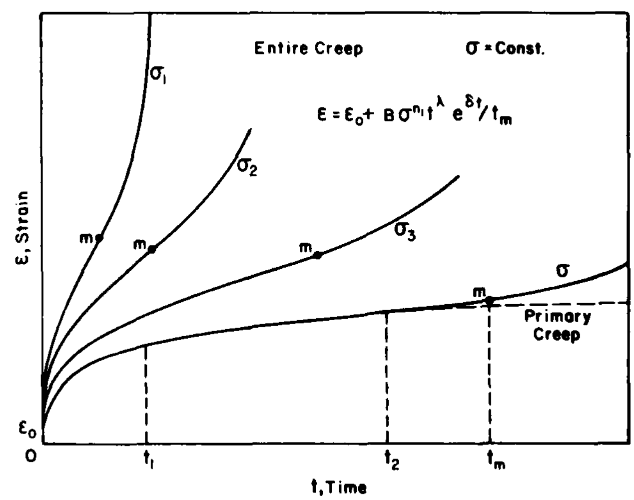


Figure 2. Creep curves of frozen soil at constant stresses (m—inflection [failure] points, t_m—time to failure).

Comparing eq 13 and 14, one can see that if the deformability parameter K' is determined from short term tests by eq 13 at time t_1 on the primary creep stage, and if the secondary creep stage begins at time t_2, the possible relative error N in the estimation of the settlement using eq 11 or 12 will be

$$N = \frac{S_{31}}{S_{32}} = \frac{K'_1}{K'_2} = \left(\frac{t_2}{t_1}\right)^{\delta} \tag{15}$$

where S_{31} and S_{32} are the settlements due to viscous flow of ice estimated using the deformability coefficients K'_1 and K'_2 in eq 13 at times t_1 and t_2 respectively[20].

For example, let us assume that the deformability coefficient K'_1 had been determined at time $t_1 = 24$ hr and the secondary creep stage starts at time $t_2 = 120$ days after initiation of the test. If the settlement were estimated using the parameter K'_1, its absolute value would be overestimated (according to eq 15 with $\delta = 0.3$) by a factor $N \approx 4$, compared with the settlement estimated using K'_2 determined for the time t_2. Figure 3 gives the possible settlement estimation error N versus the ratio of t_2 and t_1 at different values of δ. If it is assumed that the secondary stage starts at $t_2 = 365$ days or one year, the error N may range up to 20, depending upon the value of δ.

The Manual[17] requires that the deformability (viscosity) coefficient of ice be determined by the "linear" segment of the rheological curve. However, it is well known that strain rate is an approximately linear function of stress only for small stresses. This recommended procedure is reasonably valid only if the load on the ice surface does not exceed $\sigma \approx 1$ kgf/cm^2, since under higher loads, especially in the upper layers of subsoils, the relationship between strain rate and applied stress can be nonlinear. Nevertheless, the SNiP, using the linear law, allows application of a design stress considerably larger than 1 kgf/cm^2. Apparently, this makes it possible to decrease, to a certain extent, the absolute value of calculated settlement and the possible error respectively.

The additional safety factors inherent in the SNiP method, which are implied by the overestimation of settlements, may be explained by the fact that this is the first time an analytical method of estimating settlement of foundations on ground ice with time, taking into consideration seasonal changes of ground temperature and variation of temperature with depth, was developed and included in the official design codes.

A better approximation of the experimental data might be obtained by assuming that the total foundation settlement with time $S(t)$ consists (Fig. 4) of the sum of three components:

$$S(t) = S_1 + S_2(t_1) + S_3(t-t_1) \quad \text{for } t > t_1 \tag{16}$$

where

t_1 = duration of the primary creep stage

t = structure's lifetime.

Settlements S_1 and S_3 can be estimated by the provisions of the SNiP. Foundation settlements during the primary creep stage S_2 can be estimated by the following formula†

$$S_2(t_1) = \frac{1}{2}\,(p-p_o)^n\, b\, K\, t_1^{\lambda} \times \sum_{i=1}^{m} (k_{\theta i} + k_{\theta i+1})\,(\omega_{i+1} - \omega_i) \tag{17}$$

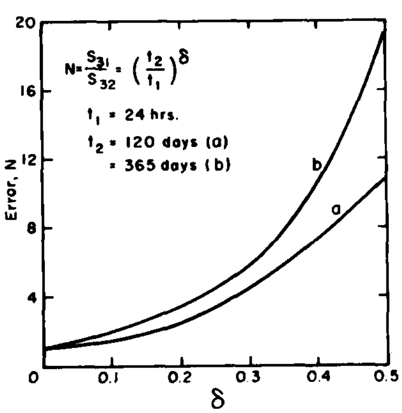


Figure 3. Possible errors N *in determining foundation settlements.*

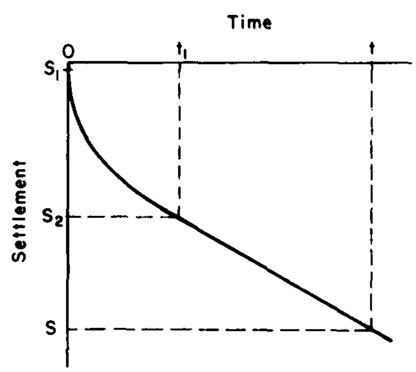


Figure 4. Diagram for determination of foundation settlements.

† It is assumed the parameters n and λ do not change with depth.

Equation 17, which was obtained[20] by the generalization of Sheynkman's solution (eq 11), makes it possible to take into account:

1. **Change of the strain rate during the primary creep stage**
2. **Change of the subsoil temperature with depth and with time**
3. **Nonlinearity of the stress-strain relationship.**

If it is assumed that the subsoil temperature is constant, eq 17 takes the form

$$S_2(t_1) = (p-p_o)^n\, b\, \frac{K\omega}{1+|\theta|}\, t_1^{\lambda}\,. \qquad (18)$$

The analysis of Votyakov's laboratory data[20] showed that the settlement rates of bearing plates on natural ground ice are in agreement with the theoretical values estimated by eq 18. The rheological parameters of ice obtained from in situ tests by means of a pressuremeter were used in this analysis. As it will be shown below, under the consideration of SR 80-34, eq 18 can also be applied to the estimation of foundation settlements on all types of frozen soils

Design of foundations for special soil conditions, and appendices

The final sections of the SNiP are devoted to recommended design methods for foundations on saline and peaty frozen soils, for foundations in seismic regions, and for foundations on thawing soils. The SNiP gives numerous data on the design resistance of frozen saline and peaty soils to normal pressure and shear along adfreezing surfaces, depending on the soil type, the temperature, and the mineral, salt, and peat content. Analysis of subsoils is performed by the method discussed above.

The coefficients of the decrease in bearing capacity for footings and pile foundations in seismic regions, depending on the type and temperature of the frozen soil and on the region's seismic activity, are of great interest. However, the basic requirements for foundation design in seismic regions are not included in SNiP II-18-76, but in other SNiPs.

Foundation stability and strength under the action of frost heave forces are also analyzed by known methods[12]. The design values of frost heave forces given in the SNiP depend upon the type of soil, the degree of water saturation, and the depth of seasonal freezing and thawing. The appendices contain formulas for thermal analyses of subsoils and numerous tables of thermal soil characteristics.

Analysis of foundations on thawing soils is performed by known empirical consolidation formulas[12]. In the general case, the settlement resulting from thawing of the subsoil will consist of the sum of two parts: that caused by thawing of excess ice or phase transitions and that caused by soil consolidation. The SNiP recommends determining coefficients of soil consolidation and thaw from field tests using heated loading plates or from laboratory tests using oedometers. The SNiP requires that the allowable load on the subsoil surface be determined in the same manner as for unfrozen soil[9].

For tentative estimates, settlements of foundations on thawing soils are allowed to be determined using simple empirical formulas and customary physical characteristics of frozen soils. However, errors in such estimations may be 100% or more[15]. For estimates of the depth of thawing in frozen ground under structures, and also the movement of thawing boundaries with time, special tables and graphs are given in the SNiP, depending on the ground temperature and simple physical characteristics of the soils. The current edition of the SNiP gives a certain latitude to the designer in selecting an analytical method of calculating deformations of thawing soils.

Thus, one can conclude that, in spite of its deficiencies, the latest edition of SNiP II-18-76 is undoubtedly the most substantial standard document on design of foundations on permafrost published in the U.S.S.R. in the last 10 years.

SR 80-34 (1980) DESIGN AND CONSTRUCTION OF FOUNDATIONS IN AREAS OF DEEP SEASONAL FROST AND PERMAFROST[22]

General information

This report contains a large amount of information on various aspects of design and construction in arctic and subarctic regions, from environmental considerations through inspection and monitoring. It comprises the following chapters:

1. Introduction
2. Basic considerations affecting foundation design
3. Site investigations
4. Foundation design
5. Survey data points
6. Construction considerations
7. Monitoring performance.

Strictly speaking, SR 80-34 cannot be compared with SNiP II-18-76 as to either its content or its form. Most of the items discussed in SR 80-34, except "foundation design," are scattered throughout different sections of the SNiP[1,10] and numerous semi-official documents. Some topics, including numerous design examples, have never been included in Soviet standard literature.

Special Report 80-34 provides guidance to the engineer, but since it does not prescribe a specific course of action, it leaves the *engineer* responsible for any design decisions. For instance, the report abounds with references to other publications; however, it is well known that authors sometimes elucidate the same questions differently. Such references expose an engineer to the constantly changing state of the construction art and the latest achievements of science, but increase the likelihood of his choosing something other than the optimum solution to a particular problem. Apparently, its purpose is to throw light on the most important problems connected with the design and construction of buildings and structures, especially foundations on permafrost, and to set examples for their solutions.

In contrast, SNiP II-18-76 is an official state document, having the force of law for all institutions and organizations throughout the U.S.S.R. The SNiP does not permit variance from its basic regulations, and it exempts an engineer from responsibility for his decisions. In the SNiP references are permitted only to other SNiPs, the number of which is limited. It probably makes design more conservative, and in some cases it makes it difficult to adopt nonstandard solutions. It is written in language that does not allow various interpretations. Many questions are omitted under the assumption that the answers are known. One of the purposes of the SNiP II-18-76 is to provide the comprehensive information necessary to simplify design procedures for foundations on permafrost.

Foundation design

Special Report 80-34 and the Soviet SNiP are substantially different from each other, which makes comparison difficult. However, their main sections—both dealing with the design of foundations—are comparable. These consist of three parts: foundation bearing capacity, settlement analyses, and determination of pile bearing capacity.

Analysis of foundation bearing capacity

In SR 80-34 one method for analyzing the ultimate bearing capacity is based on the hypothesis that frozen soil is purely a cohesive material, a conservative assumption because internal friction and adfreeze forces are neglected†. It is assumed that the long term strength limit is zero ($\sigma_\infty = 0$) and that the decrease in soil strength with time is described by Vyalov's equation

$$\sigma_n(t_m) = \frac{\beta}{\ln\left(\frac{t_m}{B}\right)} \tag{19}$$

where $\sigma_n(t_m)$ is the time-dependent frozen soil strength in uniaxial compression; t_m is time to failure. Special Report 80-34 recommends that the parameters β and B in eq 19 be determined by the results of at least two laboratory tests on undisturbed samples of each foundation soil; the tests are to be performed near the estimated temperature of the natural foundation soil. The nominal long term frozen soil strength σ_n is calculated by extrapolation over a very long period of time—the structure's lifetime $t = t_m$. The bearing capacity of foundations for all types of soils is calculated by Terzaghi's formula which, for shallow square footings on ideally cohesive soil and neglecting internal friction, has the form:

$$q^* \leqslant \frac{1}{SF}\,(1.3 \times 5.7\, c_n + \gamma_n h) \tag{20}$$

where a single safety factor $SF = 2$ is added. Notice that the recommended safety factor depends upon neither the number of tests nor variations in the soil characteristics.

The SNiP also assumes that frozen soils can be considered as ideally cohesive media, but adfreezing forces are taken into account (see eq 6). The minimum number of required tests is six. The soil safety factor (see eq 1) *does depend* on the number of tests or on variations in the determined parameter with respect to its average value. For the case of ideal test data, when there are no variations in characteristics, the soil safety factor will be equal to *unity*. In other words, theoretically, the bearing capacity of foundations on frozen soils estimated by the SNiP could be (taking into account the correction coefficients) up to 50% more than that estimated by SR 80-34. However, in practice such a difference is not observed because there always is a certain scatter in experimental data.

Unlike SR 80-34, the SNiP takes into account the dependence of footing bearing capacity on adfreezing forces. The influence of the latter on the foundation bearing capacity can be significant, and the disagreement between SR 80-34 and SNiP in the evaluation of footing bearing capacity is thereby magnified. The neglected adfreezing forces and the internal friction of the soil can thus be interpreted as an additional safety factor implicitly incorporated in SR 80-34.

As was mentioned above, an important feature of the SNiP that is not included in SR 80-34 is a group of tables showing the resistance of frozen soil of various types to normal pressure and shear along adfreezing surfaces. For preliminary design, and even

†In some cases internal friction can also be taken into account; however, the circumstances under which the incorporation of internal friction is possible are not expressed clearly in SR 80-34.

for final design of less important structures, these tables offer an opportunity to decrease the cost of soil testing, which is considerable, and to simplify the design procedure.

The convenience and simplicity of the SNiP method of calculating bearing capacity can be demonstrated by applying it to an example in SR 80-34 (p. 166-170). In the SR example, the dimensions calculated for a footing on frozen inorganic sandy silt with a temperature of 30°F ($\approx$ -1.1°C) are 4.5 x 4.5 ft = 20.25 ft^2 (18,813 cm^2). The design load N_d = 150 tons. Foundation depth is 7 ft. The apparent safety factor $SF \approx 2.25$. The lifetime of the structure is 25 years.

According to the SNiP table of allowable bearing capacities, for this type of soil† and this temperature, the nominal resistance, without a large error, can be taken as $R \approx 9.4$ kgf/cm^2. The nominal bearing capacity of the foundation, by eq 6, will be equal to

$$Q = k_{u_c} RF = 1.1 \times 9.4 \times 18{,}813 \approx 194{,}526$$

kgf or 194.5 tons.

According to eq 2 the design load on the subsoil, even without taking into account adfreezing forces, will be

$$Q^* = \frac{Q}{k_r} = \frac{194.5}{1.2} = 162 \text{ tons} > 150 \text{ tons}.$$

This comparison demonstrates the advantage of the tabulated soil characteristics for preliminary design procedures when the most economical construction method (principle) or type of foundation is being selected.

The values of the frozen soil characteristics tabulated in the SNiP are called long term resistances because they were computed by eq 19, taking the structure's lifetime to be 50-100 years. For preliminary calculations it is not important whether the computation is for 25 years or 100 years, since:

1. Results of the extrapolation from experimental data are uncertain for periods of time $t \to \infty$.
2. Variations of the final dimensions of the foundation are possible during the design procedure.

Actually, extrapolation of the experimental data for long periods of time does not, as a rule, give stable results. The nominal characteristic values with respect to time can vary as much as 100% or more, depending on the soil type and its temperature, the number of tests, the test procedure, etc. Specifics of design always lead to variation of the foundation's final dimensions, which can be about 10% more than those estimated by soil bearing capacity. In the considered example the final dimensions are therefore 12.5% larger than required by SR 80-34 for a safety factor of 2. It is not difficult to show that in this example the long term strength of the frozen soil, calculated by taking into account the structure's lifetime of 100 years, differs from that calculated for a 25-year structure lifetime by only about 7%, i.e., the difference is close to the accuracy of the measurements.

Finally, if an adequate correlation between the soil classifications of the SNiP and SR 80-34 were developed, the values of frozen soil resistances R and R_{ad} tabulated in the SNiP might be included in western design practice.

Creep deformation

Analysis of deformation (settlement) is the most important part of foundation design. Most reported foundation failures have been caused by nonuniform settlement rather than inadequate bearing capacity.

According to SR 80-34 the analysis of settlement of footings on all types of frozen soils is based on the following principles:

1. Allowable load on the subsoil surface is determined by a bearing capacity analysis.
2. *Analysis of deformation* is necessary for all types of soils except solid rock formations; allowable limits of settlements are not given.††
3. Total settlement is estimated by summarizing settlements originating in the separate layers using the formula**

$$S(t) = \sum_{i=1}^{m} \epsilon_i(t)\, h_i \qquad (21)$$

where

$\epsilon_i(t)$ = subsoil i^{th}-layer creep strain at a given temperature

h_i = thickness of the layer

m = number of layers.

4. The stress-strain relationship is nonlinear and is described by Vyalov's equation[16]. The latter can be represented in a form that is similar to eq 14

†This is a very rough approximation since it is impossible to establish strict correspondence between American and Soviet soil classifications without having a complete description of the soil.

††In some cases they are limited by local construction codes.
**In fact three different methods were presented in SR 80-34. Of the three, in the author's opinion, the method of settlement analysis considered below is preferable.

$$\epsilon_i(t) = \frac{K_i' \sigma_i^n \, t^\lambda}{(1+|\theta_i|)^\alpha} \tag{22}$$

where

$\epsilon_i(t)$ = average creep strain in the i^{th} layer along the central axis of the foundation.

The magnitude and distribution of stress under the foundation is calculated by using the elastic theory.

Comparing the SNiP's approach to settlement analysis to that of the SR, one can conclude that, in spite of the fact that both methods are based on the same theoretical premises, a certain discrepancy in the final results will occur. Special Report 80-34 does not require the calculation of settlements of foundations on solidly frozen soils, but suggests this be considered and, following the practice adopted throughout the SR, leaves the decision whether to calculate them or not to the design engineer. As was mentioned above, according to the SNiP the analysis of deformations is necessary only for foundations on plastic frozen and thawing soils. Settlements of foundations on solidly frozen soils are not calculated at all. This is one of the main differences between the SNiP and SR 80-34 regarding settlement analysis.

From the viewpoint of using the time factor in the analysis of foundation settlements, the SR approach seems to be more attractive. It makes it possible to estimate the settlement at any moment of the structure's lifetime. It takes into account the stress-time hardening effects and the temperature dependency of strain. It could be more complete even within the limits of the empirical approach if the method took into consideration the subsoil temperature changes with time as well as the overburden pressure of the soil. However, the influence of the latter factors upon the final results may not be very substantial.

A more substantial discrepancy in the results of calculations from the SNiP and SR 80-34 is caused by the fact that the deformation characteristics used in the analyses are determined by different methods.

In the SNiP, settlement of foundations on plastic frozen soils is estimated using soil deformation moduli E or compressibility coefficients a (see eq 9). This method does not permit the estimation of the accumulation of settlements with time. Nevertheless, from an engineering viewpoint it could give a satisfactory result if a *practical* methodology for determining the *frozen soil* moduli of deformation or the compressibility coefficients had been developed.

Special Report 80-34 recommends that the deformability parameter K' (see eq 22) be determined from unconfined compression tests. The best correspondence between the results of estimations and the data of field observations of structure settlements will be obtained if soil deformation moduli E, determined from field tests with bearing plates, are used in the analysis. If the compressibility coefficients a are used in the analysis, the expected settlement may be overestimated by a factor of 2 to 5 compared with the field observations. It is obvious that using deformability parameters obtained from unconfined compression tests that take into account the time factor may cause settlement to be overestimated to an even greater degree. Hence, as far as settlement analysis is concerned, the recommendations of SR 80-34 seem to be more conservative than the provisions of the SNiP.

Note that the latest data on the long term (up to 17 years) large-scale laboratory tests of Vyalov[21] showed that time-dependent settlements of bearing plates on plastic frozen soils can be described by the equation

$$S(t) = (1 - \mu^2)\, d \left(\frac{p}{A_1}\right)^n \upsilon t^\lambda \, \omega \tag{23}$$

where

d = diameter of a plate
μ = Poisson's ratio
υ, n and λ = parameters
ω = coefficient depending upon the shape of the footing
A_1 = deformability coefficient of the frozen soil.

Equation 23 was obtained by a modification of Shleicher's solution and coincides reasonably well with eq 18. However, the problem is how to determine the parameters of these equations under laboratory conditions that will give settlements close to those that will be experienced under field conditions.

This problem is closely connected with a very important question, which should be emphasized in this comparision of the two documents: how to mathematically describe the creep curves and determine the parameters in eq 13, 14, and 22. This question is fundamental, and all the analyses of bearing capacity or foundation settlement depend upon its solution.

It is well known that the strain hardening parameter n depends significantly on stress, changing from relatively high values (for some types of frozen soils n = 8–10) at high stresses to about unity for small stresses. In SR 80-34 it is assumed that the strain hardening parameter may differ from unity, while in the SNiP it is accepted as unity in all cases. The time hardening parameter λ also depends upon stress, and both n and λ can vary with time and temperatures. Hence, all the analytical methods mentioned above, which employ constant values for these terms, have an approximate character. Note that deformation of frozen soil and failure constitute a unified physical process. Therefore the problem of the analysis of foundation settlements cannot be successfully solved until a unified constitutive equation is established which describes the long term failure and deformation of the soil. Equations of this type were suggested in two works[20, 27] and were verified for various types of frozen and unfrozen soils at simple and complex stress-strain states.

Entire creep at constant stress

A unified constitutive equation derived by this author[20,23,24,25,26] on the basis of the rate process theory has the form (for a given temperature):

$$\dot{\epsilon}(t) = \frac{C}{t_m} \left(\frac{\sigma}{\sigma_o}\right)^n \left(\frac{t_m}{t}\right)^\delta \exp \frac{\delta t}{t_m} \qquad (24)$$

where

$$t_m = t_o \left(\frac{\sigma}{\sigma_o}\right)^{-m}; t_o = \frac{h}{kT} \exp\left(\frac{E}{RT}\right) \qquad (25)$$

and C, n, m and δ are dimensionless parameters

σ_o = "instantaneous" strength of the soil
t_o = Frenkel's relaxation time
h = Planck's constant
k = Boltzmann's constant
E = activation energy
R = gas constant
T = absolute temperature, K
t_m = equation (criterion) of the long term strength, i.e. the time interval from the initiation of the test until the inflection point of a creep curve (Fig. 2, 5, 6).

Substituting eq 25 into eq 24, the latter takes the form:

$$\dot{\epsilon}(t) = \dot{\epsilon}_o t^{-\delta} e^{\delta t/t_m}; \ \dot{\epsilon}_o = C_1 \left(\frac{\sigma}{\sigma_o}\right)^{n_1} \qquad (26)$$

where $\dot{\epsilon}_o$ is the initial strain rate at the time $t = t^*$ = 1 min, $C_1 = C/t_o{}^{1-\delta}$, and $n_1 = n + m - m\delta$. From a mathematical viewpoint, the structure of eq 26 corresponds to equations derived in some of this author's other works[20,27]. However, eq 24 and 26 have better dimensions that give more flexibility in mathematical transformations and better correspondence to test data.

Secondary creep

The strain rate reaches a minimum $\dot{\epsilon} = \dot{\epsilon}_m$ when the time $t = t_m$. In this case eq 24 becomes:

$$\dot{\epsilon}_m = \frac{C}{t_m} \left(\frac{\sigma}{\sigma_o}\right)^n e^\delta = C_2 \left(\frac{\sigma}{\sigma_o}\right)^{n+m} \qquad (27)$$

where $C_2 = Ce^\delta/t_o$. Taking into account eq 25, eq 27 can be presented as the following:

$$\dot{\epsilon}_m = C_3 T \exp\left(-\frac{E}{RT}\right) \left(\frac{\sigma}{\sigma_o}\right)^{n+m} \qquad (28)$$

where $C_3 = Ce^\delta k/h$.

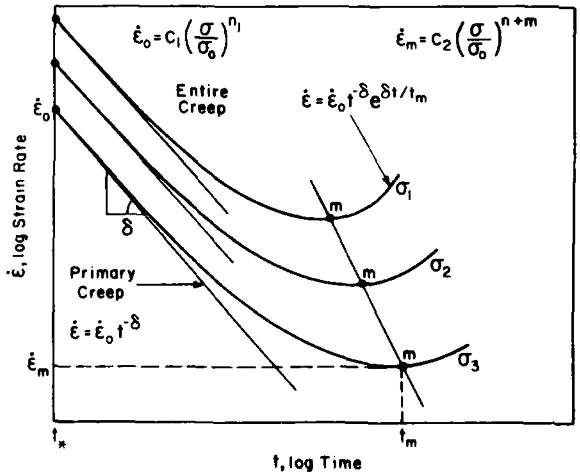


Figure 5. Creep curves in logarithmic coordinates (m—inflection points, $\dot{\epsilon}_o$—initial strain rate, $\dot{\epsilon}_m$—minimum strain rate).

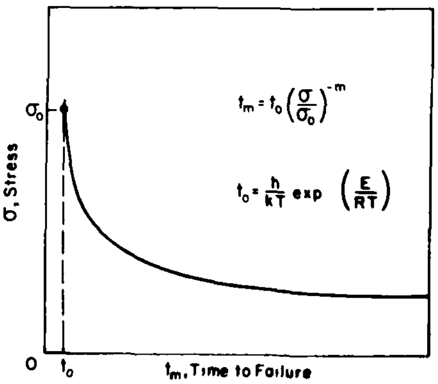


Figure 6. Time dependent failure of frozen soil (σ_o—instantaneous strength, t_o—relaxation time).

An approximate equation for deformations may be presented in the following form:

$$\epsilon \approx \epsilon_0 + B\sigma^{n_1} t^{\lambda} e^{\delta t/t_m} \; ... \quad (29)$$

where ϵ_0 is the instantaneous strain, $B = C/t_0^{\lambda}\, \sigma_0^{n_1}\, \lambda$ and $\lambda = 1-\delta$. Equation 28 expresses the temperature dependencies of the deformation process at the secondary creep and makes it possible to estimate the activation energy of frozen soil at this point. To describe the entire creep in accordance with eq 29 and to determine all six parameters (C, n, m, δ, t_0 and σ_0) three tests are needed: two creep tests and one test for σ_0.

It should be noted that the strain rates on the primary, secondary and tertiary creep depend upon the ratios of t_0/t_m and t/t_m. In practice foundations are designed to satisfy the requirements of eq 7 or 20 and eq 19 or 25. The time to failure of the upper layer of the subsoil beneath the foundation is anticipated to be equal to the structure lifetime $t_{lf} = t_m$. This means that the strain rate of this layer reaches its minimum within this time. On the other hand, the stresses decrease rapidly with the depth (Fig. 1). The times of failure t_{mi} of the soil layers below the upper layer are always larger than the selected structure lifetime, i.e. $t_{mi} > t_{lf}$. Therefore the strain rates of these layers will not reach their minimums within this time. Thus, the subsoil of the foundation will deform as an entire unit with a decreasing strain rate (primary creep) that can be approximated by eq 14, 17, 18, 22 and 23; eq 26 also describes the primary creep (Fig. 5) when the exponential term is small ($t \rightarrow 0$) and can be neglected.

Hence, eq 24 describes time-dependent deformation for all conventionally defined stages of creep over the entire range of applied stresses. It contains the equation of the long term strength and consequently can be extrapolated into the range of small stresses. It is also valid for the complex stress state of soil. A remarkable property of this equation is that it has excellent dimensions and a deep physical sense. This equation can take various analytical forms, and the process fits well to test data. It is possible that the analytical solution of the boundary problem using this equation will give calculated settlement values which are closer to field measurements.

Special Report 80-34 does not present a method for computing the amount of settlement due to the thawing of soil but states that "... thaw settlement problems should be avoided by adopting the proper foundation design approach for the conditions and by designing for full stability control ...[22]" The basis for selecting the type of foundations and control measures is outlined. Since the methods of computation for thaw settlement and for the stability of foundations subjected to the action of frost heave are not presented in the SR, no comparative analysis with the SNiP is possible on these topics.

Determination of pile bearing capacity

Piles are the most widely used type of foundation in northern regions. Therefore, the determination of pile bearing capacity can greatly influence construction costs, and also can strongly affect the reliability of a structure. The bearing capacity of single piles, considering skin friction, is determined by an empirical formula similar to eq 6, with the difference that in the SNiP the point bearing resistance of the pile is also taken into account. This provides an opportunity to increase the theoretical pile bearing capacity by 10-20%, but only in those cases when it is calculated using tabulated soil characteristics. The latter are given only in the SNiP. At the same time both the SNiP and SR 80-34 allow bearing capacity to be determined by field test data. This circumstance offers a chance to compare both documents on the basis of an analysis of experimental data.

To perform such a comparison let us use the SR 80-34 example of an estimate of bearing capacity of a single pile (Fig. 7). According to the method of analysis of field test data accepted in the SR, the nominal bearing capacity† of the pile is L_n = 147,000 lb. The allowable design load is $L_d \approx$ 45,000 lb. The apparent safety factor is $SF = L_n/L_d$ = 147,000/95,000 = 3.27.

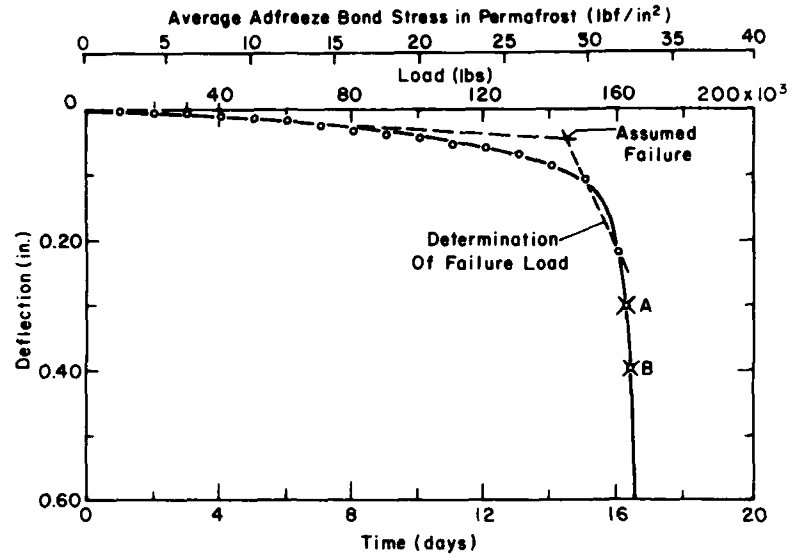
Let us determine the pile bearing capacity by the SNiP method. In Figure 8, the experimental curve of Figure 7 is presented on a logarithmic scale[7]. The nominal bearing capacity, determined by the intersection of the straight lines, is P_n = 145,000 lb. The design bearing capacity Q^* or the allowable design load N_d, according to eq 2 and 8, will be

$$N_d = Q^* = \frac{k_t}{k_r k_s} P_n \quad (30)$$

where the soil safety factor k_s = 1.1, the reliability safety coefficient k_r = 1.2 for foundations of buildings (for pile foundations of bridges, as previously mentioned, values of k_r vary from 1.4 to 1.75), and k_t is a temperature correlation coefficient.

Thus, the ratio of the allowable design loads of the SNiP and SR 80-34 is determined by the following expression:

†Failure load in SR 80-34 terminology.



Pile type: 8-in. pipe, 36 lb/ft
Pile length: 20.9 ft
Length below surface: 20.4 ft
Embedment in frozen soil: 16.1 ft

Soil profile: 0–1 ft peat, 1–20.4 ft (bottom of pile) silt
Backfill around pile: silt-water slurry
Avg temp of frozen soil: 29.2°F

Loading Schedule: 10,000 lb increments applied at 24-hr intervals. The deflection shown for an increment is that observed just prior to application of next increment.
Note: Pile not isolated from soil in thaw zone.

COMPUTATION OF ALLOWABLE DESIGN LOAD

Failure load = 147,000 lb Surface area of pile in permafrost = 5230 in.²
Average adfreeze bond stress at failure = 147,000/5230 = 28.1 lbf/in.²
Adjusting for 10,000 lb/day rate of loading (by interpolation) average adfreeze bond stress at failure = 21.5
Assuming failure stress is 40% greater than average sustainable stress, average sustainable adfreeze strength = 21.5/1.4 = 15.3 lbf/in.²
Sustainable pile load capacity = 15.3 x 5230 = 80,000 lb
Using a factor of safety = 2.5, allowable design load = adjusted failure load/FS = 21.5 x 5230/2.5 = 45,000 lb

Figure 7. Determination of pile bearing capacity. (From SR 80-34, Chap. 4, Fig. 4-80.) A and B are additional points.

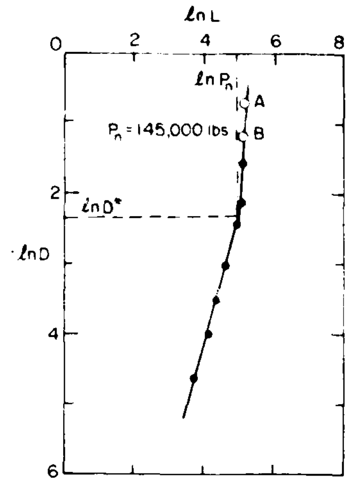


Figure 8. Determination of the pile bearing capacity according to recommended Soviet practice[7]. A and B are additional points.

$$\frac{N_d'}{L_d} = \frac{k_t}{k_r k_s} \cdot \frac{P_n}{L_n} \cdot SF \qquad (31)$$

If it is assumed that the structure in question is a bridge foundation, and the subsoil temperature equals the test temperature ($k_t = 1$), the comparison of the allowable loads will be determined by the ratio:

$$\frac{N_d}{L_d} = \frac{145{,}000 \times 3.27}{147{,}000 \times 1.1} \cdot \frac{1}{k_r} \approx \frac{2.93}{k_r}$$

and thus it will depend upon the number of piles in the foundation (Table 1).

Table 1. Ratio of allowable pile loads for bridge foundations determined by the SNiP (N_d) and SR 80-34 (L_d). (The comparable ratio N_d/L_d for building foundations with any number of piles is 2.44 for k_r = 1.2.)

Number of piles in the foundation	> 20	20-11	10-6	5-1
Ratio N_d/L_d	2.09	1.83	1.78	1.67

These ratios convey the impression that the difference between the approaches of the SNiP and the SR can be up to 100% for bridge foundations and 150% for foundations of buildings. The difference may be less if the different procedures used in obtaining the experimental curves are taken into account.

In the SR 80-34 example, the static load, applied by small increments of ΔP_i = 10,000 lb at 24-hr intervals, constitutes a step loading regime with the rate of loading equal to $\dot{P}$ = 10,000 lb/24 hr. It is known that the failure load depends substantially upon the loading regime. A family of load-deformation curves (similar to Fig. 7) can therefore be obtained for the same soil conditions by varying the rate of loading. Strictly speaking, none of the curves produced by this type of experiment can be used to determine the pile bearing capacity since the results of tests under the step loading regime cannot be directly transferred to the creep regime (the real conditions under which piles are used).

On the other hand, according to the Soviet practice[7], the load is applied to a test pile by the increments $\Delta P_i \approx 0.2$ to $0.5\ Q$, but the duration of each increment of loading is varied. The next load ΔP_{i+1} is applied if the settlement (displacement) rate of the pile under the previous step load does not exceed the conventional allowable value $\dot{D} \leq$ 0.2 mm/24 hr. The test is stopped when the vertical displacement rate accelerates rapidly. The values of Q are determined approximately by eq 6. This test procedure[7] is probably closer to the creep regime and therefore the absolute values of nominal bearing capacities of piles determined by the SNiP may be less than those determined by the provisions of SR 80-34. However, it should be noted that the SNiP method also cannot be regarded as satisfactory because it admits variation of the load-deformation curves, depending on the allowable limit of the displacement rate of the pile. Neither method takes into account the structure's lifetime, even though it is an essential variable in the determination of bearing capacity of footings under both the SNiP and SR 80-34. Methods for pile settlement analysis are not developed in either.

It would be appropriate, instead of performing one pile test, to perform two short term tests in step loading with different rates of loading

$$\dot{P} = \frac{dP}{dt} = \text{Constant}$$

The values of load increments ΔP can be the same, while the intervals of Δt, which are constant for each test, must be different (Fig. 9 and 10). If it is assumed, for the first approximation, that the total pile foundation settlement would equal the settlement of the single pile, eq 4 takes the form

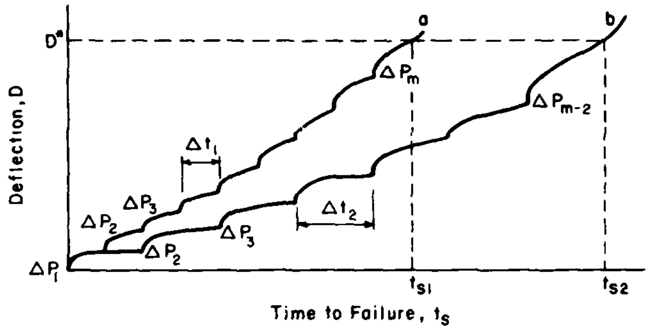


Figure 9. Diagram for determining pile bearing capacity from two tests with various rates of loading: P_1*(a) and* P_2*(b).*

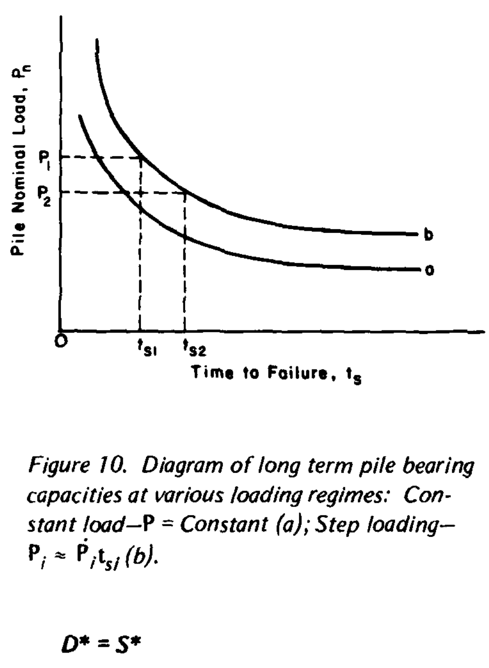


Figure 10. Diagram of long term pile bearing capacities at various loading regimes: Constant load—P = Constant (a); Step loading— $P_i \approx \dot{P}_i t_{si}$ *(b).*

$$D^* = S^*$$

where S^* is the permissible ultimate value of foundation settlement depending upon the type of structure and D^* is the permissible ultimate vertical displacement of the pile.

The pile displacement that corresponds to the inflection point of the transition from secondary creep to tertiary creep on the last step of loading can also be accepted as a failure criterion.

It is not difficult to show[20] that if, for example, pile long term bearing capacity is described for constant load by a power function,

$$t_s = B_1 P^{-n} \quad \text{for } P = \text{Constant} \tag{32}$$

long term bearing capacity for step loading will be given by the formula

$$t_{si} = (1+n) B_1 P_i^{-n} \quad \text{for } P_i \approx \dot{P}_i t_{si} \tag{33}$$

where t_s is time to failure according to eq 32.

The parameters B_1 and n can be easily determined, having from field test data two values of failure loads

$$P_{1,2} = \sum_{i=1}^{m} \Delta P_i$$

and two times to failure

$$t_{s1,2} = \sum_{i=1}^{m} \Delta t_i$$

where m = the number of steps. The value of nominal bearing capacity of a pile can be estimated, depending upon the structure's lifetime t, as

$$P_n(t) = \left(\frac{B_1}{t}\right)^{1/n} . \tag{34}$$

Comparing eq 32 and 33, one can see that for the same failure times the bearing capacity of the pile in creep conditions will be less by a factor of $(1+n)^{1/n}$ than it is in step loading. If we assume that parameter $n \approx 1.5 - 2$, the actual ratio of allowable loads on piles determined by SR 80-34 and the SNiP may in some cases be close to unity. The proposed method can be applied to piles of various lengths and various ratios of the sides, as long as the dimensions do not differ greatly from those of the test pile. Settlement of single piles can also be determined using information obtained from such tests. All these questions are closely connected with the search for constitutive equations and failure criteria of frozen soils for various loading regimes; these questions are discussed in the author's works[20,26].

In spite of the imperfections mentioned, SR 80-34 is a serious, well organized, and well considered document satisfying many requirements and needs of a designer in different fields of engineering geocryology.

CONCLUSIONS

1. SR 80-34 and the U.S.S.R. SNiP II-18-76 present methods of foundation design on permafrost based on recent knowledge of frozen soil mechanics and engineering geocryology. At the same time the documents are not equivalent. They differ in the purposes they pursue, the scope of the problems discussed, and the methods of presentation of the material.

2. Adequate unified constitutive equations for frozen soil have not been established, nor have the boundary problems been solved, so the present state of settlement analyses of foundations on frozen ground, exemplified in both SR 80-34 and the SNiP, can be regarded as unsatisfactory. Hence, further research is needed in this area and the accumulation of data from field observations should also be high priority research.

3. Within the context of the dynamic and developing state of the art, it may be concluded that the approach to bearing capacity and settlement analyses of foundations employed in SR 80-34 is more conservative than that of the SNiP.

4. The principal difference between SR 80-34 and the SNiP is in design methods and in assessment and application of appropriate values of the safety factors. This circumstance leads to a substantial discrepancy in the determination of dimensions of footing foundations and of bearing capacity of piles. In fact neither of the methods employed for determining pile bearing capacities from field static loading tests can be regarded as satisfactory. Methods for pile settlement analysis are not developed either.

5. Neither the SNiP nor the SR 80-34 method for settlement analysis contain identifiable safety factors. They are contained in both codes, however, in the form of various uncertain assumptions. It would be appropriate to include a single economic safety factor applicable to settlement analysis. Such a coefficient could take into consideration the economic consequences of excessive settlement of subsoils that could lead to large costs for repair and reconstruction of structures on permafrost.

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