PERMAFROST - Seventh International Conference (Proceedings),
Yellowknife (Canada), Collection Nordicana No 55, 1998

# RUSSIAN AND NORTH AMERICAN APPROACHES TO PILE DESIGN IN RELATION TO FROST ACTION

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**Abstract**

North American and Russian approaches to pile design are based on different views of the nature of the tangential frost heave force. In North America, the latter is associated with the long-term adfreeze bond. According to the Russian approach, the frost heave force is the result of friction from frozen soil sliding along a pile. Russian permafrost scientists have found that an adfreeze bond between a pile and frozen soil in the active layer is usually broken at the beginning of winter. Two widely used methods of design of foundation stability from frost action were analyzed. The first method is implemented by Russian building code (SNiP method), and the second one by the American manual "Arctic and Subarctic Construction Foundations for Structures" (TM-5 method). The SNiP method is reliable in most typical situations but has some limitations. The TM-5 method provides a safe design for practically all situations, but for most of them, it greatly overestimates the length of piles required.

## Tangential forst heave stress

The tangential frost heave force acting on a pile surface inside the active layer is not known, but it cannot be greater than the adfreeze bond between the pile and frozen ground in the active layer. For design purposes, it has been assumed (Tsytovich and Sumgin, 1937) that the tangential frost heave stress is equal to the adfreeze strength. At the time when this assumption was first made, only the instantaneous adfreeze strength was known and its value was recommended for design. This approach was implemented by Russian standards before W.W.II and some American Manuals (i.e., NAVFAC DM-9, 1967). Comparison of tangential frost heave forces measured in field experiments with recommended adfreeze strength values, showed that instantaneous adfreeze strength is more than 10 times greater than tangential frost heave stress (Bikov, 1939; Tsytovich, 1973).

The contemporary North American approach associates tangential frost heave stress with long-term adfreeze strength. According to the Russian approach, the design tangential frost heave stress acting throughout the entire active layer is much smaller than the long-term adfreeze strength. It has been observed many times in Russia, that the layer of frozen soil is firmly adfrozen to the side of foundation only at the beginning of winter. At some moment, under growing basal frost heave force, failure occurs at the contact between frozen soil and the side of the foundation. Soil then slides along the foundation during the rest of the frost heave period. The sliding motion of the soil attempts to drag the foundation upward. Prior to sliding, the tangential frost heave force is equal to the adfreeze bond between the pile and soil. During the sliding stage it is considerably lower (Principles of Geocryology, 1959; Tsytovich, 1973; Vialov and Egorov, 1958 and other). The main supporting factors in the Russian approach are field measurements of large displacements of the frozen soil in the active layer along the foundation, and comparison of the tangential frost heave force before and after displacement, and its comparison with long-term adfreeze strength.

It is known that even small displacements of a pile in frozen soil can destroy the adfreeze bond between a pile and the soil. According to Andersland and Anderson (1978, page 338): "Generally, the adfreeze bond of normal-length pile is broken after only 3 mm displacement," which is in agreement with observations made by N. Peretrukhin and his co-authors (1978). The displacement of frozen soil in the active layer around a pile was first studied by N. Bikov (Bikov and Kapterev, 1939). The total displacement reported was about 7 cm. According to N. Bikov, frost heave occurs if the foundation can be carried by moving soil and consequently, the tangential frost heave force is a result of the impact of the moving soil on the foundation, but not the result of the forces of heaving soil which is firmly adfrozen to a post. Displacement of frozen soil in the active layer along a pile was also studied by S. Vialov and N. Egorov (1958), V. Orlov (1962), B. Dalmatov

(Dalmatov et al., 1978), B. Elgin (1983), and some others. Measured displacements varied from 5 to 10 cm.

Based on observations of this kind, B. Dalmatov (1957) worked out a laboratory method to evaluate tangential frost heave stress. According to B. Dalmatov, tangential frost heave stress on a pile surface is equal to the stress measured at the sliding contact of a model pile (12 cm long) pushing through frozen soil to which the pile was initially firmly adfrozen. The applied force is measured and the tangential heave stress is evaluated. The design tangential frost heave stress in Dalmatov's method corresponds to the stress measured when displacement of the foundation model is more than 1 cm. This displacement guarantees that the adfreeze bond between the foundation model and the frozen soil is broken. The stress-strain curve in Dalmatov's experiment is similar to the shear stress-strain curve of unfrozen soil in the area of very large displacement. The shearing resistance of unfrozen soil after very large displacement was called the residual strength (Skempton, 1964). We think that the similar feature in Dalmatov's experiment can be called residual adfreeze strength. B. Dalmatov invented his method about 10 years before the result of Skempton's study was published. He called the strength measured at the sliding contact of a post and frozen soil the Russian word ustoychiviy, which can be translated as stable, unchangeable, or final. Because of the meaning of this word, it was mistaken many times for the long-term adfreeze strength. The term residual is well known now in soil mechanics and cannot be confused with other definitions. Russian permafrost engineers found that residual adfreeze strength is good for evaluating tangential frost heave stress and Dalmatov's method is recommended in manuals on the determination of properties of frozen soils. Some Russian scientists explain residual adfreeze strength as friction strength. It appears reasonable to compare residual adfreeze strength with residual shear strength, which is usually expressed as a frictional resistance.

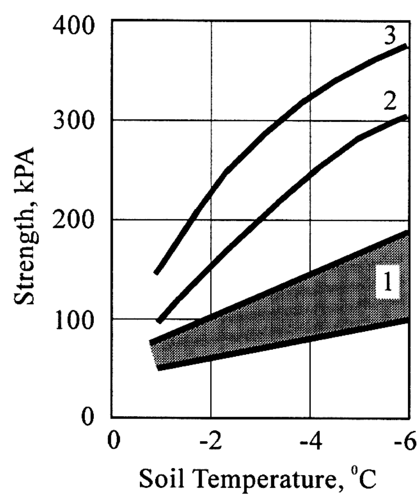
B. Dalmatov developed an empirical equation to evaluate the residual adfreeze strength (S):

$$S = a - bt \qquad [1]$$

where a and b are specific for each soil and t is soil temperature (°C) with its sign. N. Tsytovich (1973) generalized data on a and b, which had been found by several Russian permafrost scientists for different soils. He found that a varies from 40 to 70 kPa and b varies from 10 to 20 kPa/°C.

It is interesting to compare long-term adfreeze strength and residual adfreeze strength. In Figure 1, one shadowed area and 2 curves are presented. The shadowed area is an envelope of values of residual adfreeze strength estimated from Dalmatov's formula. Residual adfreeze strength computed from this formula agrees with numerous Russian field data and also with field data obtained by Penner and Irwin (1969) and Johnston ( 1981). Curves 2 and 3 in Figure 1 present data for long-term adfreeze strength of clay and silt with a concrete foundation. Curve 2 results from data recommended by Russian building code (SNiP, 1991), and Curve 3 results from experimental data for typical Yakutian soils (Votyakov, 1975). Figure 1 shows that residual adfreeze strength between frozen soil and the foundation is two to four times less than long-term adfreeze strength. A similar result was found by M. Goldstein in his laboratory experiments (Tsytovich, 1973). According to the experiments, the strength of the frozen soil to wood and concrete during continuous movement of the column relative to soil, is about one half of the long-term adfreeze strength between them. This also agrees with the opinions of K. Linnel and E. Lobacz (1980) who indicate that once the adfreeze bond between frozen soil and the foundation is broken, it does not readily reheal and only 1/2 or less of the original adfreeze bond potential may be available. Unlike Russian scientists, they applied it only to a permafrost environment and did not consider the similar situation in the active layer.

Numerous field observations on the change of tangential frost heave stress during winter have been made in Russia. Figure 2 is based on data by Vialov and Egorov (1958) and by B. Elgin (1983), and shows the change of tangential frost heave stress averaged throughout the frozen part of the active layer. At the beginning of win-




1. Area of variation of residual adfreeze strength associated in Russia with tangential frost heave stress.
2. Design adreeze strength, according to SNiP (1991)
3. Adfreeze strength of typical silty soils of Northern Yakutia (Votyakov, 1975)

*Figure 1. Tangential frost heave stress and adfreeze strength.*

ter, when the frozen part of the active layer is thin, the tangential frost heave stress reaches its maximum which is associated with the adfreeze stress in Figure 2. After the adfreeze bond is broken, the tangential frost heave stress decreases. The design tangential heave force is equal to the maximum value of the product of frost heave stress and corresponding areas of foundation in the frozen part of the active layer, at moments when the frost heave force is measured (i.e., 10 times during the active layer freeze back). Some field methods of the measurement of the tangential frost heave force note only the maximum. The design frost heave stress can be found by division of the maximum frost heave force by the entire area of the foundation side in the active layer. The design frost heave force is greatly over-estimated if it is based on the approximation of adfreeze strength (peaks on curves) throughout the entire active layer. This represents the Russian approach to evaluation of the design tangential frost heave stress.

If residual adfreeze strength between the foundation and sliding soil is several times less than the long-term adfreeze bond, it is reasonable to look for situations in which the long-term adfreeze strength is not overcome at the beginning of winter. The persistence of an adfreeze bond and an increase in the tangential frost heave stress were observed in the field by E. Penner and L.E. Goodrich (1983), and by B. Elgin (1983) when frost-susceptible soil in the active layer was covered with a layer of non-frost-susceptible soil. In experiments conducted by Penner and Goodrich (1983), a layer of gravel was used on the site to make it conform with the rest of the area. They found that heave forces were much greater than expected and "the layer of gravel over the site was thought to be the main cause of very high forces." They concluded that, "a surface of wet gravel over a heaving soil may have a serious detrimental effect and should be avoided by isolating the pile."

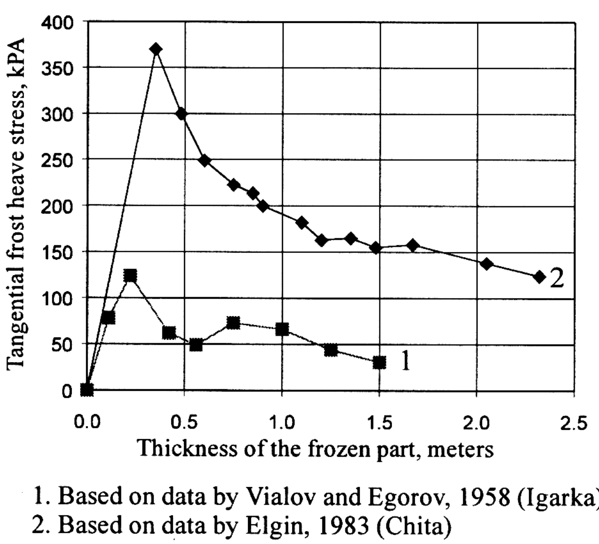



*Figure 2. Tangential frost heave stress averaged throughout the frozen part of the active layer.*

B. Elgin (1983) studied frost heave impact on piles at two sites. One site consisted of frost-susceptible soil throughout the active layer. At the second site, the upper part of the active layer was non-frost-susceptible soil and the lower part of the active layer was frost-susceptible soil. The frost heave force at the second site was at least two times greater than the frost heave force at the first site.

Based on the Russian approach, it is easy to explain the reason for the great increase in heave forces when non-frost-susceptible soil is underlain by frost-susceptible soil, in comparison with uniform frost-susceptible soil throughout the active layer. Non-frost-susceptible soil develops an adfreeze bond with the foundation, but the force which tries to break this bond does not exist (without frost heave of soil). The adfreeze bond between the non-frost-susceptible soil and the pile can be found only as a reaction to the load. In experiments with a restrain pile in non-frost-susceptible soil, the tangential force on the pile surface is zero. But sufficient adfreeze bond between the pile and non-frost-susceptible soil can be measured by other methods, such as pile extraction. This adfreeze bond can be mobilized when basal frost heave forces, from freezing frost-susceptible soil under a layer of non-frost-susceptible soil, push this layer up. When frost-susceptible soil underlying non-frost-susceptible soil starts to freeze, the frost heave forces acting on the base of freezing soil meet resistance from the adfreeze bond between the pile and frozen non-frost-susceptible soil and the adfreeze bond of freezing frost-susceptible soil. This effect of a sharp increase in the tangential frost heave force was measured by B. Elgin (1983) after the freezing surface has abandoned the layer of non-frost-susceptible soil. A combination of non-frost-susceptible soil with under lying frost-susceptible soil is much more dangerous for a frost heave impact on piles and posts than the impact of highly frost-susceptible soil throughout the active layer. Design engineers must be made aware of this situation as it requires special attention.

## Foundation design from tangential frost heave

We compared two methods which are recommended by Russian and American standards. Use of the first one is required by the Russian building code (SNiP 2.02.04-88, 1991). The second one was developed in the United States of America, by the Cold Regions Research and Engineering Laboratory (CRREL) (Linnel and Lobacz, 1980), and is a requirement of Technical Manual TM 5-852-4/AFM 88-19 "Arctic and Subarctic Construction Foundations for Structures" (1983).

**Table 1. Design standard tangential heave force based on SNiP 2.02.04-88**

| | Soil and degree of saturation | Value of $\tau_{fh}$, with active layer depth equal to 1 m | Value of $\tau_{fh}$, with active layer depth equal to 2 m | Value of $\tau_{fh}$, with active layer depth equal to 3 m |
|---|---|---|---|---|
| 1. | Silt and clay with liquidity index ($I_l$) more than 0.5, fine and silty sand with degree of saturation ($S_r$) more than 0.95; (highly frost-susceptible) | 130* | 110 | 90 |
| 2. | Silt and clay with liquidity index $0.25<I_l \leq 0.5$, fine and silty sand with $0.8<S_r \leq 0.95$, gravel with amount of clay, silt, or fine sand more than 30 percent (frost-susceptible) | 100 | 90 | 70 |
| 3. | Silt and clay with liquidity index $I_l \leq 0.25$, fine and silty sand with $0.6<S_r \leq 0.8$, gravel with amount of clay, silt, or fine sand from 10 to 30 percent; (slightly frost-susceptible) | 80 | 70 | 50 |

$I_l \quad (W - P_w)/I_w$

Where:

$W$ = Water content of soil
$P_w$ = Plastic limit
$I_w$ = Plasticity index

* Numbers are for concrete and wood piles. To find frost heave stress for steel piles, a coefficient of 0.7 has to be applied.

The Russian design of foundation safety against tangential frost heave is based on the behavior discussed above for frozen soil in the active layer. Consequently since 1954, Russian standards have not used the term adfreeze stress or adfreeze bond when they describe tangential frost heave stress. According to SNiP (1991), the stability of a foundation from tangential frost heave forces should satisfy the following equation:

$$\tau_{fh} \cdot A_{fh} - F \leq (\gamma_c / \gamma_n) \cdot F_r \qquad [2]$$

Where:

$\tau_{fh}$ is the design standard tangential heave stress (kPa), determined experimentally; in the absence of experimental data it can be taken from Table 1;

$A_{fh}$ is the area of lateral surface of foundation inside the active layer expressed in $m^2$;

F is the load on foundation including live one, kN;

$\gamma_c$ is the coefficient of operating performance, which can be taken to be equal to 1.0;

$\gamma_n$ is the coefficient of reliability, which can be taken to be equal to 1.1; for bridge foundation it is equal to 1.3;

$F_r$ is the adfreeze bond capacity mobilized between permafrost below the active layer and the pile; kN.

Examination of equation (2) can lead to the conclusion that the method assumes a very low value for the factor of safety (1.1 to 1.3). In reality, it is much higher, but it is hidden in the recommended values of $\tau_{fh}$ and $F_r$.

The data presented in Table 1 are based on numerous field experiments conducted in different regions of Russia during the last 60 years mainly with wooden piles and posts. Most experimental sites were located in the discontinuous permafrost zone. The active layer depth at the experimental sites was generally between 1.5 m and 3 m. No information was available regarding experimental sites in areas of low temperature permafrost (lower than -3°C).

According to the TM-5 method, the stability of the foundation against frost heave can be found using the following equation:

$$Q_h = (1 / FS) \bullet \left(Q_1 + Q_p\right) \quad [3]$$

Where:

$Q_h$ is the frost heave force and

$Q_h = f_h \bullet Af_h$ , that is, the adfreeze bond stress mobilized between frozen soil and pile by heave. For steel piles $f_h$ should be assumed equal to 300 kPa; for concrete and wood piles it should be increased 1.5 times;

A is the surface area of pile in the active layer;

$Q_l$ is the effective load on pile;

$Q_p$ is the adfreeze bond capacity mobilized between permafrost below the active layer and the pile;

FS is the factor of safety, equal to 3.9.

The tangential frost heave force according to TM-5 is much greater (up to 10 times and more including Factor of safety) than the same force found according to SNiP. Adfreeze bond capacities mobilized between the permafrost below the active layer and the pile recommended by SNiP and TM-5 are practically the same. As a result, the length of a pile designed according to TM-5 is several times greater than one designed according to SNiP. We compared the length of wood piles without external load designed according to SNiP and TM-5 with the real length of piles installed at two experimental sites in Russia. Stability of the piles from tangential frost heave impact at the experimental sites was proven by long-term monitoring. The first site was located at Scovorodino (Russian Far East, data from Bikov and Kapterev, 1940) and the second site was located at Igarka (Middle Siberia, data from Vialov and Egorov, 1958; and Orlov, 1962). The long-term monitoring showed that at both sites, piles installed deeper than 6 m were not affected by frost heave. According to the design based on recommendation of SNiP, the minimal safe depth of a wood pile without external load is 7.7 m at the Scovorodino site and 6.5 m at the Igarka site. Design based on the TM-5 guideline shows that the minimal safe pile length has to be more than 30 m for both sites.

Two diametrically opposed opinions have coexisted for many years in the permafrost literature regarding the impact of the foundation material on the magnitude of the tangential heave force. Some western sources (Torgenson, 1976; Johnston, 1981) give the magnitude of the tangential frost heave force acting on a steel surface as 1.5 times greater than the tangential force acting on concrete or wood surfaces. According to SNiP and TM-5 the frost heave force acting on concrete or wood surfaces is 1.5 times greater than that acting on a steel surface. This discrepancy was noticed by Gerasimov and Dokuchaev (1979). To solve the problem, they conducted thorough experiments with different materials and measured long-term adfreeze strength between a model of a post and frozen soil. They also measured roughness of surfaces and found that only a very rough steel surface (deformed reinforcement) has a similar adfreeze bond with soil as concrete. A smooth steel surface has a much smaller adfreeze bond with soil than concrete. By their very precise experiments and measurements, they proved what had been found previously in numerous laboratory experiments in Russia, that the adfreeze bond with steel is less than with concrete or wood. But in our opinion, the experiments did not address the tangential heave force acting on foundations made of the different materials for two reasons. The experiments were conducted in a cold room where temperature differences along the contact of the foundation and soil (typical for field conditions) were eliminated. The regularities which were found for adfreeze bond were extrapolated to tangential frost heave force.

In the active layer with its great temperature gradient, steel transfers heat better than concrete or wood and a steel pipe-pile can also provide convective heat transfer if its inner space is not filled. As a result, experiments which were made in equal temperature conditions cannot provide sufficient information for comparison of the impact of foundation material on the tangential frost heave force. In our opinion, the experiments provide very important information about the adfreeze bond of different construction materials and permafrost, but their results cannot be automatically extrapolated to tangential frost heave force in the active layer.

## Conclusions

There are two approaches to the tangential heave force description. The first one explains the tangential frost heave force as the adfreeze bond of freezing soil with the foundation. The second one explains the frost-heave force as residual adfreeze strength of the sliding frozen soil along the foundation (or friction) after the long-term adfreeze bond between the pile and soil has been broken. Accordingly, the tangential frost heave force by the first approach is several times greater than by the second approach.

A layer of non-frost-susceptible soil above the frost-susceptible soil in the active layer can greatly increase the frost heave impact on the foundation.

Two widely used methods of design of foundation stability from frost action were compared. The first method is implemented by Russian building code (SNiP method), and the second by the manual "Arctic and Subarctic Construction Foundations for Structures" (TM-5 method). The SNiP method is reliable in most typical situations. However, it is not reliable in some special cases; for example, at sites where non-frost-susceptible soil overlays frost-susceptible soil in the active layer. We suspect that the SNiP method underestimates the tangential frost heave force acting on a steel pile, especially a pipe pile with unfilled inner space. The TM-5 method provides a safe design for practically any situation, but for most of them it greatly over estimates the pile length. It can make pile foundations unfeasible and impractical when in reality they are the best solution.

## Acknowledgments

Comments and suggestions made by reviewers of the paper were very helpful and are gratefully acknowledged.

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