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Investigation by Cone Penetration Tests of Piled Foundations in Frozen Soil Maintained by Thermosyphons

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Abstract

Infrastructure built on permafrost may experience differential settlement and deformation due to permafrost degradation caused by many factors such as climate change, man-made impact, redistribution of snow cover, alteration of subsurface water flow, etc. Even a small change in the temperature of frozen soil can significantly alter (increase or decrease) the mechanical properties of the soil and hence the bearing capacity of piles. Modern Cone Penetration Testing (CPT) provides direct measurement of soil resistance and temperature in permafrost, thereby permitting the validation of pile capacity in permafrost conditions. An example application of CPT tests to diagnose the condition of piled foundations at the Salekhard College, Western Siberia, Russia, is described. The frozen soil is maintained by thermosyphons that are exposed in the crawl space below the building. CPT tests were completed from within the crawl space to measure cone resistance q_c [MPa], sleeve friction f_s [kPa] and temperature T [°C] at several locations close to the thermosyphons and piles. The data was obtained both within the zone of influence of the thermosyphons and at distance from the thermosyphons and piles, to provide "baseline" measurements. Based on this data; 1) the cooling effect of the thermosyphons was directly measured (soil temperature decrease), 2) the pile bearing capacity could be estimated and 3) pile capacity could be compared at different locations beneath the building. In future projects it would be useful for reference during subsequent verification of foundation conditions to obtain data on the baseline soil properties and temperature conditions before installation of the piles and thermosyphons.

Keywords: Cone Penetration Testing; Frozen Soil Foundation Diagnostics; Frozen Soil Temperature; Permafrost Soils; Pile Bearing Capacity; Thermosyphon.

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1. Introduction

Cone Penetration Testing (CPT) is one of the main geotechnical methods used in the profiling and measurement of soil conditions in geotechnical investigations (non-frozen conditions), both on land and subsea [1]. The CPT method is relatively simple, fast and highly repeatable, generally providing reliable and high quality data. CPT testing of frozen soils is still relatively new. One reason is an apparent misapprehension among permafrost engineers that it is technically impossible to penetrate the cone because of the high strength of frozen soils, perhaps based on limited experience with earlier generations of equipment. The successful application of CPT testing in frozen soil, such as the case history described in this paper, shows this concern to be unfounded [2].

CPT on permafrost soil was first time performed in 1974 by the Canadian researcher B. Ladanyi [3] in the Prudhoe Bay area of the Beaufort Sea. Only cone resistance was recorded during the test. The total thrust of CPT equipment was 30 kN (~3 ton) that proved insufficient to achieve a large penetration depth. Later in 1977-1978 the CPT on permafrost project was continued by Cold Region Research Engineering Laboratory (CRREL) researchers [4] with an upgraded cone for permafrost investigation. The depth of penetration was limited only by the load capability of the apparatus, in this case 133 kN force (~13 ton). The depth of penetration achieved was between 10 and 12 meters, depending on the location. Similar tests were performed in Russia during the 1980-s by Isaev in Vorkuta and Labytnangy, in permafrost soils with a temperature between 0 and -2.1^{o}C [2]. Further scientific studies in Canada with CPT testing on permafrost were completed by Fortier [5] to study the cryostratigraphy of permafrost. The refusal of cone penetration was reached at a depth of 15.8 m, where bedrock was located. Fortier used the most recent CPT based equipment, which included additional sensors for temperature, pore pressure and electrical conductivity. However, the method has never been used for detection of frozen soils under existing buildings or for the evaluation of thermosyphon performance, as described in this paper.

The application of CPT testing has great potential for geotechnical surveys in permafrost and cold region environments. The method tests the soil in-situ, in a highly repeatable manner (minimal operator influence), giving results that are of high value in view of the challenges and difficulties often faced during field work at such inhospitable sites. The main alternative is drilling boreholes, collecting soil samples and testing them in a special cold laboratory. That approach is strictly limited by the number of boreholes, the number of frozen soil samples and transportation requirements for frozen soil samples that must be preserved in frozen state from the moment of soil sampling to the moment of lab testing. Indeed, the degree of sample disturbance caused by temperature change during this process is not at all well defined for frozen soil samples.

The repeatable and reliable data from CPT testing that may be obtained at precise locations close to ground structures makes the test highly useful for geotechnical control. Indeed the CPT test is perhaps the only method applicable for frozen soil diagnostics in permafrost conditions. Geotechnical control is regulated by a new standard issued in Russia, STO 36554501-049-2016 «CPT Application for Frozen Soil Foundation in Permafrost and Cold Region Environment» [9]. This article provides an example of how frozen soil foundations below a building of the “Yamal Polar Agricultural College” were verified by CPT testing.

2. Yamal Polar Agricultural College

The "Yamal Polar Agricultural College" was built in 1977. The building is two stories high and supported on a piled foundation. The piles are 8 m to 10 m long and connected via a reinforced concrete capping, as shown in the archive documentation. Differential settlement of the building started to develop around 2010, resulting in settlement of the south-east corner of the structure and cracking of the walls (Figure 1). Use of the building was stopped for safety reasons. Cracks subsequently developed inside the building (Figure 2) and are being monitored (Figure 3). Additional structural support using angle steel bars was introduced in the affected areas of the building to protect against potential collapse of the structure (Figure 4).

Figure 1: Differential Settlement of the Building of Yamal Polar Agricultural College

Figure 2: Crack in the Wall of the Building of Yamal Polar Agricultural College

Figure 3: Deformation Markers on the Wall

Figure 4: Reinforced Angle Steel Bars Installed for Support

An engineering survey was performed in 2014 to determine the main reason(s) for the differential settlement of the building. The survey concluded that the settlement was caused by permafrost thawing in the soil below the building where the piles were installed. The engineering survey report indicated a pile bearing capacity 11 tons, based on recommendations and tables provided in [7]. This value of pile bearing capacity is about half the design value of 20 tons required to provide safe foundation support.

Remedial work recommended in the survey report was implemented in 2015. Supplementary structural reinforcement of the building was installed, including wall braces. The exposed structural elements of the foundations were also reinforced and the crawl space below the building cleaned to permit free circulation of air. A set of thermosyphons 8 meters long and 25 mm in diameter were installed near each pile to restore the temperature in the foundation soil that was presumed to have increased (Figure 5). By December 2016 the thermosyphons had served one full winter cycle.

Figure 5: Thermosyphon near Pile

GEOINGSERVICE LLP (Fugro Russia) was commissioned to perform CPT testing in December 2016 to gauge the performance of the thermosyphons and assess the frozen soil conditions and piled foundations below the building. The CPT testing was performed between 1st and 3rd December, 2016. The ambient air temperature during the site work varied between -20°C and -26°C. Tests were completed at three CPT locations based on reconnaissance of the building and the locations of the piles and thermosyphons (Figure 6). The tests extended to a maximum depth of penetration 12.1 m, extending below the end of the piles.

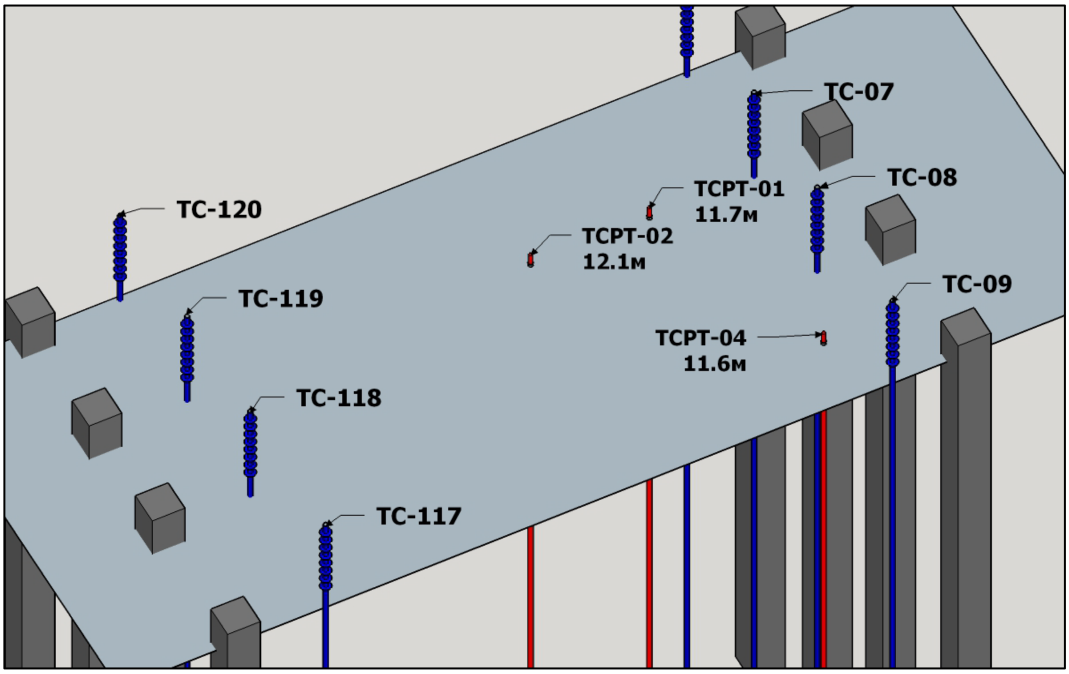

Figure 6: Location of Temperature Cone Penetration Tests (TCPT), Thermosyphons (TC) and Piles

3. Mobile CPT Equipment

3.1. Description

Fugro owns and applies the full range of CPT equipment available for geotechnical investigation on land and subsea. For use on land, CPT equipment is usually installed in a heavy truck (in a cabin protected from the weather), or on railway cars where railtrack investigations are required. Mobile CPT equipment is also used and can be carried to the test location and operated even in conditions with low space and headroom [6]. A wide range of cones with different sensors are also available providing great versatility for subsoil detection and measurement.

CPT equipment is most widely operated from a CPT truck (20 ton) or drill rig. Such equipment is not suitable for areas with restricted access such as below buildings. Therefore, mobile CPT equipment was used for the present investigation, the first experience testing permafrost this way. The mobile CPT equipment was produced in Holland and has maximum thrust capacity 100 kN (Figure 7), and is powered by an oil-pump.

Data were collected using an acquisition system installed on a laptop (Figure 7), using a scanning frequency 1 Hz. All the data can be displayed on the laptop screen during a test, permitting operational decisions and adjustments during the test and at eventual refusal due to excess cone resistance.

The ground surface in the crawl space was entirely covered by concrete. At each CPT test location, the concrete was drilled by a perforator with a hard-alloy cutter down to the level of the soil. The depth of drilling was between 120 mm and 180 mm, the thickness of the concrete foundation.

Figure 7: Mobile CPT Equipment during Testing (TCPT-04)

3.2. Constraints and Limitations

The height of the crawl space below the building varied between 1.9 m and 2.2 m. Despite this restricted head space, the mobile CPT could be used without any additional measures being taken. The thrust in the mobile CPT was resisted by bearing structures below the building.

Usually the depth of cone penetration required to verify piled foundations is specified as the pile length plus 3 m [10]. At the Salekhard College this corresponds to 11 m given the pile length is 8 m. This depth requirement was achieved for all the CPT tests. However, for investigation purposes, the authors decided to penetrate to a maximum depth, until a common CPT limitation occurs:

- Cone resistance more than 65 MPa
- Maximum capacity of the thrust machine, reaction equipment, push rods and/or measuring sensors
- Integral inclination exceeds the value of 1^o per 1 meter of cone penetration
- Sudden increase in penetrometer inclination
- Circumstances at discretion of CPT operator, such as risk of damage to apparatus or safety of personnel.

All three TCPT tests were terminated at the maximum thrust capacity of the mobile CPT equipment, 100 kN. Solid-frozen dense sand was encountered at 11.7 m (TCPT-01), 12.1 m (TCPT-02) and 11.6 m (TCPT-04).

4. Performance of CPT

The CPT test drives a cone into the ground at a constant speed 2 cm/s and the end resistance and side friction exerted by the soil on the cone are measured at each 2 cm interval (1 Hz frequency). The measured data are the cone resistance, defined as q_c [MPa], and the sleeve friction, defined as f_s [kPa]. An additional temperature sensor is included on the cone to measure the soil temperature at each 2 cm depth, to provide this critical data for frozen soils.

In order to quantify the operation of the thermosyphons, the depth of CPT penetration exceeded the depth of thermosyphon installed to 8 m depth. The CPT tests extended to between 11.6 and 12.1 m depth (Figure 8).

In summary, the cone was pushed at a constant rate of 2 cm/s, according to requirements of [8], and the following parameters measured at 2 cm intervals:

- q_c [MPa] – cone resistance, or soil resistivity under the cone tip, as defined in [9]
- f_s [kPa] – sleeve friction, or soil resistivity along the cone sleeve, as defined in [9]
- T [°C] – soil temperature.

4.1. Cone Resistance and Sleeve Friction

A plot showing the measured CPT parameters with depth illustrates the variability in the soil profile. The magnitude of cone resistance in frozen soil varies widely, ranging from several MPa in clay soils with high temperature to the maximum measured values of 50 to 60 MPa in well-compacted and cooled sands or dense frozen clay at low temperature. The value of sleeve friction is directly related to soil type, density and soil temperature. The sleeve friction also varies over a wide range, reaching values up to several hundred kPa in soils with high ice content or density.

4.2. Detection of Engineering-Geological Elements (EGE)

Interpretation of the CPT method of field testing in soil is governed in Russia by GOST 19912-2012 [10]. Following this standard, the CPT data may be used to detect engineering-geological elements (EGE) with support from engineering-geological borehole drilling. Five EGEs were identified from the CPT tests at the Salekhard College site using the data obtained in combination with the engineering-geological drilling performed earlier in 2014, (Figure 8).

It is significant that the soil description based on engineering-geological drilling fully corresponds to the CPT data interpretation. For example, in EGE-03, which is represented by yellowish-gray silty clay, drilling also identified sandy and clayey layers, which is clearly visible on the CPT profile by the peaks in sleeve friction marked by the dotted red lines. In EGE-04, which is represented by blue-gray clay, ice lenses were detected, which is visible on the CPT profile on the typical peaks in cone resistance marked by the blue lines.

TCPT-01 shows that five EGEs may be identified in the profile down to a depth of 11.5 m, which correlates with the drilling results, (Figure 8). These soil units are as follows:

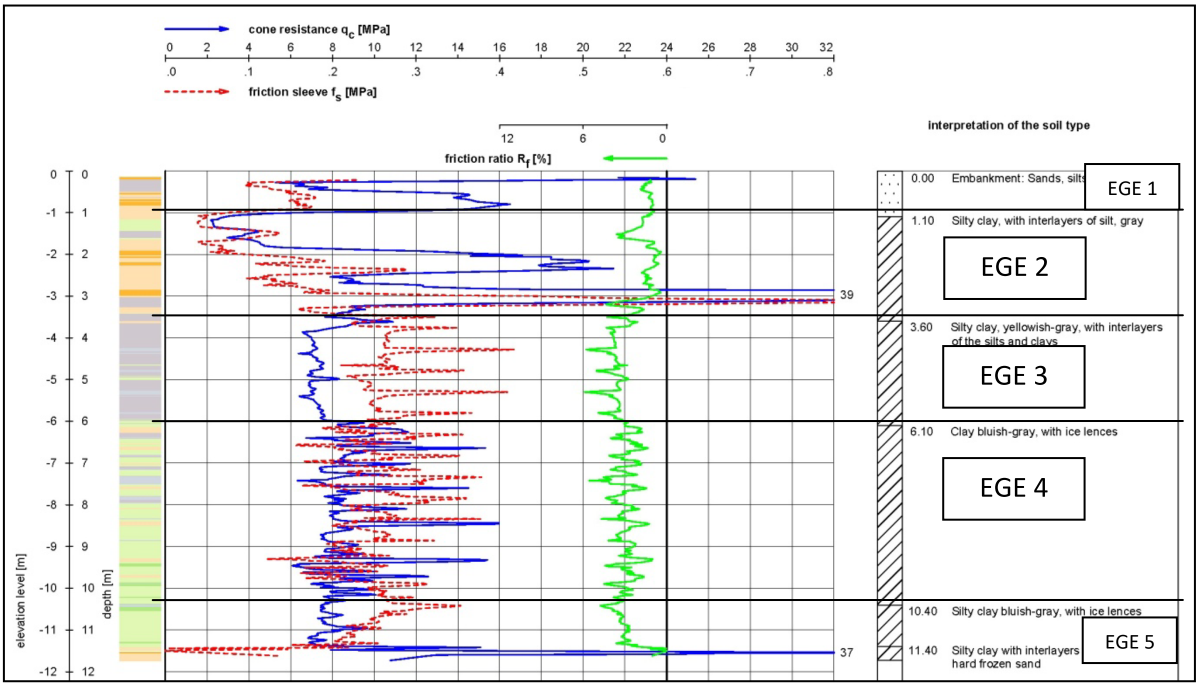

Figure 8: EGE discriminated based on CPT and engineering-geological drilling for TCTP-01

EGE 1 – Technogenic (man-made) sand, fine with admixed of sandy clay, the temperature ranges from -12.5°C to -1.2°C, shows a cone resistance 25 to 30 MPa, the sleeve friction is also high 0.3 to 0.5 MPa.

EGE 2 – Silty clay with sandy clay layers, the most heterogeneous element in the profile, since it is located simultaneously in the thawed and frozen condition, includes the boundary of the active layer. In the upper part, at depths of 1.8 to 2.0 m, it is characterized by a high temperature, up to +0.1°C and low values of cone resistance 2 to 4 MPa; sleeve friction is 0.05 to 0.1 MPa. In the lower part near the frozen/non-frozen soil boundary, a significant increase in these parameters is observed: the temperature drops to -0.5 and -1.3°C and the cone resistance increases up to 30 to 40 MPa and sleeve friction 0.2 to 0.4 MPa. At the same time, because of the ice lenses, the graph of cone resistance shows characteristic "spikes".

EGE 3 – Silty clay, yellowish-gray with clay interlayers. The element is homogeneous in its parameters, it contains a relatively smaller number of ice lenses, but a greater number of clay interlayers of small thickness, as shown by the large number of "spikes" in the graph of sleeve friction. Cone resistance is about 6 to 11 MPa, with a tendency to increase with decreasing temperature. The sleeve friction is high and varies between 0.15 to 0.25 MPa with "spikes" up to 0.4 MPa.

EGE 4 – Clay bluish-grey with high ice content. This element is characterized by a wide range of values for both cone resistance and sleeve friction, due to high ice content and the presence of ice lenses. Similar CPT data in this soil unit was seen in all TCPTs, showing greater soil resistance with decreasing temperature. This EGE has been influenced the most by the installed thermosyphons. The magnitude of cone resistance is 8 to 12 MPa, with distinct "spikes" up to 20 to 22 MPa. The sleeve friction is 0.15 to 0.30 MPa, with a wide range of values.

EGE 5 – Silty clay is bluish-gray with high ice content. Cone resistance in the range 8 to 10 MPa has low variability but with noticeable spikes in the intervals with ice content. The sleeve friction is 0.2 to 0.3 MPa, with a moderate range.

It is worth noting that the soils in EGE-3 and EGE-4 are most significant in terms of varying properties with temperature. A significant increase in measured cone resistance and sleeve friction are observed even for relatively modest reduction in soil temperature.

4.3. Temperature Measurements

It is helpful to classify two types of cone penetration - continuous and discontinuous (or static) [2]. Continuous cone penetration into the ground occurs at a constant rate, usually 2 cm/s, and penetration breaks (temporary halt) occur naturally when adding a CPT rod. Discontinuous cone penetration into the ground also occurs at a constant rate, but the breaks are carried out at a predetermined depth and static soil testing is carried out by sensing using special techniques (relaxation-creeping, dissipative, quasi-static and other tests). During the testing at the Yamal Polar Agricultural College site, discontinuous cone penetration was carried out with breaks for measuring the soil temperature, both in accordance with the express procedure (Figure 9) and the temperature stabilization method, as governed by [9] (Figure 10) .

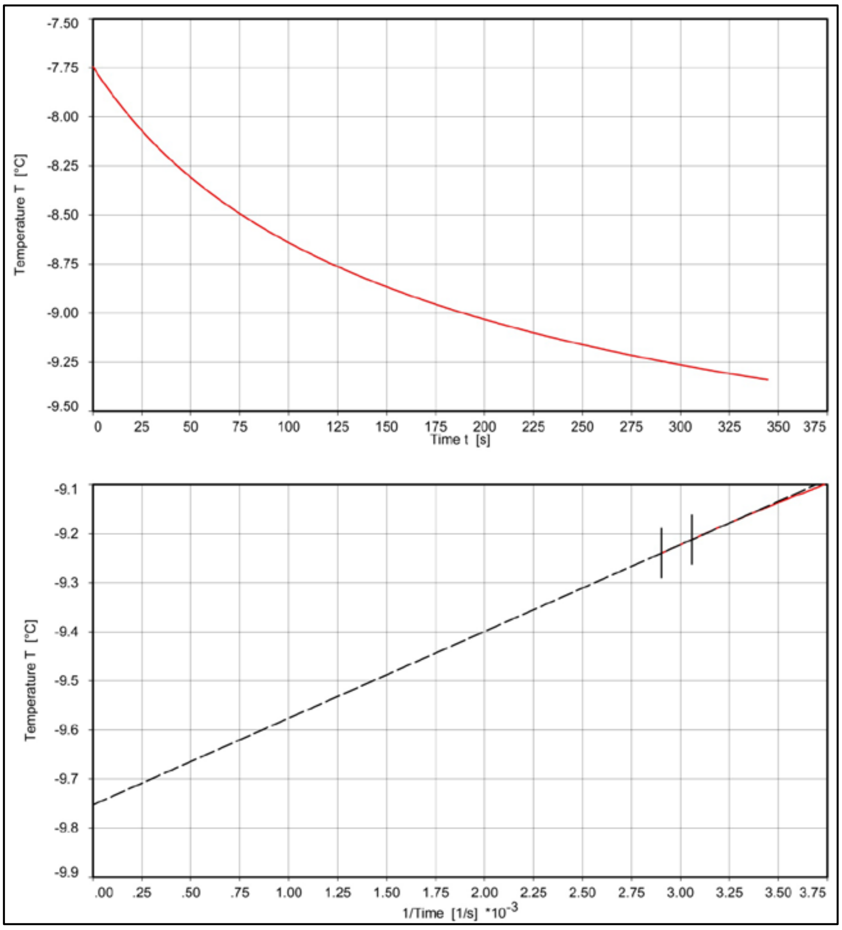

Figure 9: Express Temperature Measurement in TCPT-01 at the depth of 0.28 m

The authors provide the result of temperature measurement at a depth of 0.28 m in Figure 9. At this depth, the soil temperature was -9.73°C, since the measurement was made in early December, and the air temperature varied within the range from -20°C to -30°C. Soil at such a depth is usually not taken into account, as was the case in this study. However, it is worth noting that even such low temperatures of frozen soil was penetrable for CPT, which contradicts the generally accepted opinion that CPT is impossible for frozen soils at low temperature.

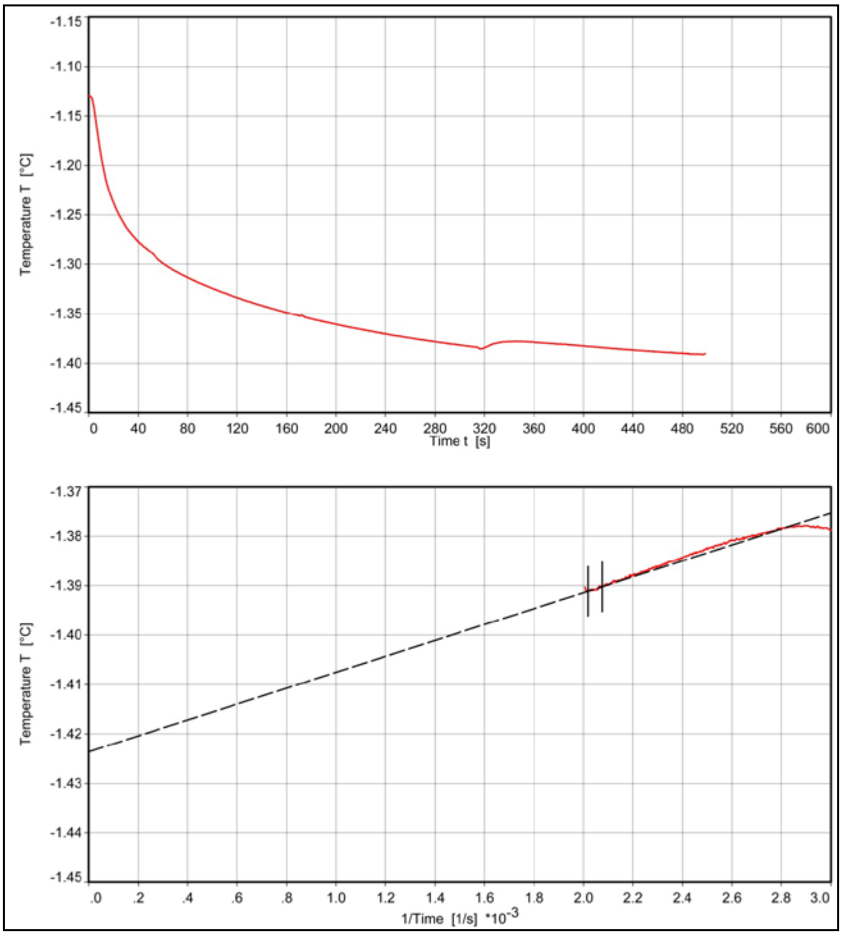

Figure 10: Temperature Measurement in TCPT-01 at depth of 5.38 m according to STO 36554501-049-2016 requirements

During the cone penetration, as a rule, the cone is heated due to friction between the cone surface area and the ground. However, in some cases, such as during the penetration of frozen clay soils, the cone may be slightly cooled. This effect has not yet been studied. Perhaps this happens due to a shift in the ice crystallization point due to the locally increased pressure. The ice may start to melt due to the increase of pressure caused by cone penetration rather than due to a rise in temperature. Ice melts, absorbs heat from the surrounding soil, which leads to a slight drop in temperature. This effect also works in sandy soils, but its appearance is overlapped by heating caused by friction. Based on Fugro’s experience, it can be concluded that the warming in frozen soils is significantly lower than in non-frozen soils. This effect has not been studied fully and requires the accumulation of more empirical data. The intensity of cone heating depends on a number of factors, such as ground state (frozen / non-frozen), soil type, soil density, cone penetration rate, etc. This effect will be discussed in a separate scientific article.

The layout of the piles, thermosyphons and TCPT locations are shown on Figure 11. The measured temperature profiles (Figure 12) show the effect of thermosyphon operation on the surrounding soil at TCPT-01 and TCPT-04. Test TCPT-02 is located at a distance of 2.05 m from the thermosyphon and provides measurement of a "reference" natural temperature profile.

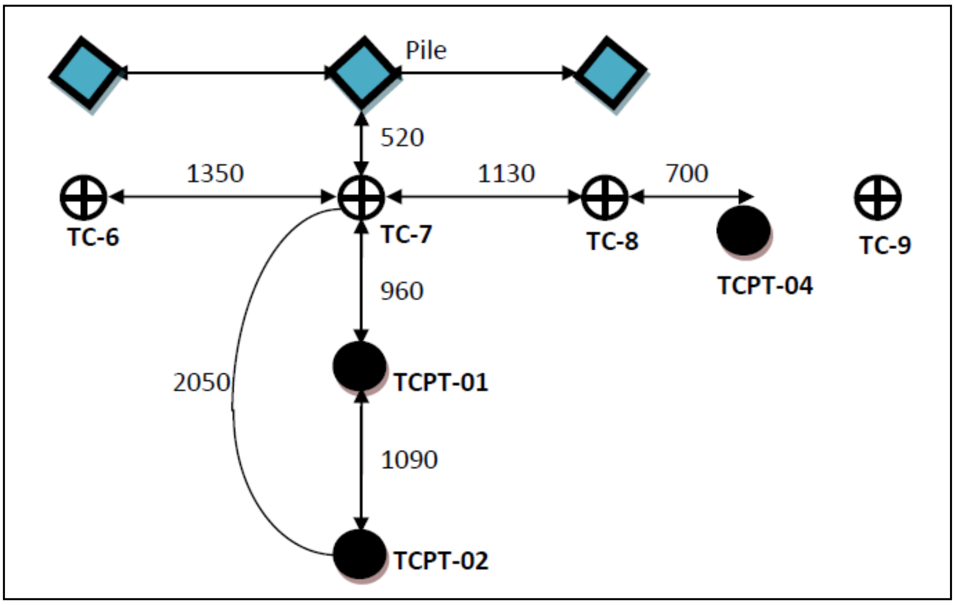

Figure 11: Location Scheme of Piles, Thermosyphons (TC) and TCPT Tests

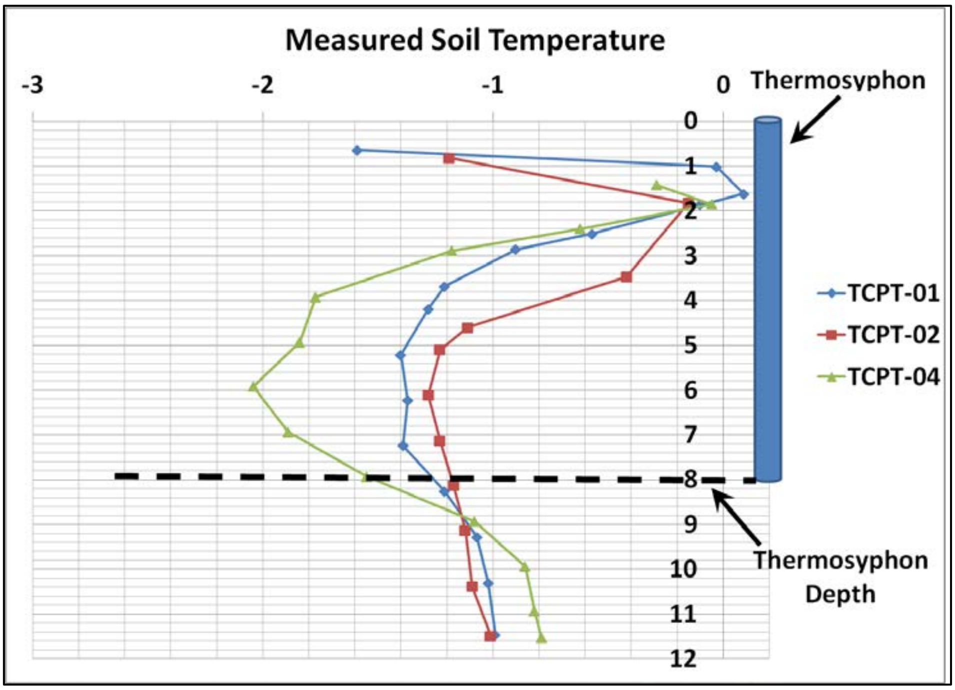

Figure 12: Measured Soil Temperature Profiles at TCPT-01, TCPT-02 and TCPT-04

At location TCPT-01, 0.96 m distant from the thermosyphon, the soil temperature is noticeably lower than the natural ground temperature. At location TCPT-04, only 0.72 m from the thermosyphon, the measured soil temperature is lower still indicating significant cooling due to the thermosyphon.

Analysis of the measured temperature values (Table 1) shows that at a depth of 2 to 3 m, the soil temperature is -0.5°C to -1.0°C. Further at the reference TCPT-02 with the depth the soil temperature changes within the range -1.0°C to -1.28°C, reaching its minimum at a depth of 6.1 m. At TCPT-01, a slight temperature decrease is observed in comparison with the reference TCPT-02 and the soil temperature varies within the limits -1.0°C to -1.4°C, reaching its minimum at a depth of 5.2 m. The greatest decrease in soil temperature was measured at TCPT-04, which is located closest to the thermosyphons, at a distance of 0.72 m from TC-8 and TC-9. So at the depth of 5.9 meters the temperature of frozen soils reached -2.04°C. Such a decrease in soil temperature is also due to the location of the point TCPT-04, where radial heat fluxes are directed toward two thermosyphons.

Table 1: Measured Soil Temperatures vs Depth at TCPT-01, TCPT-02 and TCPT-04

TCPT-01		**TCPT-02**		**TCPT-04**	
Depth, m	Temperature, °C	Depth, m	Temperature, °C	Depth, m	Temperature, °C
0.28	-9.73	0.34	-7.08	0.42	-12.56
0.80	-1.59	0.82	-1.19	0.91	-4.63
1.17	-0.03	1.82	-0.15	1.42	-0.29
1.88	0.09	3.47	-0.42	1.85	-0.05
2.02	-0.10	4.60	-1.11	2.40	-0.62
2.67	-0.57	5.10	-1.23	2.89	-1.18
3.03	-0.90	6.11	-1.28	3.92	-1.77
3.84	-1.21	7.13	-1.23	4.93	-1.84
4.35	-1.28	8.13	-1.17	5.92	-2.04
5.38	-1.42	9.14	-1.12	6.93	-1.89
6.39	-1.37	10.38	-1.09	7.93	-1.55
7.40	-1.39	11.49	-1.01	8.93	-1.08
8.42	-1.21			9.94	-0.86
9.44	-1.07			10.93	-0.82
11.62	-0.99				

Based on the temperature measurements, it can be stated that the thermosyphons are active (working properly) and have significantly cooled the frozen soil during the first winter season. However, it should also be noted that the thermosyphons have a sufficiently large "passive" part of the evaporator, and starting from a depth of 6 m, the cooling effect of thermosyphons decreases rapidly, as seen from a temperature gradient from 6 to 9 m (TCPT-04). Thus the temperature of frozen soils at TCPT-04 increases from -2.04°C to -1.1°C; at a depth of 9 m, the temperature of frozen soil is about -1.1°C at all test points.

Lowering the temperature of frozen soil by -0.76°C may not, at first sight, seem very significant. However, this magnitude of temperature reduction does result in a measurable increase in the mechanical properties of the frozen soil.

5. Temperature Monitoring

A solid plastic pipe with a diameter of 32 mm was used to install a temperature monitoring well inside the hole created by the CPT test. The pipe was sealed with a plug at the bottom of the well and has no joints, which excludes the possibility of groundwater leakage. Also, due to the small diameter of the pipe, the precision of the soil temperature measurement is increased due to reduced air convection in the well and more reliable sealing between the well casing and the soil (a very precise hole geometry and dimensions created by the CPT test).

The installation of the pipe and the thermistor chain were performed within an hour (Figure 13). In this case, the technology of temperature monitoring well installation fully meets the requirements of GOST 25358-82.

Figure 13: Temperature Monitoring Well Installed on the Studied Site

6. Results and Analysis

6.1. Comparison of the measured data on cone resistance, sleeve friction and soil temperature

The soil units EGE-03 and EGE-04 were chosen for more detailed analysis of soil characteristics. The soil units

EGE-01 and EGE-02 are located in the active layer (the layer of seasonal freezing/thawing) and do not provide reliable support for piles and are ignored in the pile capacity assessment; soil unit EGE-05 is located below the piles and does not contribute to the calculation of the pile bearing capacity.

In addition to calculating the pile bearing capacity, special interest was paid to the evaluation of the effect of the temperature field (more precisely, the lower temperature of frozen soils caused by thermosyphon influence) on the mechanical properties of frozen soils. To assess the temperature effect on the mechanical properties of frozen soils, it is interesting to compare the graphs of temperature, cone resistance and sleeve friction with depth for EGE-03 and EGE-04 (Figures 14 and 15). When constructing the graphs, the data was processed, where the value for each point at depth was calculated as a mean value, following the same algorithm as used in calculating the pile bearing capacity. This data processing "smoothes" the sharp peaks on the curves of cone resistance and sleeve friction, which makes it possible to visualize the effect we are interested in.

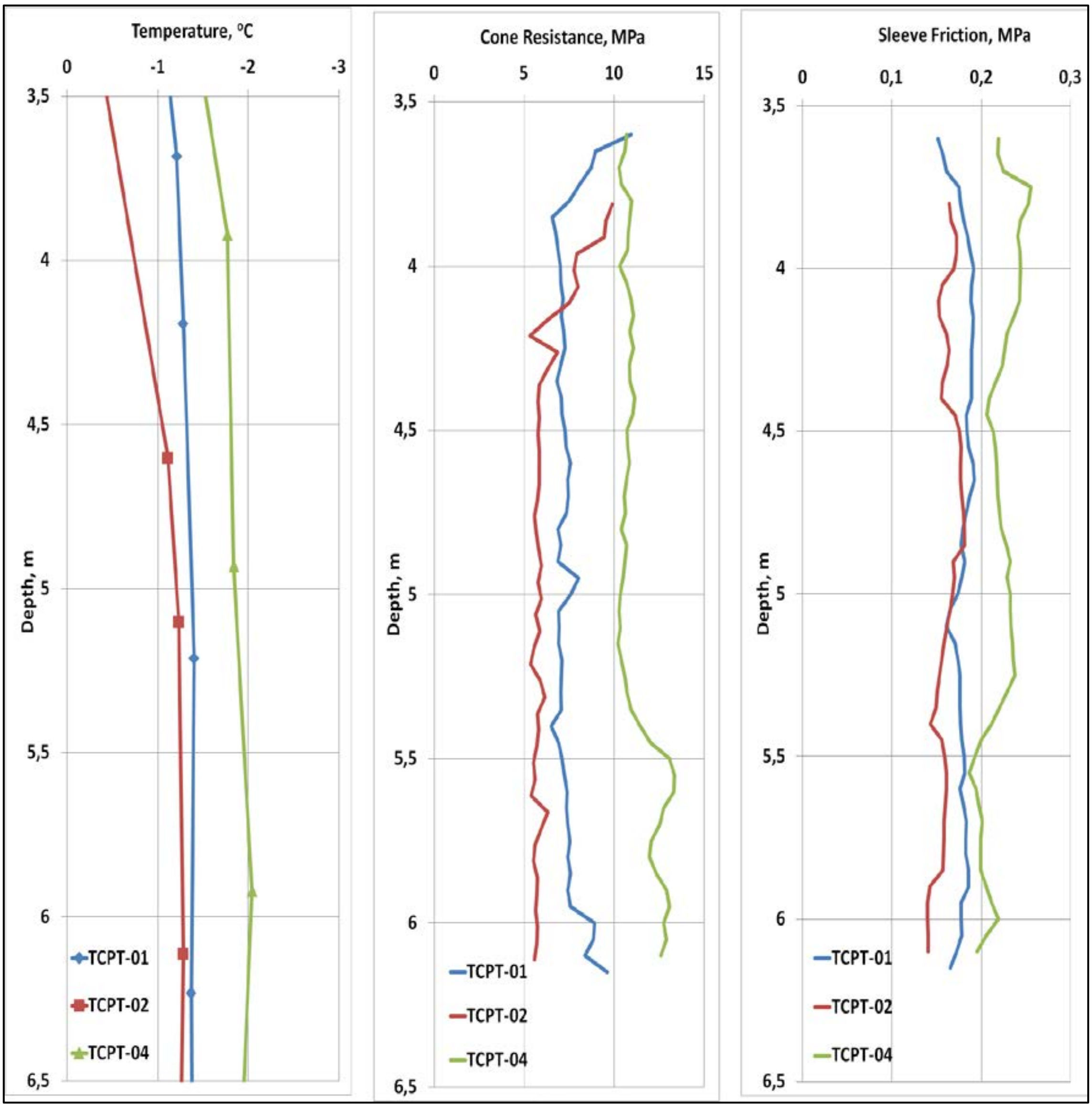

Figure 14: Temperature, Cone Resistance q_c and Sleeve Friction f_s vs Depth for EGE-03

In Figures 14 and 15, it can be seen that as the temperature decreases, the cone resistance and sleeve friction increase correspondingly. Moreover, at the test point of TCPT-04, the temperature reached the lowest values because of the closest position to the thermosyphons, which also results in the highest values of cone resistance and sleeve friction as shown on Figures 14 and 15.

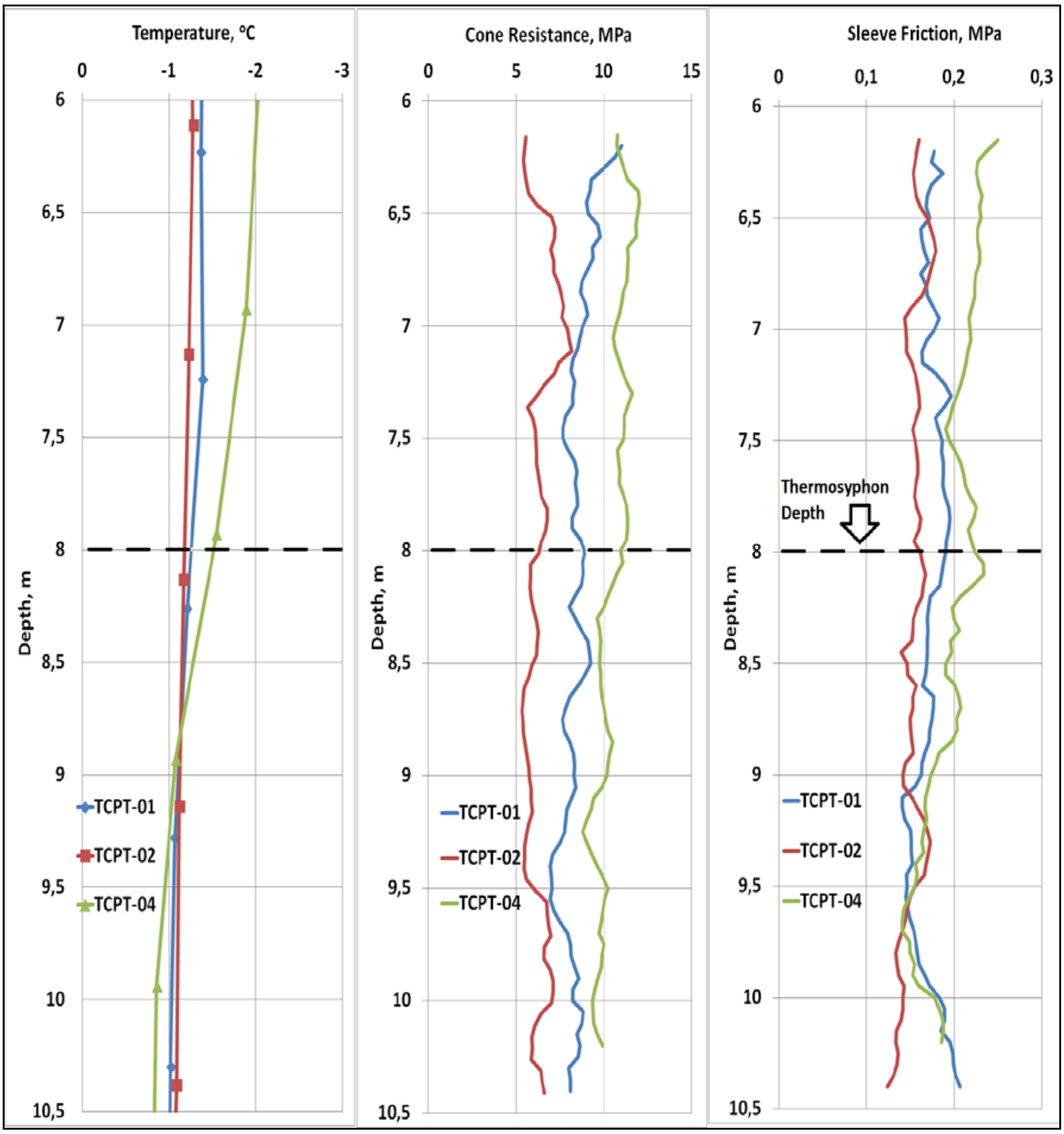

Figure 15: Temperature, Cone Resistance q_c and Sleeve Friction f_s vs Depth for EGE-04

6.2. Calculation of Pile Bearing Capacity

Calculation of pile bearing capacity was carried out in accordance with the requirements of SP 25.13330.2012 "Bases and foundations on the permafrost soils" (Updated SNiP 2.02.04-88), Appendix Л "Determination of mechanical properties and pile bearing capacity in permafrost soils based on CPT" [8]. Reinforced concrete piles with a cross section 300 mm x 300 mm were installed by direct pile driving at the site. To perform the analysis, an interpretation of the CPT data was carried out to determine the type of the soil, frozen or thawed. The calculation did not take into account the first three meters of the soil section, since the common practice of calculations usually excludes the active layer (backfill soil, seasonal freezing/thawing layer).

Table 2: Results of Pile Bearing Capacity Calculation

Depth, m	TCPT-01			TCPT-02			TCPT-04		
	Tip	Friction	Total	Tip	Friction	Total	Tip	Friction	Total
	ton			ton			ton		
1	-	-	-	-	-	-	-	-	-
2	-	-	-	-	-	-	-	-	-
3	-	-	-	-	-	-	-	-	-
4	23	5	28	24	6	30	28	8	36
5	24	13	37	20	13	33	29	19	48
6	26	21	47	22	19	41	30	28	58
7	25	28	53	22	26	48	29	40	69
8 (Pile End)	25	37	**62**	21	32	**53**	28	49	**77**
Comparison			+17%			0%			+45%
9	24	44	68	21	38	59	27	59	86
10	24	51	75	22	44	66	27	65	92
11	28	59	**87**	24	51	**75**	34	73	**107**
Comparison			+16%			0%			+42%

Calculation of the bearing capacity of a driven pile, carried out according to the CPT data (Table 2), showed that the pile 300 mm x 300 mm and 8 meters long in ground conditions, when the soil was not cooled by thermosyphons, has bearing capacity of 53 tons (TCPT-02), and in the conditions, when the soil was cooled by thermosyphon, 62 tons (TCPT-01) and 77 tons (TCPT-04). In relative terms, the increase in pile bearing capacity was 17% (TCPT-01) and 45% (TCPT-04). It should be noted that the calculation was carried out for the entire tested depth and at a depth of 11 meters the relative ratio of the pile load-bearing capacity for the three test points was 16% (TCPT-01) and 42% (TCPT-04). The decrease in these values, if compared for 8 meters and 11 meters, can be explained by two factors: the heterogeneity of the soil conditions and the attenuation of the cooling effect caused by thermosyphons with depth, since the thermosyphons only extend to 8 m depth.

6.3. Recommendations

The following engineering measures are recommended based on the obtained results:

- Continue monitoring the Building of Yamal Polar Agricultural College visually and already established deformation marks

- Establish temperature monitoring in the temperature monitoring wells which were installed at the CPT locations
- Organize a network of geodetic reference points on the building of Yamal Polar Agricultural College and surrounding territory for continuous monitoring of possible subsidence of the structure and the ground surface
- Perform a geophysical investigation by electro-tomography to identify zones of possible thawing under the building of Yamal Polar Agricultural College
- Check the actual length of the piles under the building of Yamal Polar Agricultural College.

7. Conclusion

As a result of the tests, it was determined that all the soils in the structure foundation are in frozen state and that there is no permafrost thawing to a depth of 8 m, as had been assumed by the previous investigation. The measured temperature values in the reference CPT locations range from -0.4°C to -1.3°C. Based on the calculation results, the bearing capacity of a single driven pile 300 mm x 300 mm and 8 m long is about 53 tons without taking into account the active layer; this significantly exceeds the design load of 20 tons. The thermosyphons had a cooling effect on the frozen soil and during the first season the soil temperature decreased by -0.5^{o}C to -0.8^{o}C (down to -1.0^{o}C to -2.1^{o}C in absolute values), which resulted in an increase in the pile bearing capacity up to 77 tons or 42% compared to the reference value measured in the ground.

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References

[1]. T. Lunne, P.K. Robertson and J.J. Powell. *Cone Penetration Testing in Geotechnical Practice*. London and New York: Spon Press, 2004, 312 p.

[2]. I.B. Ryzhkov and O.N. Isaev. *Cone Penetration Testing of Soils in Geotechnics.* Stockholm, Sweden: ASV Construction, 2016, 408 p.

[3]. B. Ladanyi. “Use of static penetration test in frozen soils.” *Canadian Geotechnical Journal*, № 13 (2),

pp. 95-110, 1976.

[4]. S.E. Blouin, E.J. Chamberlain, P.V. Sellmann, D.E. Garfield. “Penetration Tests in Subsea Permafrost, Prudhoe Bay, Alaska”. CRREL Report 79-7, 1979, 49 p.

[5]. R. Fortier and W. Yu, “Penetration Rate-Controlled Electrical Resistivity and Temperature Piezocone Penetration Tests in Warm Ice-Rich Permafrost in Northern Quebec (Canada)” in *Proc. Cold Regions Engineering 2012: Sustainable Infrastructure Development in a Changing Cold Environment, ASCE,* 2012, pp. 757-767.

[6]. I.S. Sokolov, N.G. Volkov and V.S. Isaev. “Cone Penetration Testing for Railways on Permafrost,” in *Proc. XI International Conference on Permafrost*, 2016, pp. 1144-1145.

[7]. SNiP 2.02.04-88 “Bases and Foundations on the Permafrost Soils” (in Russian)

[8]. SP 25.13330.2012 “Bases and Foundations on Permafrost Soils”. Appendix Л. Moscow: Minregion Rossii, 2011, pp. 94-97 (in Russian)

[9]. STO 36554501-049-2016 “CPT Application for Soil Foundation Control on Permafrost”. Moscow: AO NIC “Stroitelstvo”, 2016, 53 p. (in Russian)

[10]. GOST 19912-2012 “Soils. Methods of cone penetration testing and dynamic soundings”. EASC: MNTKS, 2012, 33 p. (in Russian)