THAW SETTLEMENT AROUND A BUILDING ON WARM ICE-RICH PERMAFROST

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Abstract

A storage building was constructed in Hay River, NWT, in 1967. The subsoil consisted of up to 7m of ice-rich clay permafrost underlain by unfrozen, hard clay till. In 1973 a maintenance shop extension was constructed. Both buildings consist of metal frame structures with structural slabs supported on piles based in the till. To this date there is no evidence of any movement of any part of the structure, however, considerable thaw settlement beneath and around the building over the years has required expensive addition of material and repaving around the building to restore grades. Settlement is still occurring at approximately 10 cm per year. Soils data from 1989 boreholes indicate that thaw is complete adjacent to the building. Settlement is expected to continue for some time. Geothermal modelling predicts the ultimate thaw bulb around the building and analyses predict the total and remaining thaw settlement. Downdrag forces are assessed with respect to the capacity of the piles. Measures are proposed to reduce the impact of subsequent settlement.

Résumé

Un entrepôt a été construit dans la ville de Hay River, dans les Territoires du Nord-Ouest en 1967. Le sol sur lequel repose la structure consiste en un pergélisol argileux riche en glace d'une épaisseur atteignant 7 mètres et reposant sur un till argileux ferme et non-gelé. Un atelier d'entretien y fut annexé en 1973. Les deux bâtiments ont été construits en utilisant des structures de support métalliques ainsi qu'une semelle supportée par des pieux, lesquels pénètrent dans le till. Jusqu'à présent, aucun signe de mouvement n'a été observé sur les différentes parties de la structure, cependant, des tassements considérables dûs au dégel sous et autour des bâtiments pendant des années ont nécessité des dépenses importantes pour le matériel de remplissage et de pavage pour rétablir les niveaux originaux. Les tassements se manifestent encore de façon continue à un taux approximatif de 10 cm par année. Les données provenant des trous de forage exécutés en 1989 indiquent que le dégel est complet près des murs de fondation du bâtiment. Des tassements additionnels devraient être observés encore pour une certaine période de temps. La modélisation géothermique prévoit la zone de dégel maximale autour du bâtiment et des analyses prévoient le tassement total et celui à venir. Des forces d'enfonçement sont assignées en vertu de la capacité portante des pieux. Des solutions pour réduire l'impact des tassements éventuels sont proposées.

Introduction

A shop/stores building was constructed in 1967 at the Vale Island Coast Guard Base, Hay River, NWT, Canada. The metal frame structure and structural floor are supported on piles based in an underlying unfrozen stiff to hard clay till at a depth of 10 to 12 m. The structure itself has not been subject to significant settlement. The area surrounding the building has however undergone considerable settlement due to the thawing of ice-rich permafrost overlying the till. Substantial maintenance work has been required to build up and repair the paved area around the building. A geotechnical investigation was conducted to determine if the settlement will continue much longer and if there is some means of preventing or retarding the settlement.

Site history

Hay River, is situated on the south shore of Great Slave Lake, N.W.T. (Fig.1), in the southern fringe of the discontinuous permafrost region (Brown, 1970). The mean annual air temperature is -3.6 °C (freezing index 3023 DDC; thawing index 1739 DDC). Permafrost is absent or degrading over much of the surrounding region. The initial site development consisted of a stores building, constructed in 1967. Three boreholes drilled in 1966 and 1967 identified ice-rich permafrost beneath the site, underlain by unfrozen till. The structure was founded on steel pipe piles based in the clay till unit. The building was constructed with a 1.2 m crawl space with 50 mm board insulation on the underside of the structural floor slab. Settlement of the base of the crawl space occurred as the ice-rich clays thawed. As the base of the crawl space settled beneath the exterior grade beams, the exterior soil started sloughing in causing major settlement around the exterior of the building and impeding access to the building. In order to prevent the soil from sloughing into the crawl space, sand was added to the crawl space in 1973.

An extension, consisting of an automotive and maintenance shop building, was constructed in 1974. The extension is founded on concrete piles based in the clay till unit, with a similar crawl space beneath the structural floor.

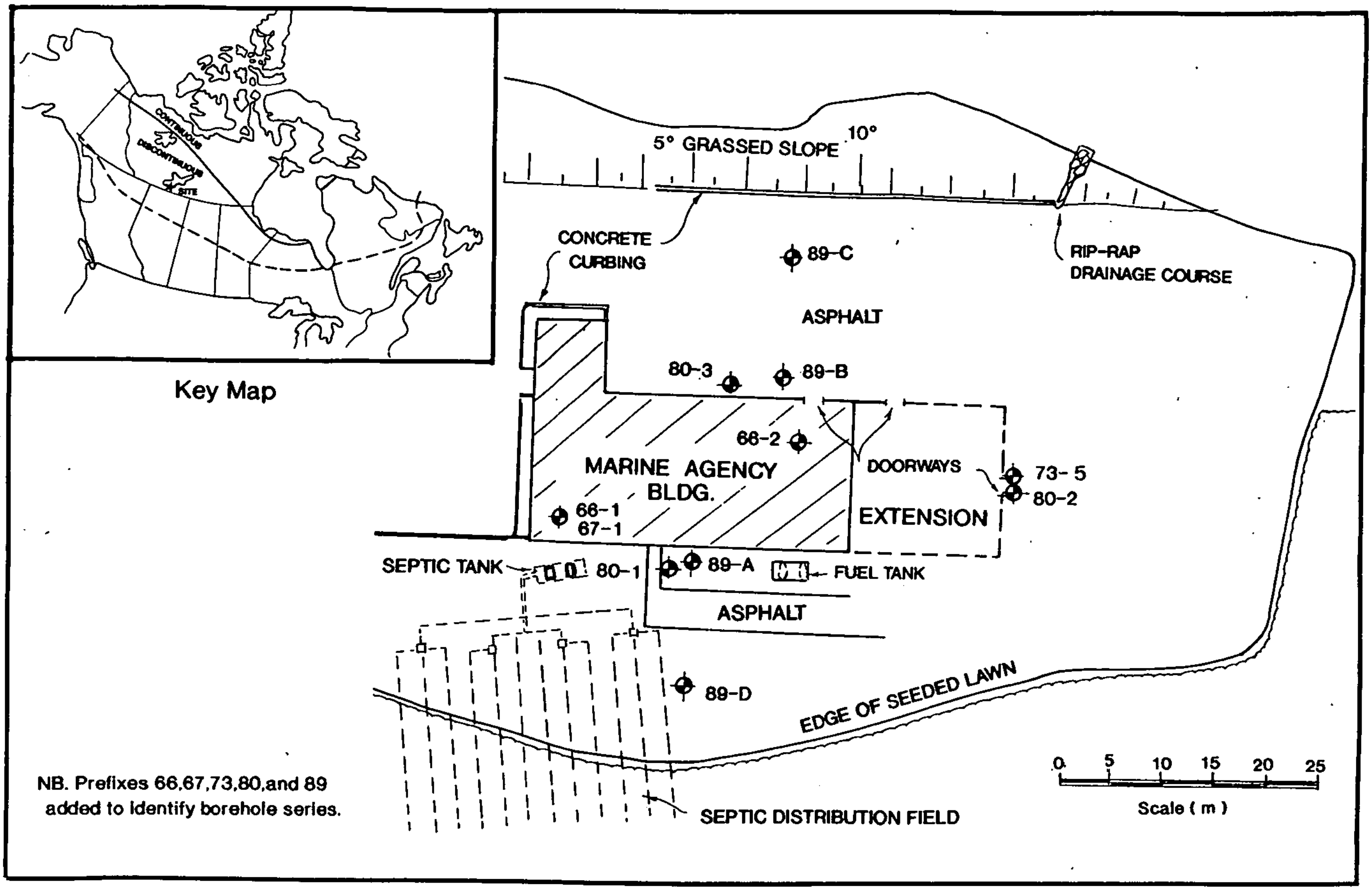

Figure 1. Site plan.

In 1980, three holes (Fig.1) were drilled to investigate the thaw settlement. The permafrost appeared to have completely degraded adjacent to the south side of the building. On the north and east side of the building there was about 2.5 m of permafrost remaining at depth. Clearly more thaw settlement was to be anticipated.

Four boreholes (Fig. 1) were drilled in 1989. Standard penetration tests were conducted at regular intervals and several undisturbed samples were obtained. Details on the subsurface conditions are incorporated in the ensuing discussion. Eleven-bead thermistor strings were installed in Boreholes A and B. The thermistor beads were concentrated in the 4 to 9 m depth range where it was thought permafrost might still remain.

Recent settlement observations

Settlement since late 1987 has been observed at three doorways, indicated on Figure 1. Settlements measured in March 1989 indicate that the ground adjacent to the building is settling as much as about 0.1 m per year. Settlement to date beneath the stores building is difficult to quantify due to the added sand. It is believed that in 1973 up to 1.5 m of sand was added. If this is the case, up to 2m of settlement may have occurred beneath the interior of the original building.

Additional fill has not been placed in the crawl space of the shop extension except for gravel placed against the exterior walls to reduce external sloughing. The original insulation had been placed beneath the floor of the crawl space. Pieces of the insulation are still visible attached to the sides of some piles. The total settlement was measured at several locations as the displacement of the insulation. Settlements ranged from 0.5 to 0.6 m along the north wall, 0.6 to 0.9 m in the central area, to as much as 1.1 m near the south wall. Based on these measurements, the ground on the south side of the shop building appears to have settled from 0.3 to 0.6 m more than the north side. These data represent the total settlement to date since sand has not been added throughout this crawl space.

Ground conditions

All of the previous and present boreholes drilled on this site have encountered a typical soil stratigraphy; a 7 to 9 m unit of clay and sand over the till. A sand or gravel layer varying in thickness from zero to 2.4 m has been reported in the 10 boreholes drilled around the building. Permafrost was encountered in all three boreholes drilled in the immediate area of the building prior to site development. The base of the permafrost was as deep as 9.7m in the 1966 boreholes. Moisture contents in the medium plastic silty clay ranged from 27 to 99 % (by weight) with an average of about 41 %. Visible excess ice was noted in the form of lenses.

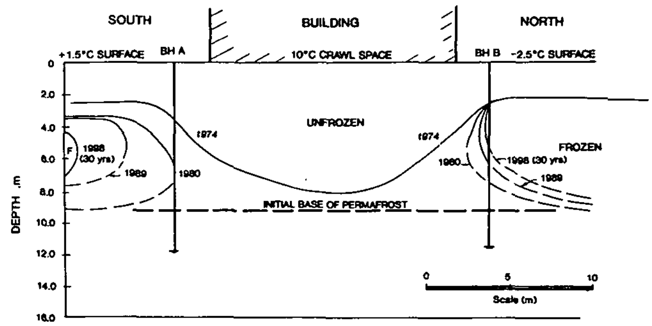

Figure 2. Predicted progression of thaw.

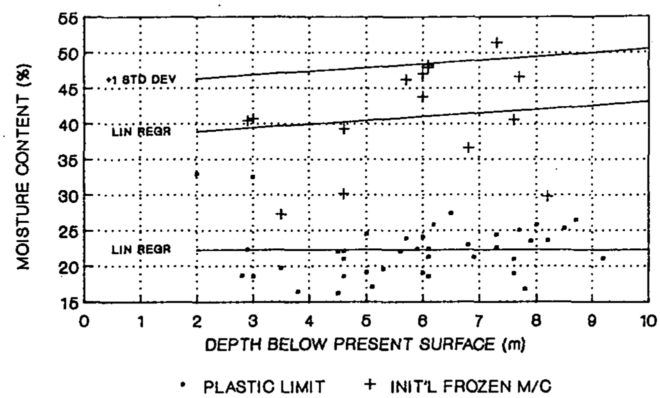

Figure 3. Initial moisture profile.

Boreholes A to D were drilled in March, 1989. Underlying a gravel fill was a unit of medium plastic silty clay with occasional pockets and lenses of sand and was soft to firm, with N values ranging from 5 to 10, for the most part. Moisture contents ranged from 24 to 40% with an average of about 30%, however, moistures were notably higher in Borehole B. In Borehole D, beneath frozen gravel fill and organics, the clay was unfrozen. Moisture contents ranged from 23 to 31% for an average of about 27%.

In Borehole C, beneath frozen sand and gravel fill, the clay was frozen to 6.9 m and ice-rich from 3.8 to 6.9 m. Below 6.9 m, the clayey silt unit was unfrozen and similar to the material encountered in the other boreholes. In Boreholes A and C the silty clay unit was underlain by a medium to coarse grained, loose to compact wet sand. Underlying the sand was a very stiff to hard, reddish brown, unfrozen silty clay till. N values ranged from 23 to greater than 60.

Permafrost degradation

The thermistor strings installed in Boreholes A and B have indicated that all of the permafrost is now degraded at these locations. Borehole C, drilled in the paved area at 17 m north of the north side of the building, indicates little permafrost degradation since all of the active layer appeared to be refrozen by March 10, 1989. The base of the permafrost, logged as 6.9 m is shallower than recorded in previous 1966 boreholes and may reflect some degradation. It is surprising that degradation is not more advanced beneath this asphalt pavement in Hay River.

Geothermal analyses were conducted on an in-house 2-D finite difference thermal model to simulate the present thaw and project further thaw around the building. Assumed surface temperatures represented the aspect of the building as follows:

Location	Assumed Mean Annual Surface Temperature °C
South side (warm)	+1.5
Crawl space	+10
North side (cold)	-2.5

The progression of the thaw bulb from the time of the original construction is shown (Fig.2) at year 6 (1974), year 12 (1980), year 21 (1989) and year 30 (1998). The difference in thaw depth between the warmer and cooler sides of the building is quite notable. The thaw on the south side continues to progress to some distance from the building. Borehole D indicates complete thaw, 14 m from the south side of the building, however, the effect of the septic field is not determined and is not represented in the thermal simulation.

On the north side, there is a limited extent of thaw, with little predicted increase between years 1989 and 1998. This situation appears consistent with the limited degradation evident in Borehole C. As plotted on Figure 2, Borehole B might have expected to encounter frozen ground according to the predictions. This, however, is considered to be within the accuracy of the thermal analysis, especially for lateral degradation.

Thaw settlement

Prediction of thaw settlement is based upon the following relationship:

$$\varepsilon = A_o + p'\, m_v$$

where ε = thaw strain
A_o = the initial thaw strain
p' = the effective stress
m_v = the coefficient of compressibility

Values for A_o and m_v are available in the literature for various soil types (Hanna et al, 1983). Using the relationship for plastic clays, the thaw strain for the full frozen section can be estimated.

The initial soil profile consisted of 7 m of ice rich clay, based on the 1966 boreholes. The frozen, silty sand gravel beneath the clay is ignored in the settlement analysis, since it appears inconsistent across the site. Allowing for varying thickness of fill over the clay, Table I shows the total predicted average thaw settlement at final equilibrium. These predictions are based upon the average, initial moisture content profile of the frozen clay (Fig.3). Based upon previous experience, using the average moisture content is considered realistic to determine the average settlement. If the average plus one standard deviation is taken as the initial

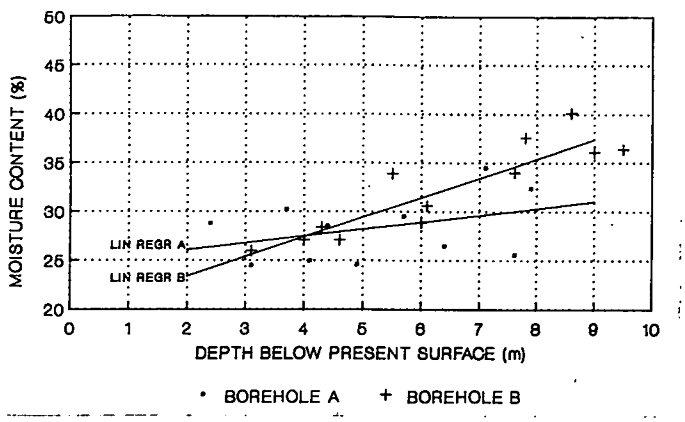

Figure 4. 1989 moisture profiles.

frozen moisture content the predicted total settlement becomes greater. It is possible that locally settlement could reach the higher values given in Table I, however, the lower values should provide a better average prediction for the central area of the thaw zone.

Settlement along the south side of the building would be expected to approach these values, however, along the north side there may be slightly less settlement due to the limited thaw beyond the building. At this stage, settlement should be more advanced beneath the interior of the building as the ice-rich clay beneath the centre of the building has been thawed for about 15 years.

By considering the present moisture contents of the thawed clay, it is clear that not all the settlement has occurred at the edges of the building. Any moisture content in excess of about 25 %, indicates the soil is still under-consolidated. A simple approximation of the remaining settlement can be made by considering the present moisture profiles for Boreholes A and B,(Fig.4). Table I gives the predicted remaining exterior settlement, based on the thaw settlement approach, under the present surface loading conditions.

It is always difficult to accurately predict the time to complete settlement in clays. An approximation can be obtained from the following relationship.

$$t = T_v \frac{H_2}{c_v}$$

where
t = time to a certain degree of consolidation (say 98 %)
T_v = time factor for the selected degree of consolidation
H = the length of drainage path
c_v = coefficient of consolidation

In the absence of test data for this project, c_v can be assumed from textbooks and MacKenzie Valley test data (McRoberts et al, 1978). Using values of c_v between 1×10^{-2} and 1×10^{-3} cm²/sec, the predicted time to complete settlement ranges from 5 to 30 years. This prediction does not account for the thawing process which has taken about 10 years beneath the centre of the building and longer around the edges of the building. The time to completion of remaining exterior settlement can be expected to be in the order of 10 to 15 years from now.

Downdrag on piles

The 300 mm pipe piles were driven to a total depth of up to 16.5 m. At that depth the ultimate pile capacity is calculated at 328 kN. The ultimate capacity of the 400 mm concrete piles beneath the shop building is calculated at 520 kN. Based on normal floor design loads for such building usage and the given pile spacing and grouping, the design loads on interior piles have been computed and are shown in Table II. (For the stores building there are groups of 5 and 6 piles for interior loads.) It is standard practice to design pile foundations at 30 to 40 % of the ultimate capacity. Therefore, the interior piles appear adequately designed for the structural loads.

The downdrag load on the piles has been calculated using the effective stress approach. The actual skin friction, s, is given by

$$s = \sigma_v K \tan \phi'$$

where
σ_v = average effective vertical stress
K = the horizontal earth pressure coefficient
ϕ' = internal angle of friction
$K \tan \phi'$ = 0.2 for clay on steel
= 0.3 for clay on concrete,sand on steel and concrete

Table I. Thaw settlement predications.

Location/Condition Thickness	Gravel Mean (m) Moisture	Total Predicted Settlement, m Mean + 1 Std. Initial Content	Dev. Moisture	Remaining Exterior Settlement, m South side (Borehole A)	North and east side (Borehole B)
Initial, crawl space	0.8	1.8	2.4	0.7	0.8
Initial, adjacent to building	2.0	1.9	2.5	0.8	0.9
Present, away from building	2.4	2.0	2.6	0.8	1.0
Present, adjacent to building	3.0	2.0	2.7	0.9	1.0
Present, crawl space	3.8	2.1	2.8	0.9	1.1

Table II Design and Downdrag Loads Versus Ultimate Capacity

Building Group Capacity kN	Ultimate Load kN	Design Floor load/Ult. load %	Design Downdrag kN	Maximum Load kN	Total Ult.Cap %	Total Load/
Stores	(5) 1312	366	28	(5) 600	966	74
(6) 1574	366	23	(6) 720	1086	69	
Shop	520	185	36	200	385	74

N.B. Assumed group capacity at 0.8 efficiency.

perimeter, the consolidation process is likely at a maximum rate now. Under the present conditions, it is expected to be in the order of 10 years or so before settlement might become noticeably reduced so as to be less of a maintenance problem.

Using the assumed values for K tan ϕ', the maximum downdrag loads have been calculated. The total loads on the piles/groups are compared to the ultimate capacity in Table II. The downdrag loads are assumed to act fully on each pile within a pile group.

It would appear that some piles beneath each building may be loaded to over 70% of their estimated ultimate capacity. If the loads are indeed this high, some foundation movements might be expected.

It can be reasonably assumed that the downdrag forces are at or close to the maximum at this stage since thawing appears to be complete around the piles. Since there is no present evidence of structural damage there should be a low potential for future structural problems. This structural and downdrag loading requires more accurate assessment if certain recommended remedial measures are to be implemented.

Conclusions

The main question at the outset was the extent of permafrost degradation. Since thaw appears to be completed immediately adjacent to the building, a thermal solution would have limited effect. Based primarily on the structural performance of the building to date, the downdrag load on the piles resulting from the thaw settlement appears to be acceptable. Some remedial measures may tend to increase the downdrag, however, Table II is expected to represent the worst case for downdrag loading.

It appears that in the order of 1 m of settlement has yet to occur around the perimeter of the building. The timing for this remaining settlement is difficult to predict with accuracy, however, it is probably in the order of 10 to 15 years. Because the thaw is probably only completed quite recently, especially along the north side of the building.

Remedial concepts

In order to reduce the impact of further settlement on the access to the building certain basic concepts have been considered:

- reduce the total amount of remaining settlement
- reduce the rate of the remaining settlement
- accelerate the remaining settlement so the problem ceases sooner

By removing sand from the crawl space and installing lightweight backfill around the exterior, a small reduction in the remaining settlement might be achieved. However, the cost-benefit is expected to be marginal. Therefore, the following concept has been proposed to attempt to accelerate the remaining settlement. A series vertical drains could be inserted into the clay deposit at such spacing as to greatly improve the drainage of the excess water out of the clay. The intention would be to cause the majority of the remaining settlement to occur in (say) 2 to 3 years so that at the end of that period, restoration of exterior grades might be closer to final. Acceptance of temporary access ramps or steps would be required during this period. Extra shoring will be required to reduce sloughing into the crawl space.

Based upon monitoring of the settlement rate during this process, it should be possible to predict when subsequent settlement would be negligible. The exterior grade should then be built slightly high to allow for some minimal remaining settlement. It is expected that subsequent maintenance could be carried out using small hand-held equipment and cold mix asphalt.

Future research needs

It would be useful to know how negative skin friction actually develops during thaw settlement of an ice-rich fine grained soil. Initially, the excess pore pressures would actually result in reduced friction, however, eventually there is the consolidation of the soil as pore pressures dissipate. The friction could conceivably build up considerably as the upper layers consolidate and therefore stiffen, while the lower layers are still thawing and causing settlement of the whole soil mass.

Knowledge on the development of downdrag on a group of piles would be especially valuable to this project.

Acknowledgements

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References

BROWN, R.J.E., 1970. Permafrost in Canada, University of Toronto Press.

HANNA, A.J., R.J. SAUNDERs, G.N. LEM & L.E. CARLSON, 1983. Alaska Highway Gas Pipeline Project (Yukon Section) - Thaw settlement design approach. Proceedings of the Fourth International Permafrost Conference, Fairbanks, Alaska

MCROBERTs, E.C.,E.B. FLETCHER & J.F.NIXON, 1978. Thaw consolidation effects in degrading permafrost. Proceedings of the Third International Permafrost Conference, Edmonton, Alberta.