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Design, construction, and field monitoring of cross passage tunnels constructed using artificial ground freezing in London

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ABSTRACT: The artificial ground freezing (AGF) technique was employed as temporary support for four of the seven sprayed concrete lining (SCL) cross passages (CPs) on the Silvertown Tunnel project in London. Three of these CPs were located beneath the River Thames, where surface access during construction was restricted. For the remaining CPs, alternative ground treatment methods, including groundwater depressurisation and deep soil mixing, were adopted. This paper provides an overview of the site's ground conditions, presents laboratory test results of frozen soil samples, and describes the geotechnical design parameters selected for the project. It also outlines the thermal design considerations and structural design aspects of the CPs. Construction-related aspects, such as ground temperature monitoring, thermal imaging, and surface settlement observations, are discussed. A comparison of ground movements around two onshore CPs, one constructed using ground freezing and the other with groundwater depressurisation, demonstrated that ground freezing was more effective in minimising ground movement, despite similar geological conditions. These findings demonstrate AGF's effectiveness as a temporary support method in complex urban tunnelling projects.

KEYWORDS: artificial ground freezing (AGF), sprayed concrete lining (SCL), groundwater depressurisation, tunnel, settlement.

1 INTRODUCTION

The Silvertown Tunnel project involved the construction of two 1.4 km-long, 10.66 m internal diameter road tunnels beneath the River Thames, linking the Greenwich Peninsula on the south bank with the Royal Docks at Silvertown on the north bank. These twin-bore tunnels were excavated using tunnel boring machine (TBM). This is the first road tunnel built under the Thames in London since 1980, and notably the first to incorporate cross passages.

Gall Zeidler Consultants were appointed by the Riverlinx Construction Joint Venture as the designer for the cross passages, with Züblin appointed as the specialist contractor responsible for implementing the AGF works.

Eight CPs were provided to ensure safe emergency egress for tunnel users, facilitate emergency intervention, and accommodate essential utilities for tunnel operations. Seven of the CPs were constructed using the SCL method. Artificial ground freezing was employed at four of these, while the remaining CPs used groundwater depressurisation or low-strength cement-soil mixing techniques where located onshore.

The AGF cross passages (CP3 to CP6) comprised a 250 mm-thick steel fibre-reinforced sprayed concrete primary lining and a 250 mm-thick cast in-situ concrete secondary lining, also reinforced with steel fibres. The permanent collars supporting the TBM tunnel openings were reinforced with steel bars. A compartmentalised PVC membrane waterproofing system was installed between the primary and secondary linings, with bespoke detailing at the interfaces with the TBM segmental lining.

This paper extends earlier work presented at WTC Stockholm (Morgan et al., 2025) by incorporating additional construction monitoring data, expanding the discussion on geotechnical parameter selection, and introducing a new comparative evaluation of ground movements between AGF-treated and depressurised cross passages.

2 BASIS FOR GROUND FREEZING

The design and implementation of the AGF system followed the requirements set out in the project-specific ground freezing specification, which was aligned with the British Tunnelling Society's Specification for Tunnelling, 3rd Edition.

Ground freezing is increasingly preferred in mixed or layered soils where traditional grouting techniques are unreliable due to soil heterogeneity (Casagrande, 1973; Soga & Ovalles 2011; Morgan et al. 2025). AGF was selected for CPs on the Silvertown Tunnel project where confined, water-bearing strata were interbedded with cohesive soils. Grouting was deemed unsuitable due to the risk of incomplete treatment caused by clay layers obstructing grout flow. Groundwater depressurisation beneath the river or built-up area would have required deep wells from the surface, posing significant logistical and environmental challenges. In contrast, AGF offered a more reliable and practical solution under such complex conditions.

By circulating chilled brine through a closed pipe network, AGF freezes the in-situ porewater, forming a frozen annulus around each CP. This frozen body significantly increases the strength and stiffness of the surrounding ground to ensure safe excavation while also serving as an effective groundwater cut-off until waterproofing and lining works are completed.

The AGF process comprised three phases: a freezing phase to establish the required frozen thickness and temperatures; a maintenance phase to sustain these conditions during CP excavation, verified by continuous monitoring; and a thawing phase after construction and secondary lining installation. These three phases "freezing, maintenance, and thawing" represent the standard methodology for AGF systems, consistent with the framework described by Andersland & Ladanyi (2004).

AGF thus addressed both groundwater control and ground stability requirements, making it the most suitable technique for the complex geological and logistical conditions encountered on the project.

3 GROUND CONDITIONS AND PROPERTIES OF FROZEN SOIL

3.1 *Geology and hydrogeology*

The site is located within the geological province of the London Basin, a synclinal fold in Cretaceous and younger sediments that dips gently eastward. Along the tunnel alignment, a typical London Basin sequence is present, comprising Made Ground, Alluvium, River Terrace Deposits, the Thames Group (London Clay Formation and Harwich Formation), Lambeth Group, Thanet Sand Formation, and Seaford Chalk, as shown in Figure 1.

At cross passages CP3 to CP6, the ground comprised units of the Lambeth Group, specifically the Laminated Beds (sand), Lower Shelly Beds (clay), and Lower Mottled Beds (sand).

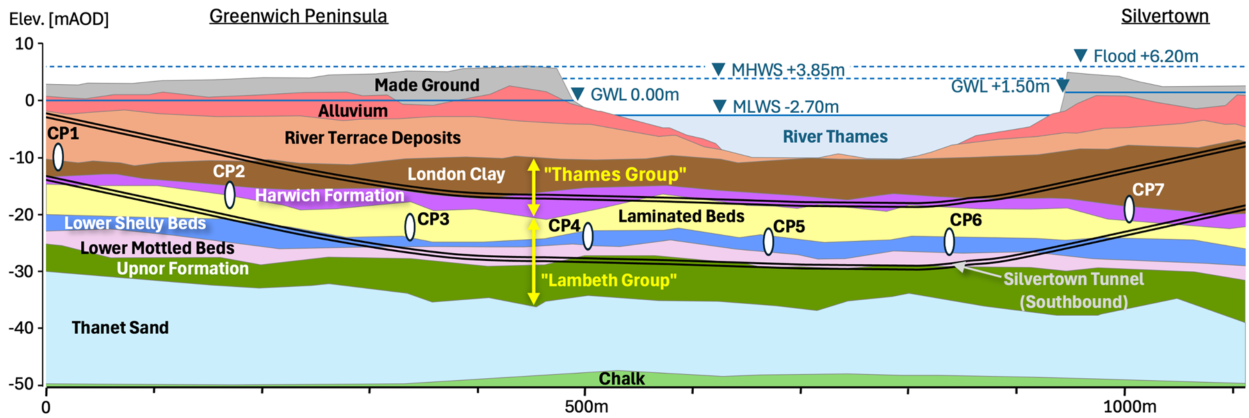

Figure 1. Geological long section along Silvertown Tunnel.

Three of the four AGF cross passages (CP4 to CP6) were constructed directly beneath the River Thames, while CP3 was located onshore on the Greenwich Peninsula. Approximate overburden depths above the CP crowns were 25 m (CP3), 20.5 m (CP4), 13 m (CP5), and 14 m (CP6).

The London Basin contains three principal aquifers: the Upper Aquifer within the Alluvium and River Terrace Deposits, the Intermediate Aquifer within the Thames and Lambeth Groups, and the Lower Aquifer within the Thanet Sand Formation and Chalk Group. Because low-permeability materials within the Intermediate Aquifer act as an aquitard, the granular soils below become pressurised, with piezometric heads exceeding 1.5 bar at the cross passage axis levels.

3.2 *Geotechnical design parameters*

Based on results from the in-situ and the laboratory tests undertaken, and published data, the geotechnical design parameters were derived. The parameters of the soils encountered at the CPs are presented in Table 1.

3.2.1 *Frozen and thawed soil properties*

The thermal and mechanical behaviour of frozen soil differs markedly from that of unfrozen soil due to the presence of ice. Some of the key characteristics of the frozen soils are listed below:

- the thermal conductivity of frozen soils is greater than that of unfrozen soils;
- the bonding of particles by ice is the major stabilising factor;
- the compressive strength and stiffness increase with a decrease in temperature;
- with a decrease in the loading rate its compressive strength reduces and its behaviour changes from brittle to ductile;
- frozen soils are prone to creep and relaxation effects; and
- when pressure is increased, ice melts at a temperature lower than its usual melting points.

Soil samples from the project site, comprising London Clay, Harwich Formation, Laminated Beds, Lower Shelly Beds, and Lower Mottled Beds, were collected and sent to a specialist laboratory in Bochum, Germany, for low-temperature testing. This facility was selected because it is one of the few in Europe equipped to carry out specialist low-temperature testing, including frozen-soil strength and creep behaviour, to the standards required for AGF design. Specimens were reconstituted by tamping in plastic moulds to achieve the target in-situ densities. Two groups of these prepared specimens were then frozen in cold chambers at temperatures of -10 °C and -20 °C for two days. After freezing, the specimens were extruded from the moulds and subjected to laboratory testing.

Laboratory testing established the thermal and structural design parameters of frozen soil. The thermal parameters were used in thermal analyses to predict the extent and progression of the freeze front, thereby defining the required freezing duration and the capacity of the freeze plant. The structural parameters were incorporated into the soil-structure interaction analyses undertaken for the design of the CPs. Selected results from the thermal tests on the soils, as well as from the strength tests on the frozen and thawed specimens, are summarised in Tables 2 and 3, respectively. Granular soils, when frozen, were found to exhibit greater strength characteristics compared to cohesive soils at the same temperatures.

3.2.2 *Time-dependent stiffness of frozen soils*

The time- and temperature-dependent behaviour of frozen soils has been extensively documented in the literature (e.g. Andersland & Ladanyi, 2004; Nixon, 1991). The visco-elastic behaviour of ice is influenced by several factors, including time, temperature, pressure, strain rate, crystal orientation, and density. To characterise this behaviour for design purposes, particularly the variation in elastic modulus of frozen soils over time and temperature, a series of uniaxial creep tests (UCTs) was carried out for each soil type encountered on the project.

These tests were performed at four levels of constant axial stress, ranging from 30% to 60% of the frozen unconfined compressive strength (UCS), over a period of 90 hours at two target temperatures: −10 °C and −20 °C.

The power-law formulation proposed by Klein (1978) provides a widely adopted framework for characterising creep behaviour in frozen soils and was used here to interpret the project-specific UCT results.

The time-dependent stiffness of the frozen soil, $E_{f(t)}$, was approximated from the UCT results using the power law equation proposed by Klein (1978):

$$E_{f(t)} = \frac{1}{E_0 + A \cdot \sigma_1^{B-1} \cdot t^C} \qquad (1)$$

where, A, B and C = creep test parameters; E_0 = initial Young's modulus at $t = 0$; t = time; and σ_1 = constant applied axial stress.

This equation captures the creep behaviour of frozen soils, in which stiffness decreases over time due to time-dependent deformation under sustained loading.

The time-dependent Young's modulus given by the Equation (1) can be converted into the elastic modulus of the frozen ground, E_s, at time t using the following isotropic elasticity relationship:

$$E_s = E_{f(t)} \frac{1 - \nu}{(1 + \nu)(1 - 2\nu)} \qquad (2)$$

where, ν = Poisson's ratio (for frozen soil assumed to be 0.33).

The stiffness values determined from the creep tests at two reference durations, $E_{f,0}$ (2 weeks) and E_f (3 months), represent the modulus at the start of excavation and at deactivation of AGF system, respectively.

According to Klein's creep law, the rate of creep remains constant and failure is unlikely provided that the sustained compressive stress does not exceed 45% of the UCS. Based on this criterion, and considering project-specific conditions, it was assumed that the stress acting around the CPs would not exceed 750 kPa. Consequently, the time-dependent Young's modulus of the frozen ground, E_s, for each soil type was calculated using Klein's equation with a constant axial stress of σ_1 = 750 kPa, as presented in Figure 2.

Table 1. Geotechnical design parameters of the soils encountered at the cross passages.

Strata	Unit weight, γ [kN/m³]	Friction angle, φ' [Deg.]	Cohesion, c_u, c' [kPa]	Stiffness E_u, E' [MPa]	Permeability k [m/s]
London Clay / Harwich Formation	20	23	125, 0	50, 40	1 x 10^{-9}
Laminated Beds – Sand unit	21	35	- , 0	- , 150	5 x 10^{-5}
Lower Shelly Beds – Clay unit	21	24	150, 0	150, 120	1 x 10^{-8}
Lower Mottled Beds – Clay unit	21	28	250, 0	150, 120	5 x 10^{-8}
Lower Mottled Beds – Sand unit	21	33	- , 0	- , 150	5 x 10^{-5}

Note: 1) c_u is the undrained shear strength and increases with depth (the parameters presented are the average value of each stratum).
2) Poisson's ratio ν_u = 0.49 for total stress (undrained) behaviour in cohesive soils.
3) Poisson's ratio ν' = 0.2 for effective stress (drained) behaviour in all soil types.
4) The coefficient of earth pressure at rest, K_0, is taken as 1.2 for all strata listed in the table.

Table 2. Laboratory-derived soil thermal parameters.

Strata	Plasticity index	Thermal conductivity [W/mK] "Unfrozen"	"Frozen"	Frost pressure at −15 °C [kPa]
London Clay	49	1.95	3.47	55
Harwich Formation – Clay unit	40	1.87	3.44	39
Laminated Beds – Sand unit	6	2.21	4.05	138
Lower Shelly Beds – Clay unit	28	2.08	3.69	288
Lower Mottled Beds – Clay unit	21	1.93	3.36	131

Table 3. Laboratory-derived soil shear strength parameters.

Strata	Unfrozen strength, c_u [kPa] and φ'	Frozen UCS [MPa] (strain at failure) −10 °C	−20 °C	Frozen strength c_f [kPa] and φ'_f −10 °C	−20 °C	Thawed strength (from −10 °C) c_t [kPa] and φ'_t
London Clay	110, 23°	2.5 (11%)	5.2 (11%)	1200, 3.0°	2100, 3.0°	30, 18.7°
Harwich Formation – Clay unit	140, 23°	2.4 (15%)	5.0 (14%)	1100, 2.5°	2100, 2.5°	30, 17.8°
Laminated Beds – Sand unit	-, 35°	4.1 (20%)	7.0 (19%)	1460, 3.5°	2400, 3.5°	27, 24.8°
Lower Shelly Beds – Clay unit	150, 24°	2.6 (17%)	5.0 (14%)	1100, 3.0°	2050, 2.0°	4, 26.1°
Lower Mottled Beds – Clay unit	250, 28°	3.0 (18%)	4.8 (10%)	1000, 4.0°	1800, 4.0°	25, 18.1°

Note: 1) The drained cohesion, c', for the "unfrozen" soils is zero.
2) Poisson's ratio for frozen soils is assumed to be 0.33.
3) The shear strength parameters presented for the unfrozen, frozen, and thawed soils were determined from laboratory triaxial compression tests.

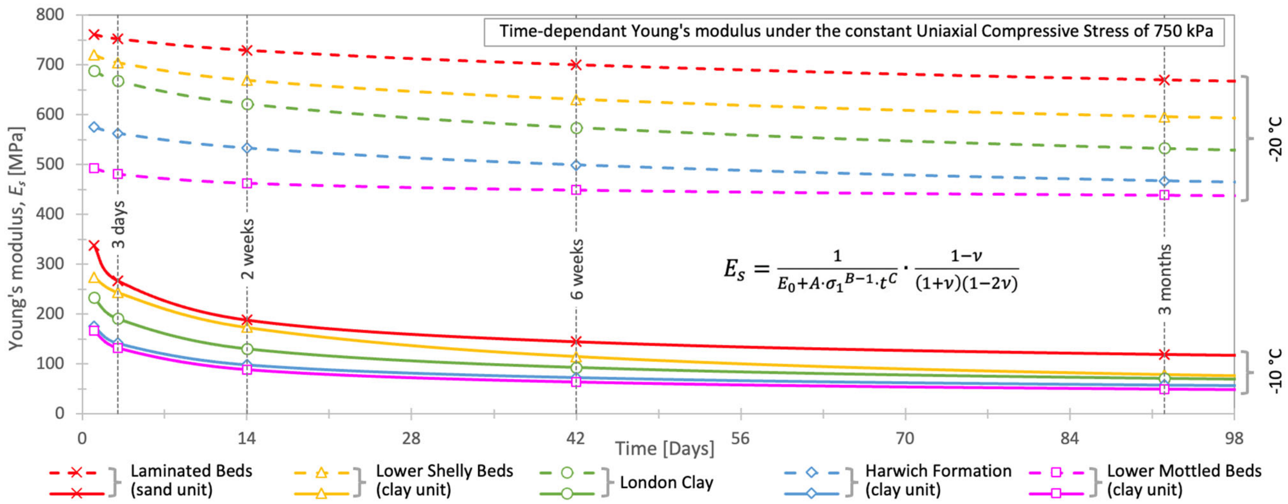

Figure 2. Time- and temperature-dependent Young's modulus of frozen soils under the constant stress of 750 kPa.

The results indicate that frozen soils with lower plasticity indices and lower freezing temperatures (–20 °C) exhibit higher stiffness and slower creep rates. In particular, soils frozen at −10 °C experienced a more rapid reduction in stiffness, with the Young's modulus decreasing to approximately 30% of its initial value after three months under constant loading.

For design purposes, it was conservatively assumed that the duration of AGF would not exceed three months. This assumption was used in selecting time-dependent stiffness parameters to ensure that the temporary support system remained effective throughout the excavation and support installation phases.

4 DESIGN OF CROSS PASSAGES CONSIDERING AGF

As shown in Figure 3, the AGF system comprised 26 freeze pipes installed from the southbound tunnel, arranged in a 'picture frame' configuration around each cross passage collar. CDM Smith served as the ground freezing consultant supporting Gall Zeidler by providing thermal design calculations and freeze pipe layout optimisation, which were reviewed and incorporated into the overall cross passage design. These calculations confirmed the key requirements: a 1.5 m thick frozen soil ring maintained at an average temperature of –10 °C (between the –2 °C isotherms), created using a single 94 kW freeze unit circulating brine chilled to –35 °C.

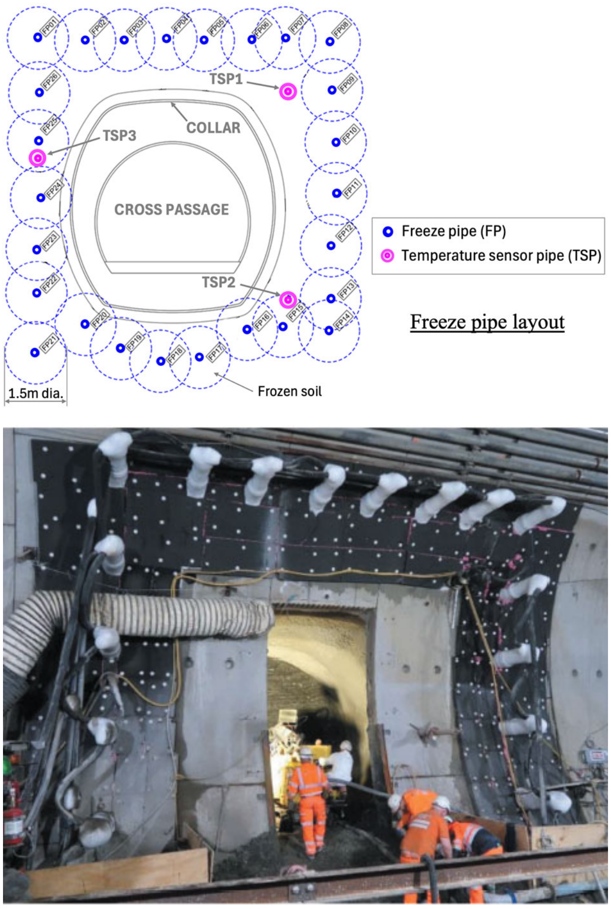

Figure 3. Freeze pipe layout (CP4 as built) and photograph during excavation from southbound tunnel, showing black insulation tiles and ice formed on the freeze pipes.

Thermal modelling indicated that approximately 70 days of initial freezing were necessary to achieve the required frozen thickness and to ensure a proper seal onto the extrados of the TBM tunnels before any breakout and excavation could commence.

The temporary and permanent linings were assessed by soil-structure interaction analyses using 2D finite element modelling, which incorporated frozen ground parameters within the 1.5 m ring and later updated to thawed properties after the secondary lining had sufficiently cured.

Due to uncertainties in sprayed concrete strength development against frozen ground, the first 50 mm of sprayed concrete was treated as sacrificial. Furthermore, the design strength of the C30/35 concrete was conservatively reduced by 50% to 15 MPa, effectively resulting in a primary lining thickness of 200 mm at this adjusted strength for structural design purposes.

5 CONSTRUCTION

Specialist contractor Züblin was commissioned to carry out the ground freezing works. The designer and ground freezing contractor collaborated closely throughout the entire process. The AGF CPs were constructed sequentially from CP3 to CP6. Freeze pipe drilling and installation began in May 2023, with the first freeze plant at CP3 activated in July 2023 and the final plant deactivated in late December 2023.

5.1 *Ground temperature monitoring*

Ground temperatures were critically monitored to assess the progress of ground freezing, maintain the freeze, and evaluate the performance of the freeze system. At each cross passage, three temperature sensor pipes (TSPs) were installed (see Figure 3), strategically positioned between the freeze pipes and at the outer perimeter of the required 1.5 m frozen ring. Each TSP was equipped with thermocouples spaced at 1 m intervals along its length.

As expected, sensors within the TSPs located close to TBM tunnels recorded higher ground temperatures than those near the midpoint between the tunnels. Despite the insulation tiles installed on the segmental lining (see Figure 3), the sensors immediately adjacent to the TBM tunnels indicated temperatures approximately 5 to 7 °C higher than those at the centre of the CPs.

At the target (northbound) tunnel, 14 temperature sensors were initially installed within the segmental lining to monitor ground freezing at the CP–TBM interface and to confirm closure of the frozen ring at the extrados of the target segmental lining.

5.2 *Criteria for commencing excavation*

A comprehensive assessment was undertaken at each cross passage to establish target temperature criteria for commencing excavation. This included analysing temperature monitoring data to confirm formation of a 1.5 m thick frozen body with an average temperature of –10 °C between the –2 °C isotherms, based on the as-built positions of freeze pipes and sensors. Given the low porewater salinity (< 2.7%), no significant freezing point depression was expected. Groundwater pressure data were also reviewed to ensure pore pressures within the frozen ring did not exceed the design limit of 1.8 bar. Excess pressures were relieved through drainage pipes installed to protect the TBM lining and to maintain the stability of the frozen ground.

Following confirmation of target temperatures and pressure conditions, excavation of the AGF CPs commenced. Excavation typically progressed in 1.0 m advance lengths, with the four AGF CPs ranging between 9 m and 12 m in total length. Excavation and primary lining installation generally took up to 12 days, as summarised in Table 4. Figure 3 shows the construction of an AGF CP from within the southbound tunnel, where the ends of the freeze pipes extend approximately 1 m beyond the tunnel lining and insulation panels remain fixed throughout the freezing period to maintain ground temperatures.

Table 4. Number of freeze and excavation days for each CP.

	CP3	CP4	CP5	CP6
No. of freezing days before excavation	52	55	60	69
Total no. of freeze days before plant deactivation	122	120	122	126
No. of excavation days	10	9	12	12

5.3 *Thermal imaging of CP face*

Thermal imaging was carried out by the designer's site representatives to monitor ground temperatures at the tunnel face during excavation. Prior to each excavation round, thermal images were taken to visually capture the temperature distribution, providing a clear indication of the extent of frozen ground. These images were reviewed to identify any warm spots that might indicate insufficiently frozen zones or potential seepage. Figure 4 presents an example of thermal images alongside a photograph of the tunnel face at CP3, clearly illustrating the frozen annulus and the relatively warmer centre of the face.

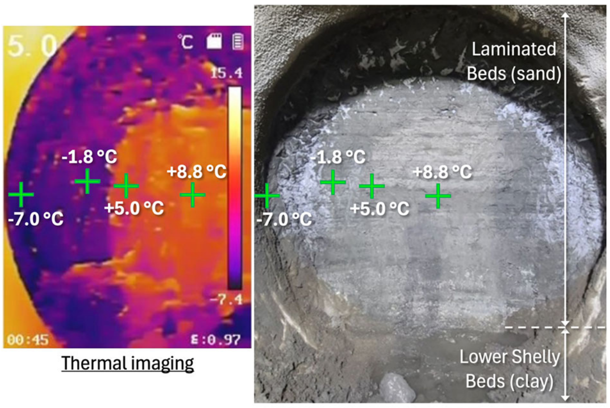

Figure 4. Thermal imaging and photograph of the frozen face (CP3).

5.4 *Sprayed concrete performance at low temperatures*

To reduce the risk of sprayed concrete failing to adhere to the frozen ground, warm water (30–35 °C) with an accelerator dosage of 6.5–7.0% was used in the C30/35 concrete mix. This proved effective, and no adhesion issues were observed when this procedure was implemented.

To evaluate the in-situ strength of the primary lining, three core samples were taken at CP6 thirteen days after spraying. Their compressive strengths at 14 days ranged from 20.0 to 23.8 MPa, highlighting the significant impact of low-temperature curing on strength development, as comparable non-AGF SCL cores had achieved the design strength of 30 MPa within the same period.

6 FIELD MONITORING

6.1 *Ground temperature evolution*

As presented in Figure 5, ground temperature evolution at CP3 was monitored using TSPs, excluding collar zones, to assess the effectiveness of AGF. Prior to freezing, the ground temperature ranged between +17 °C and +19 °C. During the first two weeks of AGF, temperatures in the mid-section of CP3 dropped rapidly, reaching 0 °C. Probes near the southbound tunnel recorded comparatively higher temperatures due to thermal influence from the tunnel, while centrally located sensors reached significantly lower values. Minimum recorded temperatures at CP3 eventually fell to –23 °C.

Excavation commenced after 53 days of freezing at CP3. During excavation, ground temperatures rose, by up to 8 °C overall and up to 4 °C in the mid-section, primarily due to heat generated by tunnelling equipment and concrete hydration. Following excavation, temperatures gradually decreased again during subsequent construction activities, including waterproofing and installation of the secondary lining. Significant temperature increases were confined to excavation and early sprayed concrete phases, reflecting transient thermal effects. A further temperature rise observed after completion of excavation and primary lining installation may be attributed to heat generated by curing of sprayed concrete and the ingress of warm air into the cross passage from TBM tunnels. A slight temperature increase was recorded during the concrete pour for the secondary lining, likely mitigated by the insulating effect of the primary lining.

6.2 *Ground movements during construction*

Surface ground movement data, also presented in Figure 5, were analysed alongside the temperature records. CP3 is located in onshore ground conditions consisting of 15 m of superficial deposits, underlain by 8 m of the Thames Group and 10 m of the Lambeth Group. Excavation took place within the Laminated Beds (sand) and Lower Shelly Beds (clay), beneath approximately 25 m of overburden. Situated just 150 m from the River Thames, the site experiences tidal influence (±3 m), which affects groundwater levels and may contribute to surface ground movement.

During the first eight-week AGF period, ground temperatures dropped from +17 °C to –20 °C, and a slight upward trend in ground heave was recorded. During the 10-day excavation phase, ground movements remained negligible. Settlement only became apparent after AGF was deactivated, continued gradually over two months. This was primarily attributed to thaw consolidation, as the melting of ice crystals within the frozen soil led to volume reduction. The associated reductions in stiffness further influenced the ground settlement. By the end of the monitoring period, three months after AGF termination, total settlement at CP3 was limited to 3 to 6 mm.

In contrast, CP2, located approximately 165 m southwest of CP3, was constructed under geologically and hydraulically comparable conditions but using groundwater depressurisation instead of AGF. As shown in Figure 6, surface settlement above CP2 continued to develop even after the target drawdown level (–20 m AOD) was achieved, due to the ongoing increase in effective stress until pumping ceased. Consequently, the maximum recorded settlement was 26 mm. This comparison demonstrates that AGF was significantly more effective in limiting ground movement, highlighting its suitability as a ground treatment method, particularly in water-bearing soils and areas close to sensitive infrastructure.

7 CONCLUSIONS

Artificial ground freezing was successfully employed as a temporary support system for four sprayed concrete lining cross passages on the Silvertown Tunnel project. It provided both structural stability and an effective groundwater cut-off, enabling safe excavation and primary lining installation in complex ground conditions, including beneath the River Thames.

Laboratory testing and creep modelling were undertaken to establish time- and temperature-dependent geotechnical

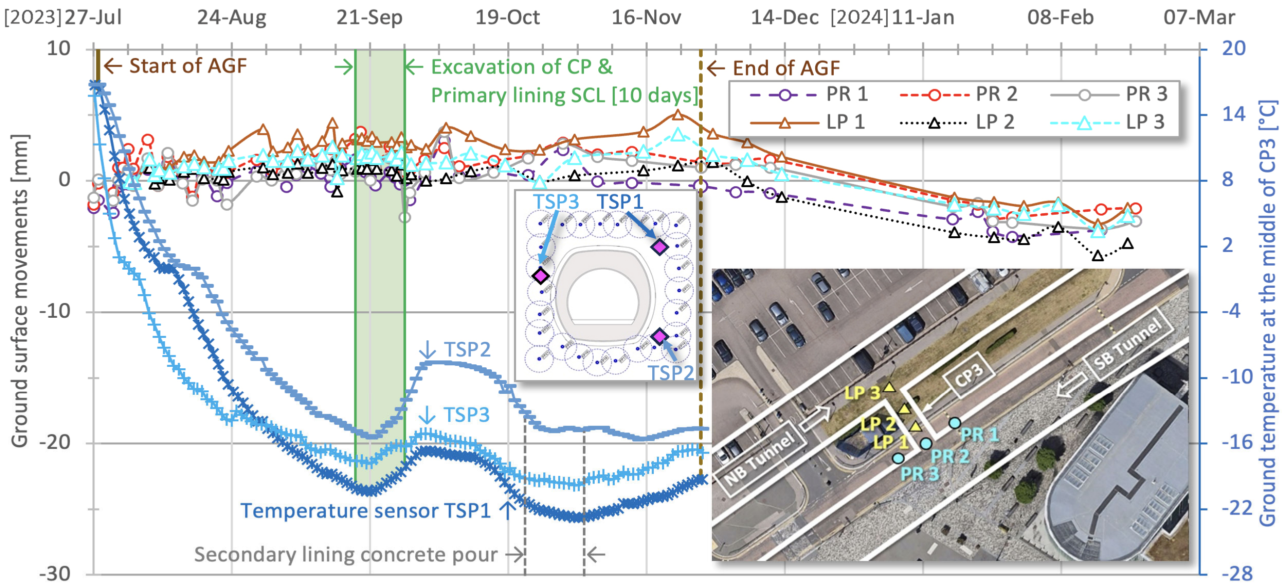

Figure 5. Ground temperature development and ground movements during construction of CP3 using AGF.

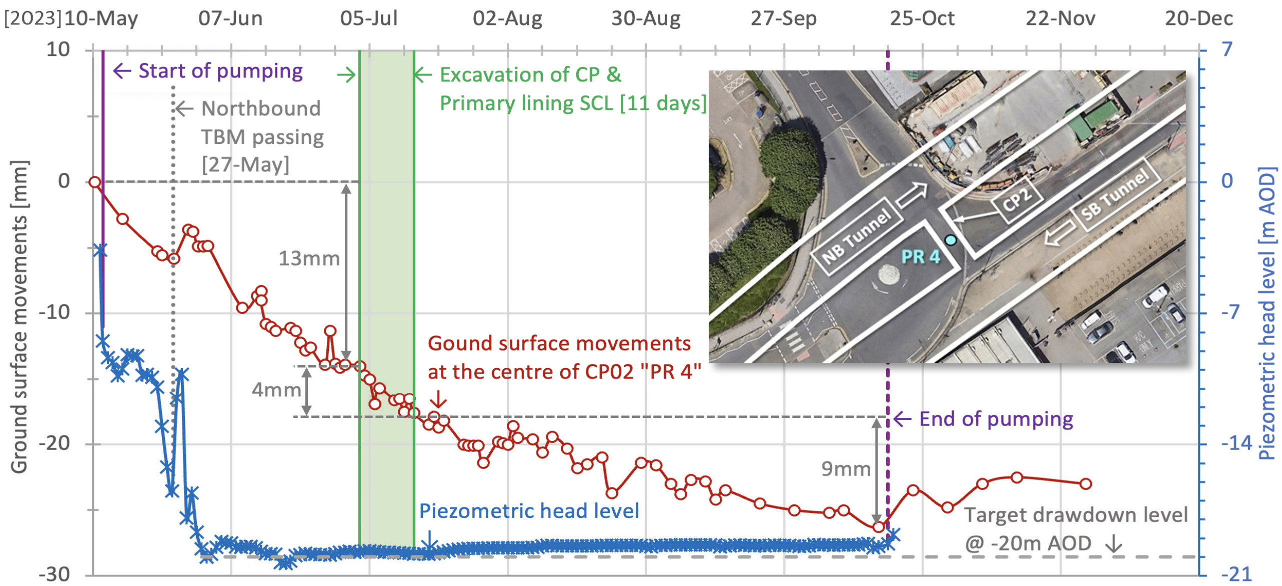

Figure 6. Piezometric and ground surface responses during construction of CP2 using groundwater depressurisation.

parameters for the frozen soils. These parameters informed the design of the CPs and ensured safety throughout excavation. Notably, no design modifications were required during construction, demonstrating the reliability of the original design approach.

Field monitoring showed that AGF resulted in significantly lower surface settlement compared to groundwater depressurisation under comparable geological conditions.

These findings confirm AGF as a reliable and effective ground treatment technique, particularly suited to water-bearing strata and locations where tight control of ground movement is critical.

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