Proceedings of the XVII ECSMGE-2019
Geotechnical Engineering foundation of the future
ISBN 978-9935-9436-1-3

doi: 10.32075/17ECSMGE-2019-0217

XVII European Conference
ECSMGE 2019
Reykjavik
on Soil Mechanics
and Geotechnical Engineering

Yamal LNG: REX on a mega Oil&Gas Project built on the permafrost

Yamal LNG: retour d'expérience sur un méga projet pétrolier bâti sur le pergélisol

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ABSTRACT: The Yamal LNG project (Novatek, Total, and CNPC JV) consisted of the construction of a giant natural gas liquefaction plant in the Russian Arctic on the Yamal Peninsula. The project is located on a continuous permafrost zone 400m thick with an active layer of about 1.5m thawing in summer. The soil is a mixture of sand, silt, peat and clay. Building such an installation on this type of soil was a premiere. In particular, the design of the foundations of the four LNG tanks – tanks of 80 m diameter prestressed concrete with a capacity of 160,000 m3 - with very demanding settlement constraints was one of the challenges of this project. Two standards references concerning the foundations on the permafrost were at the disposal of the Project team: the Russian which does not consider settlements as critical and was therefore difficult to apply alone, and the Canadian which on the contrary gave so conservative results that the feasibility was uncertain. With the help of various on-site tests, careful engineering and meticulous construction; the reservoirs were built with excellent results in terms of settlement during the first years.

RÉSUMÉ: Le projet Yamal LNG (JV de Novatek, Total, et CNPC) consistait en la construction d'une usine de liquéfaction de gaz naturel géante dans l'Arctique Russe dans la péninsule de Yamal. Le projet est situé sur une zone de pergélisol (permafrost) continu de 400 m d'épaisseur, avec une couche active d'environ 1.5m qui dégèle l'été. Le sol est un mélange de sables, de limons, de tourbes et de matériaux argileux. Construire une telle installation sur ce type de sol était une première. En particulier, la conception des fondations des quatre réservoirs de LNG - réservoirs de 80 m de diamètre en béton précontraint d'une capacité de 160 000 m3 - avec des contraintes de tassement très exigeantes s'est révélée être un des défis de ce projet. Deux référentiels concernant les fondations dans le pergélisol étaient à disposition de l'équipe Projet: le Russe qui ne considère pas les tassements comme critiques et était donc difficilement applicable seul, et le Canadien qui au contraire donnait des résultats tellement conservatifs que la faisabilité était incertaine. A l'aide de différents essais sur place, d'une ingéniérie soignée et d'une construction méticuleuse; les réservoirs ont été construits avec d'excellents résultats en terme de tassement au cours des premières années.

Keywords: piles, creep; LNG; Oil&Gas; permafrost; thermostabilization; Yamal

1 INTRODUCTION

The Yamal LNG project (Novatek, Total, and CNPC JV) consists of the production, treatment, liquefaction, and shipping of natural gas and stabilized condensate from the South Tambey Gas and Gas Condensate Field located close to Sabetta along the Ob bay. The facilities to be developed include well pads for production drilling, wellheads, gas gathering lines, flow-lines, inlet facilities, gas treatment, liquefaction, LNG storage and loading facilities, utilities, accommodation, and support facilities including airport and seaport.

The project started in 2011 with front end engineering (FEED); first piling works started end of 2013 and first phase of the Project was in operation in December 2017. Among the many challenges of this project, the design of the foundations has proved to be difficult as the applicable Russian standards for foundation construction in permafrost were focussing only on the bearing capacity without considering the problem of the settlement in exhaustive manner while the settlement criteria are governing the design for this type of plant; particularly for the LNG tanks which were monolithic structures of 80 m diameter and 130 000 t in operation (filled with LNG product):

- The total settlement is limited by the flexibility of the connections to the tank.
- The planar tilting settlement for the tank must not exceed 150 mm across the diameter.
- Because of the bottom insulation, refrigerated storage tanks are relatively intolerant of bottom dishing type settlements. Center to edge dishing settlements for the LNG tank must not exceed 150 mm.
- Creep settlement of 1 single pile is limited to 25mm over the design life of the foundation (50 years).

2 SITE & SOIL CONDITIONS

The site terrain is typical arctic marshy tundra within low-level marine terraces 1 to 7 m above sea level. The Sabetta area has an arctic coastal maritime climate and is within the arctic climatic zone. Mean monthly air temperatures in the Sabetta area range from 6.2°C in July to – 26.2°C in February. The mean annual air temperature in the area is -10.5°C.

The soils in the area are permanently frozen (permafrost). During summer (June to October), topsoil undergoes thawing. The seasonally thawed layer (or active layer) standard depth is about 1.5m. The average soil temperature is constant around -5 °C from -5 m down to 100 m depth.

Soil investigations in the LNG tanks area were carried out during the summer 2012. 52 boreholes ranging from 80 m to 100 m were drilled with soil sampling every 0.75 m to 5 m depth, at every change of soil type with a maximum interval of 3 m from 5 m depth to 35 m depth and then every 5 m to the bottom. Core samples were collected to determine physical (soil composition, structure, water-ice relationships), thermal (thermal conductivity, heat capacity), mechanical (strength and deformation parameters), and chemical (salinity) properties of the foundation soils. Sampling, packing, and shipping of the samples were carried out in accordance with the Russian Federation standards (GOST 12071-2000). All tests were conducted in accordance with the Russian state standards and industry norms. Physical properties of the soils were determined in accordance with GOST 5180-84. Mechanical properties of the frozen soils were determined in accordance with GOST 12248-96. Classification of the soils was conducted in accordance with GOST 25100-95. The survey was completed by geophysical investigations (CMP seismic sounding to measure P and S waves velocites, GPR profiles, TDEM sounding and down hole geophysical sounding). 13 pile load tests were also carried out as part of the geotechnical survey on 127 mm and 153 mm piles of 10 m long. 12 tests in static compression and 1 in tension.

3 PILE CAPACITY

Piles are steel pipe piles filled with sand cement grout. As FEED preliminary asumption based on 2 m*2 m pile grid spacing (i.e. 1365 piles/tank), maximum factored vertical load at service was 1360 kN on one pile and 1474 kN during hydrotest.

The main parameters to calculate the pile axial capacity in permafrost soils are:

- Salinity (Dsal)
- Ice content due to ice inclusions or Iciness (Ii)
- Frozen bulk density
- Temperature

In table 1; typical soil characteristics under Tank 1 recorded during 2012 survey are summarized.

3.1 First calculation using Russian Standard

In accordance with SNiP 2.02.04-88 and SP 22.13330.2011, allowable pile capacities were calculated on the pile test results and the survey.

Calculated allowable pile capacity for Tank 1 area and 10m deep piles was 433 kN for 127 mm pile, 1580 kN for 426 mm pile and 1786 kN for 530 mm pile.

Table 1: typical soil model under Tank 1

Type of soil	Depth of Layer (m)	Bulk Density (g/cm3)	Visible ice content (%)	Salinity (ppt)	Temperature (◦C)
Silt, Ice Rich, Low Saline	0.5	--	--	--	
Organic Silt, Ice Rich, High to Very High Saline	2.9	1.49	21	10.49	-5
Fine Sand, Ice Poor, High Saline	6.4	1.93	6	18.32	-5
Silt, Ice Poor, Very High Saline	7.1	2.04	3	29.17	-5
Clay Silt, Ice Poor, Very High Saline	4.6	2.19	5	39.4	-5
Fine Sand, Ice Poor, Medium to High Saline	13.5	2.03	4	8.6	-5

Based on this result; 1365 piles of 426mm diameter at 10m depth were enough to carry one tank.

Russian Standard SNiP 2.02.04-88 focuses only on bearing capacity of the piles. Settlement issues are not covered and its calculation is not mandatory. This aspect had been identified by the project team from the beginning of FEED so that the pile capacity was assessed using Canadian practises by a Canadian specialized contractor. Particularly, the settlement criteria regarding the long term creep limited to 25mm over 50 years proved to be the governing one for pile design.

3.2 Calculation of pile capacity with consideration of pile settlement with Canadian practise

3.2.1 Theory

A frozen soil will deform over time when subject to the specific conditions of an applied loading, salinity, soil temperature, and composition. This phenomenon, known as creep, is typically divided into three intervals illustrated in Figure 1:

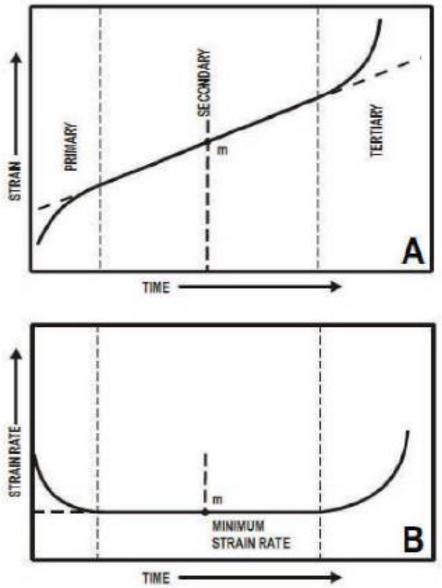

Figure 1: creep of the permafrost

1. Primary creep, where the creep rate continually decreases over time;
2. Secondary creep, a near-zero or steady state value; and
3. Tertiary creep, an increasing creep rate which eventually leads to the ultimate failure of the soil.

Design guidelines in North America first call for a definition of the soil as being ice-rich or ice-poor in nature. This determination is generally based on Morgenstern and Weaver (1981), who recommend classification of an ice-rich material if the frozen bulk density is less than 1.7 g/cm3. Ice-rich soils are usually governed by time-dependent deformation rather than ultimate strength, as pile behaviour is dominated by the presence of ice rather than the soil structure, with the bond stress between the frozen soil and the pile grout/slurry playing a significant role in the pile behaviour.

The stress-strain flow law presented by Morgenstern and Weaver (1981) was adapted to piles by Johnston and Ladanyi (1972) and Nixon and McRoberts (1976) and is generally expressed by the equation:

$$\frac{\mu a}{a} = \frac{3^{(n+1)/2} B \tau^{n}}{n-1} \quad (1)$$

where μa is the constant pile displacement rate; *a* is the pile radius; *n* is a secondary creep parameter (equal to 3 for ice-rich soils); B is a temperature dependent secondary creep parameter and τ is the adfreeze shaft stress.

Ice-poor soils on the other hand, are devoid of a continuous network of segregated ice which damps any long-term creep characteristics and adfreeze strength usually governs, although it is recommended that design be based on both settlement and adfreeze strength criteria (Morgenstern and Weaver, 1981). Morgenstern and Weaver provided the following expression for predicting the damped creep of piles in ice-poor soils:

$$\frac{Ua}{at^{b}} = \frac{3^{(c+1)/2} D \tau^{c}}{c-1} \quad (2)$$

where Ua is the displacement, *t* is the time elapsed after application of the load; *b* depends of the soil type and this factor is inferior to 1; *c* is a primary creep parameter (ranging from 1 to 5) and *D* is a temperature dependent primary creep constant. However, the primary creep constants provided by Morgenstern and Weaver for various soil types do not account for the presence of salinity, which has been demonstrated by numerous sources to have a large, detrimental impact on axial pile capacity. Cavanagh et al. (2012) presented secondary creep parameters for ice-poor soils based on analysis of an extensive set of unconfined compressive strength tests conducted on samples of varying salinities and composition. This research demonstrated that use of secondary creep parameters is applicable to pile design in ice-poor soils of any salinity with the use of grout or sand slurry, with the assumption that the failure surface will be at the soil/slurry interface.

3.2.2 Design assumptions

Ice-poor soils are assumed to have similar secondary creep parameters as those analyzed in a series of uniaxial compression tests by Cavanagh et al. (2012).

Ice-rich soils are assumed to have creep parameters similar to those presented in Morgenstern and Weaver (1981). A Factor of Safety (FS) of 1.5 was taken.

Those design asumptions were very conservative for 2 reasons:

- Parameters were not back calculated from pile tests and lab tests held by Russian Institutes as Russian method statements were different from Canadian ones so that they were no direct correlation between the tests and the parameters of equations (1) and (2)
- Considering Ice-poor soils creep governed by secondary creep as explained in the previous paragraph proved to be detrimental to the calculated capacity.

3.2.3 Results

Considering the initial design assumptions mentioned in section 3.2.2, it was calculated that 35 m depth 530 mm piles were necessary which has a considerable cost impact on the project and rose feasability concerns as rigs to drill down to 35 m in the permafrost were not available in Russia. Nevertheless, the call for tender for execution and the construction permit documents (Proyekt) were built with this hypothesis of 35 m depth piles.

3.3 Pile design at detailed engineering stage

The selected Contractor, the Entrepose - Vinci Construction Grands Projets consortium (ENVI) proposed, by means of a study carried out by Cathie Associates, France, during 2013, a pile design based on the asumption that Ice poor soils creep is governed by primary creep rather than secondary one (despite the high salinity) using an envelope of parameters among those parameters proposed by Biggar (1991) regarding to saline soils ice poor soils. The debate was still opened at the time of construction whether the saline ice poor soil will be governed by primary creep or by secondary. A careful work to link all the site survey samples to typical creep parameters has been carried out to limit any conservative approach.

But in another hand, it has been imposed at detailed engineering stage to consider a soil temperature of -2°C instead of -5°C to take into account the potential effect of global warning. Despite this unfavorable hypothesis, the hypothesis of primary creep behavior has allowed to considerably improve the pile design as ENVI proposed an optimized design of 948 piles to 20 m depth (i.e. 24 m long to take into account backfilling and ventilated space between soil surface and tank bottom) with an improved safety factor of 2.

It can be seen figure 2 that at -5°, the creep (black line) and the adfreeze strenght from pile tests based on Russian standards (blue and green lines) provide more or less the same pile capacity with a minimum embedment depth of 12m. At -2° (figure 3); the creep clearly governs and minimum pile embedment depth is 17 m.

With this design, elastic settlement at the end of the construction is estimated at an average of 30 mm and 120 mm during hydrotest of the tanks. Tilt is limited to 60 mm and center to edge differential settlement is 98 mm max so all the settlement constraints are met.

4 THERMOSTABILIZATION

Due to low soil heat transfer, the heat produced by the drying grout/slurry of the piles, particularly for a group of 948 piles, needs a considerable time to be evacuated. The usual technique, as per either Russian or Canadian construction practices in permafrost soil, is to use thermosiphons to speed up the process.

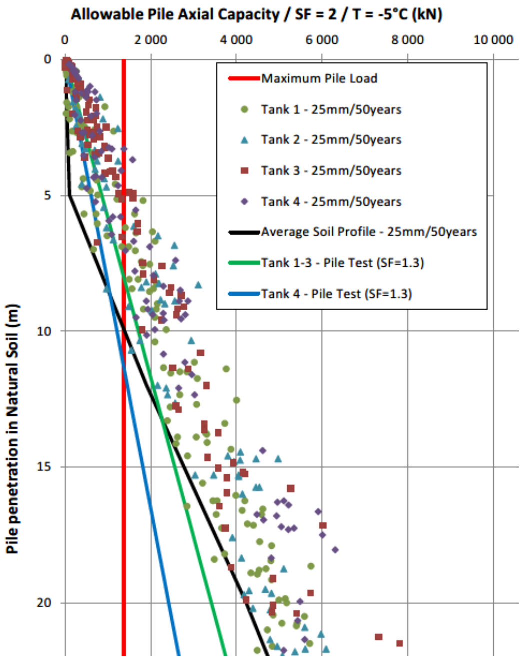

Figure 2: Allowable pile axial capacity based on creep settlement with soil temp=-5° - S.F=2 (Cathie Associates - ENVI, France, 2013)

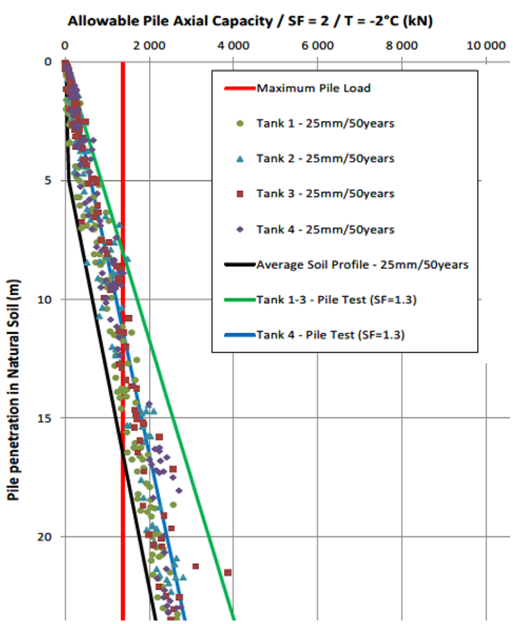

Figure 3: Allowable pile axial capacity based on creep settlement with soil temp=-2° - S.F=2 (Cathie Associates - ENVI, France, 2013)

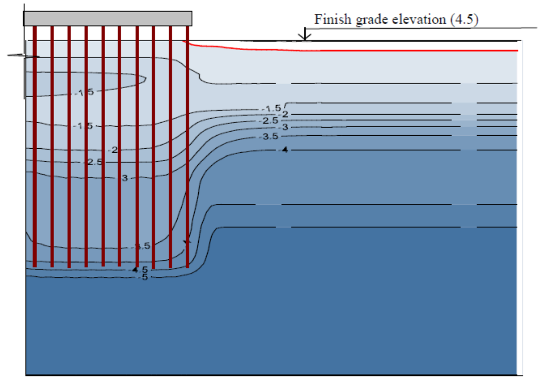

Figure 4: simulation of soil temperature under LNG Tank after 5 years without thermosiphon (Fundamentproyekt, 2012)

As can be seen in figure 4, without thermostabilization, the temperature of the soil remains well above the original -5°C after 5 years (-2° after the first winter). In fact it would never recover as numerical simulations showed an average temperature of -3°C under the tank after 50 years. At the contrary (figure 5), thermostabilization do not only allow the soil to come back quickly to its initial temperature, but to considerably decrease ground temperature to values below -10°C in less than 1 year.

For each LNG Tank, 1330 termosiphons of 20 m depth were installed. As seen on figure 6, in the frame of one winter between October 2014 and March 2015, the temperature along the piles dropped down to -14°C (average of -12.8° along the pile), well below the -5°C of the original soil and the -2°C for which the piles have been calculated.

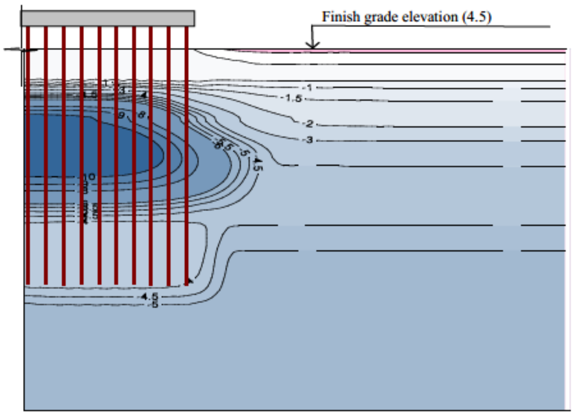

Figure 5: simulation of soil temperature under LNG tank after 1 year with thermosiphons (Fundamentproyekt, 2012)

5 RECORDED SETTLEMENTS

Settlements were monitored for each tank from the pouring of the slab (summer 2014 for tanks 1&2 and summer 2015 for tanks 3&4) to June 2017. Level was taken on 61 points per tank.

Observations are the following:

- During the hydrotest, the tanks have settled of 5 mm with following rebound of 3 mm during discharging
- During the period between hydrotest and June 2017 (1 to 2 years depending the tank), the settlements of the LNG Tanks stabilized, no significant creep has happened.
- Planar tilting and center to edge settlement have been limited to 15 mm

Those results are above expectation mostly due to the efficiency of the thermostabilisation that

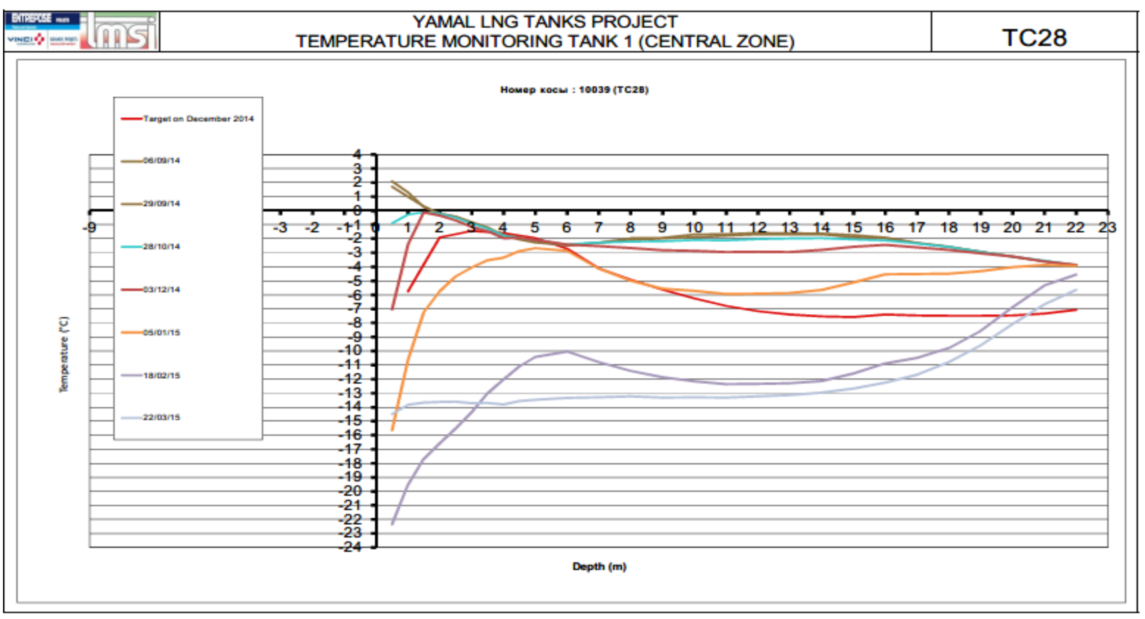

Figure 6: recorded soil temperature under LNG tank 1 between 09/2014 and 03/2015

was able to quickly drop the soil temperature far below pile calculation hypothesis. During the hydrotest, the 5 mm settlement are far below the calculated 120 mm. Planar tilting and center to edge settlement remains far less than expected so that the tank foundation has proved to be perfectly reliable. Long term creep during the service will be monitored but the first 18 months of service has shown no creep. It has also to be noted that the current loading/unloading rate of the tanks is of course more favorable vs. the calculation hypothesis considering 100% of Tank capacity.

6 CONCLUSIONS

The Yamal LNG project has proven that it was possible to build a large monolithic structure of 80 m diameter weighting 130 000 tons at service on mostly saline permafrost without significant settlement. This has been achieved with a considerably optimized design compared to the one based on Canadian practices which consider in very conservative approach secondary creep settlement for saline ice-poor soil. Use of thermosiphons has proved to be highly efficient and the design could certainly have been optimized further if thermal stabilization analysis could have been considered at early design stage.

7 ACKNOWLEDGEMENTS

We would like to thank Mme Fabiola Hernandez (Total); Mr Aurelien Accart (Total); Mr Loran Karwacki (Total) and Mr Nicolas Schneider-Maunoury (Vinci Construction Grands Projets) who have kindly reviewed and commented this article; and all the people who have been involved in Yamal LNG and have made this incredible project a reality with a particular attention to the workers who have built this plant in the harshest meteorological conditions that can be imagined.

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