Artificial ground freezing application in shield tunneling

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ABSTRACT

One of the methods to create a stable working environment around tunnel and seal the tunnel periphery against underground water is artificial ground freezing. The method is economically beneficial at large scale construction sites, and long-term containment. It is also superior method when temporary high strength soil or temporary impermeable barrier is needed. In this study, application of freezing in shield tunneling is investigate. A case study of a comprehensive underground tunneling project in Tokyo suburb is considered. The construction includes a junction of main tunnel with a ramp way that connect underground tunnel route to the ground surface. In this construction site, connection segment of main route and ramp way need to be widen. Because of this enlargement and sequence of works below water table, temporary stable work front and hydraulically water proof zone is demanded. As an effective way of ground improvement, freezing method is used in this case. In this way, a parametric study is conducted and amount of horizontal and vertical freeze-thaw induced displacement and effect of soil stiffness around tunnel is estimated numerically. In addition, the effect of design parameters of intermittent freezing on the thickness of freezing zone is investigated. Results of analyses indicates that based on reliable design scheme, an energy-efficient plan for application of freezing in shield tunneling is possible.

Keywords: artificial ground freezing, shield tunneling, ground heave and settlement, intermittent freezing

1 INTRODUCTION

The use of frozen soil for tunneling application during construction has become a major technology when there is problem with underground high water pressure and unstable soil condition. Artificial ground freezing (AGF) application for soil and rock first was used in shaft and mining industry. Later on, by advent of horizontal drill excavation, the method was employed in the drifts and in underground spaces (2). The method is economically favourable for the large scale sites, and long term confinement applications. After creation of frozen barrier, maintenance cost is low. During the expansion of underground tunnels or widening of connection segments between underground spaces, ground freezing aids to create a safe, impermeable and stable working front. In this way, usually entire circumference of the temporary excavation is frozen by installation of horizontal or vertical freezing pipes.

In this study, AGF application in an underground tunnel expansion project is investigated. Ground behavior around tunnel and near greenfield surface is anticipated due to freeze-thaw action. Effect of surrounding soil stiffness on freezing behavior is also considered. A construction site data related to the part of a comprehensive underground space enlargement project in Tokyo suburb is employed for this purpose. Due to the enlargement and sequence of works below water table in this site, temporary stable work front and hydraulically water proof zone is established using freezing method. Among the routine methods, AGF technique is employed. A parametric study is conducted using introduced case study data. In addition, by carrying out of a coupled thermal-mechanical analysis, effect of design parameters of intermittent freezing on the thickness of freezing zone is studied.

2 A CASE STUDY OF ARTIFICIAL GROUND FREEZING

An artificial ground freezing implementation at a construction site in Tokyo suburb is introduced briefly. The site concerns construction of approximate 16 km long highway tunnel that passes along Tokyo outskirt to reduce traffic overload inside of it. Along the tunnel route, access ramp ways are designed to connect it to the ground surface highways. At the location of connection between ramp way and main route, underground excavation is widened. Due to the enlargement and sequence of works below water table in this site, temporary stable working environment and hydraulically waterproof zone established using horizontal freezing pipes around enlarged area. The general studied cross section, water table, main tunnel and ramp way and final outer lining are indicated in Fig. 1. Focus of this study is on the top three mini-tunnels and corresponding freezing zone. Three top mini-tunnels called s10, k9, and k10, and location of horizontal freezing pipes inside of s10 tunnel are magnified in Fig. 2. By exploiting of mini-shields, mini-tunnels are excavated in a special order in few steps, then mini-tunnel lining are installed. Afterward, soil around outer lining is frozen to keep the overall construction stable; after freezing, mini-tunnels are connected to create final outer in situ ring. In the next step, soil inside of outer ring are removed gradually to the road level of main tunnel and ramp way (Fig. 1). Points A, B, C, and D depicted in Figs. 1 and 2 are employed to monitor freezing and thawing effect on ground deformation during construction steps. Excavation, freezing and thawing analyses in this site are carried out using a numerical code called SOIL PLUS. Geotechnical specifications of ground layers in this site are listed in Table 1. Frozen soil specification is taken to be the same as “His” soil layer in terms of internal friction angle and cohesion. For the elastic modulus of frozen soil, recommendation from other studies were used (4, 5).

3 INFLUENCE OF FREEZING SOIL THICKNESS AND SOIL STIFFNESS ON FREEZING-INDUCED DISPLACEMENTS

The amount of horizontal and vertical freeze-thaw induced displacement of soil must be anticipated before construction to reduce the damages against existing structures or utilities. In this study, settlement, heave and horizontal displacement are investigated in analyses. In addition, depend on the type of the soil, freeze-thaw cycles cause increase or decrease in undrained shear strength of soil. Change in material property during freeze-thaw happens at the same time when expansion or shrinkage in soil volume happens. In order to perform a parametric study, five analyze cases are examined as listed in Table 2. Parametric study includes effect of material property improvement during freezing, effect of freezing zone thickness and effect of soil stiffness around frozen zone on freezing-induced displacement. As the numerical code SOIL PLUS is unable to simulate gain or lose in strength of material during freezing and thawing, freezing process is carried out in two steps; temperature alteration and change in material stiffness, as shown in Table 2.

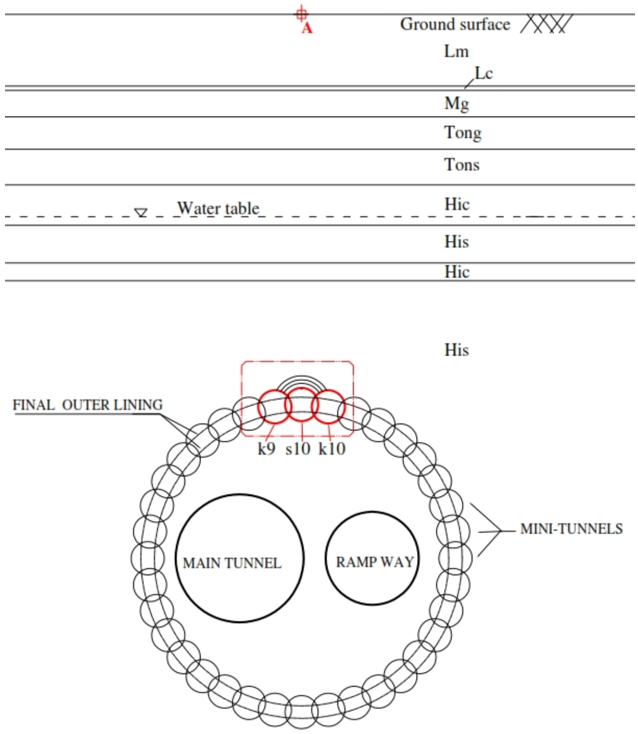

Fig1. Cross section of case study.

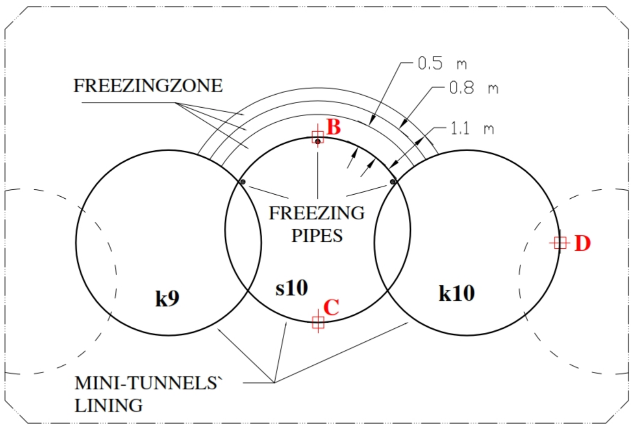

Fig2. Magnified cross section under study.

Table 1. Geotechnical specification of ground layers[1].

Soil ID	N_{spt}	t (m)	γ (kN/m^3)	ν	E (MN/m^2)	c (kN/m^2)	Φ (°)	K_0
Lm	3	8.86	14.0	0.45	3	120	15	0.65
Lc	5	0.54	14.0	0.45	3	120	15	0.65
Mg	50	3.27	19.0	0.3	25	25	39	0.35
Tong	48	4.01	21.0	0.3	290	25	42	0.35
Tons	44	4.39	20.0	0.3	140	25	41	0.35
Hic	47	5.01	19.0	0.45	98	580	10	0.35
His	49	4.63	20.0	0.3	160	25	42	0.35
Hic	47	2.19	19.0	0.45	98	580	10	0.35
His	49	83.9	20.0	0.3	160	25	42	0.35
Frozen soil (His)	---	---	---	0.3	600	25	42	---

[1]In Table 1, N_{spt} is SPT value, t is Layer thickness, γ is Unit weight, ν is Poisson ratio, E is elastic modulus, c is cohesion, Φ is friction angle, and K_0 is Lateral earth pressure coefficient.

Table 2. Analysed cases used in parametric study[1].

Cases	t (cm)	E (MN/m^2)	Freezing sequence[2]	n
Case 1	50	160	T-E	33
Case 2	50	160	E-T	33
Case 3	80	160	T-E	45
Case 4	110	160	T-E	57
Case 5	110	480	T-E	57

[1]In Table 2, t is freezing thickness, E is elastic modulus of "His" soil layer (Table 1), n is Number of construction steps.
[2]In freezing sequences, T-E indicates temperature alteration, then change in material strength, and E-T indicates change in material strength, then temperature alteration.

Based on Fig. 2, points of A (ground surface), B (crown of s10 tunnel), C (invert of s10 tunnel) and D (spring line of k10 tunnel) are chosen to manifest ground behavior during constructing steps. In this study, construction steps simulation is solely limited to the three mini-tunnel at the top, which is the small part of large construction sequences. First, s10 tunnel is excavated and its lining is installed, then, it is filled with air-mortar to prevent excessive displacement during construction, later on, k9 and k10 tunnel are excavated and their lining are installed concurrently. Next, using freezing pipes, surrounding soil outside of lining is frozen. After the frozen wall thickness reaches to the required value, air-mortar inside of s10 tunnel is removed and final outer lining is installed (shown in Fig. 1). According to Table 2, three freezing thickness of 50 cm, 80 cm, and 110 cm are analyzed. In case 5, except elastic modulus of "His" soil layer which introduces stiffer soil around freezing area, geotechnical parameters same as other cases are employed for all soil layers (Fig. 1, Table 2). Elastic-perfectly plastic Mohr-Coulomb model for soil and linear elastic for tunnel lining are assumed.

Fig. 3 demonstrates behavior of points A, B, C, and D for analysis cases of 1, 2, 3, and 4 during entire of construction steps. Ground surface undergoes a settlement during mini-tunnels excavation, then during freezing it heaves and then subsides slightly by thawing (Fig. 3-a). In the analysis cases of 1 and 2, frozen wall thickness increases to 50 cm and then decreases by thawing to 0 cm step by step.

As expected, due to the freezing expansion, s10 tunnel crown first experiences downward and then upward displacement during unfreezing. Because of small displacement, entire of stress path during freeze-thaw happens to be in elastic state (Fig. 3-b). It is notable that freezing induced heave in s10 tunnel crown is almost twice of ground surface (see Fig. 3-a, b).

As seen in Fig. 3-c for horizontal movement of k10 tunnel springline, the difference between cases of 1 and 2 is noticeable; by freezing application, because of arc shape of freezing area, in the case 2 where temperature changes is applied after soil stiffening, horizontal forces are notably higher than the case 1 resulting in bigger horizontal movement.

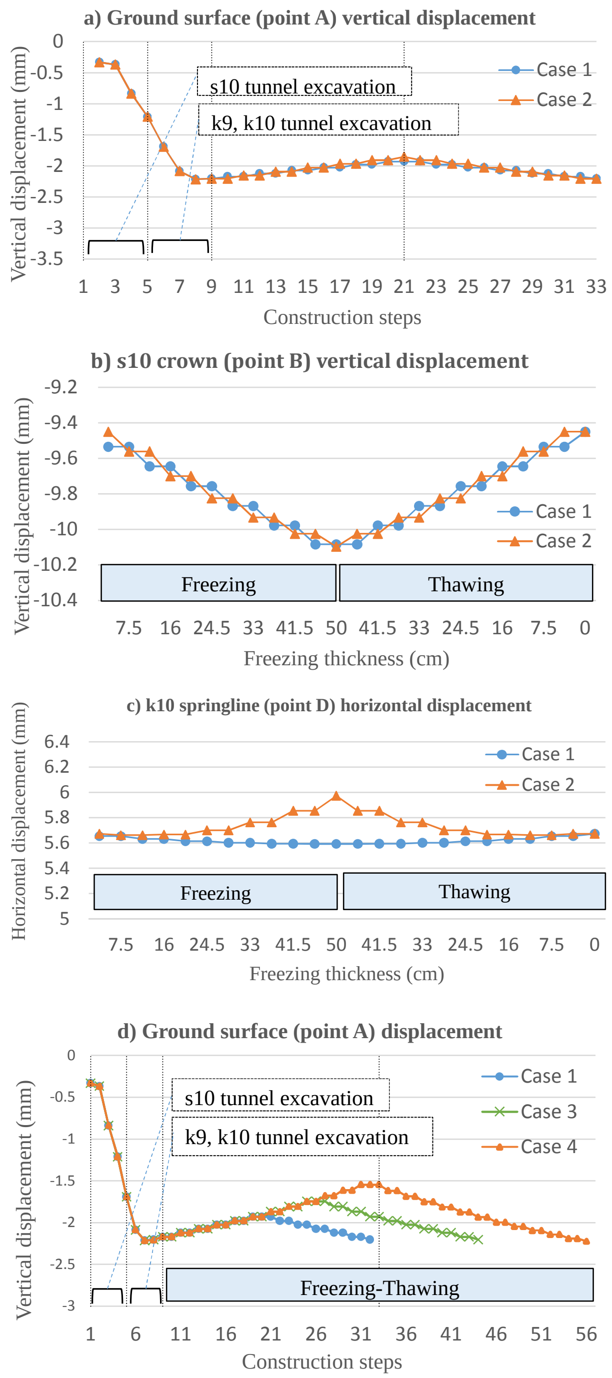

Fig. 3. Freeze-thaw induced displacement (cases of 1, 2, 3 and 4).

As during freezing, thermal conduction and material stiffening occurs simultaneously, actual field behavior is thought to be between cases of 1 and 2. Frost wall thickness effect during freeze-thaw action is shown in Fig. 3-d. After unfreezing, because of small displacement, in all three cases of 1, 3, and 4, ground subsides to its original level before start of frost action.

Tunnel crown and invert manifest similar behavior during entire of construction step except for s10 tunnel excavation as shown in Fig. 4-a. Based on Fig. 4-b, despite the soil with various stiffness around frost zone (case 4 and 5), the ground surface heave in both of cases are approximate identical to 0.7 mm.

According to Fig.4-c and Fig.4-d, due to the frost action, in case 5, crown and invert of s10 tunnel move downward 1.8 and 1.6 mm, while same points moves downwards 1.5 and 1.4 mm respectively for case 4. It clearly depicts that stiffer soil around freezing zone (case 5) limits freezing heave, increase frost induced pressure, and expands more toward tunnel center, pushing down the s10 lining.

4 INFLUENCE OF INTERMITTENT FREEZING TIME ON FREEZING ZONE THICKNESS

Frozen soil forms in the shape of cylinders around freeze pipes. As cylinders gradually grow larger, they join, forming a continuous wall. Once, the wall reached to specific required thickness, freezing power plant level is reduced just to keep the thickness fixed. In this part, the effect of design parameters of intermittent freezing during maintenance period on the thickness of freezing zone is investigated. In this analysis, initial ground temperature is 10 C°, ground surface temperature is 20 C° and freezing pipes temperature is initially taken to be -20 C°. Freezing thickness is decided by propagating of freezing front (0° isotherm) around freezing pipes by passing of time. Cross-section indicated in Fig. 1, geotechnical specification listed in Table 1, and thermal property of soil listed in Table 3 (1, 3) are selected for analyses in this part. Coupled thermal-mechanical analysis is performed for thermal distribution around freezing pipes. Based on analyses results, the time needs for freezing fronts (0° isotherm) to meet each other is approximately 14 days. During first days, the brine temperature is kept constant at -20 C° to build up frozen wall to the required thickness. In the following days, intermittent freezing is carried out to provide a constant thickness of the frozen wall. Fig. 5 shows employed temperature history in freezing pipes in one of analysis. In first 42 days (six weeks), freezing pipe temperature (T_{max}) is kept constant at -20 C°; the next day temperature is kept at -5 C°, and the next day it returns back to -20 C°, and then this process is continues for 21 days (three weeks). Fig. 5-b, shows freezing wall thickness history based on three different scheme. Three schemes are assumed the same temperature during first 42 days and varies during last 21 days of freezing as follow: a) changes of freezing temperature between -20 C° and -5 C°, b) changes of freezing temperature between -20 C° and -10 C°, c) changes of freezing temperature between -20 C° and -15 C°. Assuming 1 m thickness in frozen area is demanded according to Fig. 5-b, by employing of changes of freezing temperature between -20 C° and -10 C°, freezing wall thickness remains almost constant at required thickness.

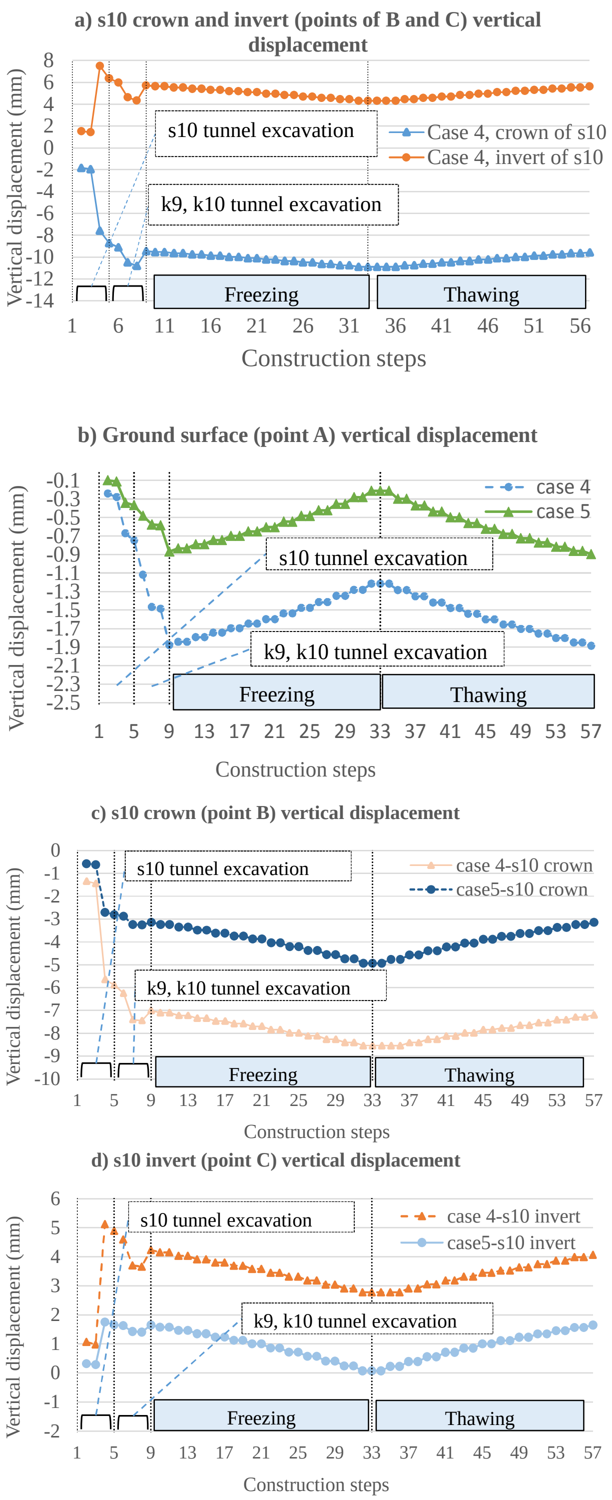

Fig. 4. Freeze-thaw induced displacement (cases of 4 and 5).

Table 3. Thermal property of soil[1]

Material	Specific heat c (kJ/kg.C°)	Conductivity k (kJ/m.h.C°)
Unfrozen Soil	c_i =1.0	k_i = 9.0
Frozen soil	c_f =0.8	k_f = 18.0

[1] In Table 3, c_i is initial heat capacity of soil before freezing, c_f is final heat capacity of soil after freezing, k_i is final thermal conductivity of soil before freezing, and k_f is final thermal conductivity of soil after freezing.

Changing of temperature from -10 C° to -15 C° causes the frozen wall thickness to increase which is not economical, but by variation of temperature from -10 C° to -5 C°, thawing of frozen wall is started. Actuate of proper freezing temperature during intermittent freezing should be based on accurate analyses otherwise may lead to the collapse of frozen wall.

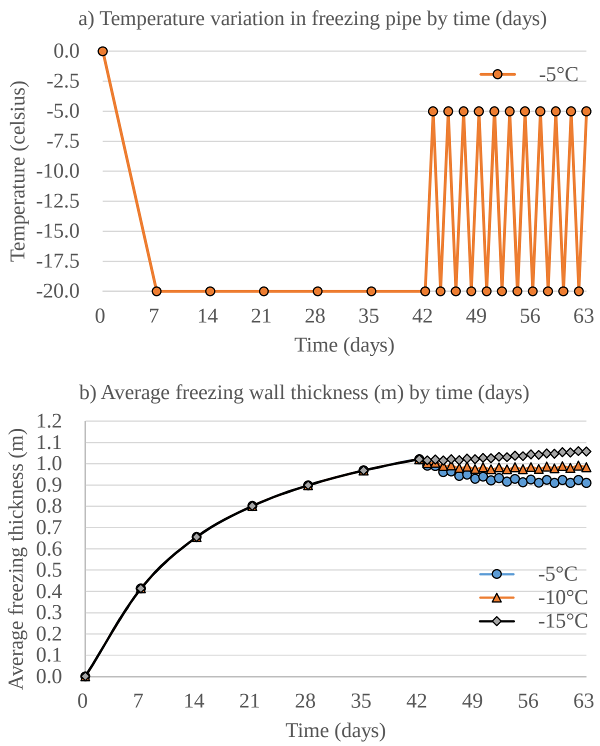

Fig. 5. Creation of frozen wall thickness for different temperature levels T_{max}.

5 CONCLUSIONS

This paper dealt with artificial ground freezing application in shield tunneling. An AGF applied case study is introduced and its field data are used for analyses. Amounts of vertical and horizontal soil displacements due to the freezing-thawing were investigated in analyses. Using of introduced case study data, a parametric study was carried out numerically and effect of frozen zone thickness and soil stiffness around tunnel on ground surface and tunnel periphery displacement were investigated.

Monitored heave and settlement results at four points around underground space and on ground surface revealed that order of displacements are small and reversible to their initial condition; furthermore, thicker freezing barrier generates higher heave elastically. The stiffer soil around freezing zone limits freezing expansion; increase frost induced pressure, and try to expand more toward less-stiffer regions. In addition, effect of design parameters of intermittent freezing during maintenance period on the thickness of freezing zone was considered. It revealed that based on 1 m required thickness for frozen area, in this case, changes of freezing temperature during maintenance period between -20 C° and -10 C° keeps freezing wall thickness almost constant at required thickness. Therefore, employing of proper freezing temperature plan during intermittent freezing should be based on accurate design to guarantee required frozen wall thickness.

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