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Prediction and performance of a ground freezing application to the rehabilitation works of an existing tunnel

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ABSTRACT: The Cassia-Monte Mario railway tunnel, which runs through a hill in the northern part of Rome (Italy), has required invert demolition and reconstruction. The tunnel crosses heterogeneous sandy and silty soils below the water table. In order to prevent the ingress of water, the ground freezing technique was applied. The paper describes the design predictions based on simplified thermal analyses, laboratory tests and real-scale trial fields which were performed to integrate and validate the design predictions. Field tests were supported by automatic monitoring system implemented to measure the temperatures in the ground and deformation effects induced by freezing on the lining. The paper focuses on the role played by field tests which gave a decisive contribution to the correct and effective planning of the execution phase. This allowed to reach a better insight into the application of the ground freezing for underground works.

1 INTRODUCTION

In the last years ground freezing application to tunnelling is largely spreading in Italy in response to problems due to groundwater conditions: frozen soil provides a strong watertight barrier allowing the excavation to safely proceed (e.g. Colombo *et al*. 2009).

Recent developments in ground freezing techniques have indeed resulted in more efficient and cost-effective freezing systems that can compete with other ground improvement techniques. However, the design of a freeze system is still challenging as it involves the prediction of freezing process and soil response: real-scale trial fields are required to integrate and validate the design predictions and to accurately plan the construction stages.

In this paper the ground freezing application to rehabilitation works of an existing tunnel is described and the role of trial fields in the design and execution stage is highlighted.

2 THE CASSIA-MONTE MARIO TUNNEL

The Cassia-Monte Mario tunnel belongs to the double track railway line forming the Northern Railway Belt of the city of Rome (Italy). The 8.8 m internal diameter tunnel, which has a length of 4.4 km, crosses the Monte Mario hill, made up of cohesive and granular deposits of Plio-Pleistocene age below groundwater table. The maximum overburden is 75 m. The excavation of the tunnel began in 1944 and was carried out quite irregularly, with many interruptions and breaks, until 1984. The long construction history was indeed characterized by many problems due to difficult soil and groundwater conditions: various instability phenomena occurred in the southern section (km 21.100 ÷ km 21.700), which also produced significant effects on the surface. To overcome this situation, a 150 m long pilot tunnel was built to carry out ground improvement works prior to excavation. Extensive cement and silicate injections were also performed from ground surface.

In recent times (2004) the tunnel required rehabilitation works aimed at strengthening the original structure (particularly the invert), controlling the water leakage and enlarging the tunnel clearance profile. During the execution of the invert reconstruction, a serious disruption took place: a sudden ingress of water and, then, water and soil occurred from the invert excavation. The water inflow (up to 100 l/s) could not be stopped easily and the tunnel was flooded. The amount of mud which accumulated into the tunnel was about 5000 m^3. Despite the significant overburden (60 m) (Fig. 1), the event had repercussions on the surface with settlements up to several hundreds of millimetres.

After this event the suspension of works was imposed and a detailed geotechnical study was carried out to identify the most appropriate design solution to continue invert reconstruction.

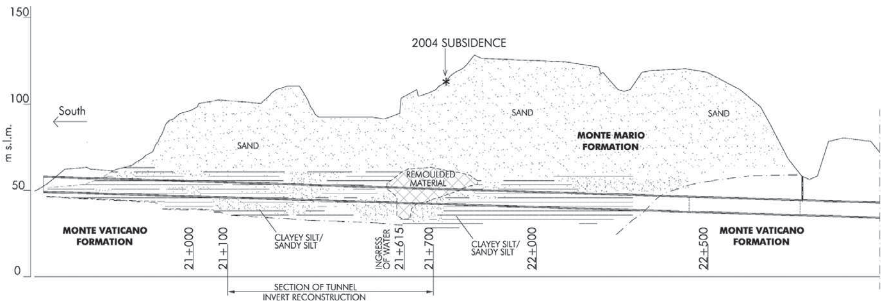

Figure 1. The schematic geotechnical profile along the Cassia-Monte Mario tunnel where the Monte Mario Formation is crossed.

3 THE GEOTECHNICAL CONTEXT

The geological profile along the tunnel route consists of a marine deposit of stiff overconsolidated clay of Pliocene age (Monte Vaticano Formation), overlain by a Pleistocene deposit of clayey silt and sandy silt (Monte Mario Formation), with sandy interbeddings, becoming medium dense sand in the upper part.

Along the stretch which has been characterized by instability phenomena during the construction, the tunnel crosses the Monte Mario Formation (Fig. 1).

Soil properties have been investigated by means of numerous geotechnical surveys which were carried out both from the ground surface and from within the tunnel.

The soil at the tunnel level is characterized by a silt content between 40÷50%, while the sand fraction can vary between 30÷48% and the clay between 13÷22%. The Liquid and Plastic limits are respectively equal to 26% and 18%. The natural water content at the tunnel depth has an average value of 20%. Strength parameters are typical of fine dense granular soils: c' = 40÷50 kPa, ω' = 34° for a range of confinement pressure between 200 and 600 kPa, with the high value of cohesion mostly related to the mutual interlocking of the grains.

Geotechnical investigation also revealed that the ground around the tunnel where the inflow of water occurred is loosened and remoulded with high water content.

The average horizontal permeability determined by Lefranc tests is about 5×10^{-5} m/s. Vertical permeability measured by means of laboratory test varies between 1.2×10^{-8} and 9×10^{-9} m/s. The high anisotropy of permeability has to be correlated to the frequent sandy interbeddings.

Piezometers installed in boreholes drilled from within the tunnel and from the ground surface showed piezometric levels increasing in the stretch between km 21.100 and km 21.700 and reaching the maximum value of 20 m above the tunnel invert. The piezometric measurements often show significant variations at short distances but no particular variations over time have been identified.

It is reasonable to assume that the extensive ground improvement works, carried out during the tunnel excavation to counteract the difficult soil conditions, modified the original soil structure and properties, particularly creating unpredictable groundwater flow-paths.

In some cases, during geotechnical investigations, small cavities, probably related to previous boreholes, have been identified.

As a consequence, the soil permeability and groundwater flow-paths represented the main uncertainty the ground freezing design had to face with.

4 THE DESIGN OF THE GROUND FREEZING APPLICATION

4.1 *Design criteria*

To resume works in safety, it was necessary to ensure the water-tightness and improved strength of the soil around the tunnel invert. At the initial design stage, jet grouting and artificial ground freezing seemed to be the suitable methods to meet these requirements.

Considering the heterogeneity of the subsoil and the necessity of realizing a continuous water cut-off, the effectiveness of jet grouting seemed to be low and ground freezing was selected as the most appropriate solution.

Artificial ground freezing is normally realized through the circulation of a refrigerant fluid into freeze pipes, installed in the soil, to extract heat from the ground until the temperature falls below the groundwater freezing point. In this case, the indirect method (brine method) was adopted as freezing technique (Harris 1995; Andersland & Ladanyi 2004).

This method requires the use of an industrial refrigeration plant, from which the refrigerant liquid (a water solution of calcium chloride $CaCl_2$, brine, whose operational temperature is generally in the range from −18°C to −30°C) is delivered to freeze pipes drilled in the ground. A freeze pipe consists of two concentric tubes: the refrigerant fluid enters into the open-ended inner tube and flows through the annulus between the two tubes, drawing heat from the ground. The outer tube is connected to the return lines so that the brine is sent back to the plant, where it is cooled and delivered again to the freeze pipes.

Adopting the ground freezing, the tunnel invert reconstruction has required the preliminary freezing of an arch of soil around the invert and side walls by an array of freeze pipes drilled from the tunnel and, then, the removal of the pipes to proceed with the excavation and cast of the new invert.

The frozen soil arch around the tunnel was aimed at temporarily preventing the ingress of water and improving strength and deformation properties of the soil. As the excavation took place when pipes were dismantled, the freezing system was designed to ensure the maintenance of the above requirements until the completion of the new invert.

4.2 *Design predictions*

The prediction of the real ground behaviour during and after the freezing process was an hard task to tackle, due to the uncertainties about the changes which the soil around the tunnel has suffered in the long history of instabilities and ground improvements. As a consequence, real-scale trial fields were considered a necessary step in the design stage.

The design analyses first focused on the evaluation of geometrical features and mechanical properties of the frozen soil required to ensure water-tightness and stability at the bottom of the excavation.

These design requirements were evaluated by means of 2D numerical analyses based on the finite element method. The analyses were carried out reproducing the initial tunnel conditions by the back-analysis of lining stress measurements and then simulating the preliminary stage of the ground improvement and invert excavation. The results indicated that a 3 m thick arch of frozen soil was necessary below the invert and around the lower part of the side walls (Fig. 2). Lining displacements induced by excavation are about 20 mm (Fig. 2).

Parametric analyses were performed progressively decreasing strength and stiffness of the frozen soil; the results showed that an average compressive

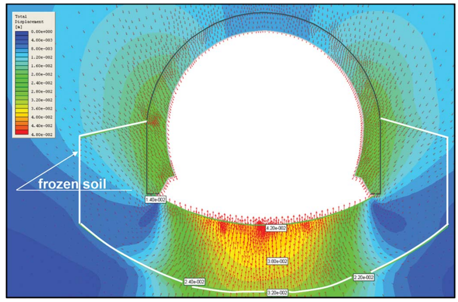

Figure 2. Displacements field induced by invert excavation with the protective arch of frozen soil.

strength of about 1.5 MPa was required to allow the construction safely proceeds.

Laboratory tests under temperature and strain rate controlled conditions were performed to measure strength and stiffness properties of the Monte Mario silt in the frozen condition. Uniaxial compressive tests were carried out on samples frozen at the temperatures of −5°C, −10°C and −15°C, imposing a strain rate of 1%/min (Sayles *et al.*, 1987) The results showed that the average short-term compressive strength of the frozen soil ranges from 3.5 MPa (−5°C) to 7 MPa (−15°C) and the initial modulus from 120 MPa (−5°C) to 600 MPa (−15°C). Experimental tests performed on similar soils (e.g. Lunardi *et al.* 2001) indicated that long term uniaxial compressive strength can decrease almost to 40% of the short term value, considering a free standing time of the frozen soil of about 7 days.

Hence, on the basis of the numerical analyses and laboratory tests results, the mechanical properties shown by the soil at the temperature T = −5°C were set as the minimum design requirements for the arch of frozen soil. As a consequence, the temperature T = −5°C was identified as the maximum temperature that could be accepted during the thawing process, after the removal of the freeze pipes, until the completion of the new invert.

To predict the rate of freezing, thermal computations were carried out by means of a simplified model (Sanger & Sayles 1979). According to this model, the freezing process develops in three stages: at first, the frozen soil columns grow from separate freeze pipes; then, the separate frozen soil columns merge to form a continuous wall and, finally, two separate rows of frozen columns merge to form a thicker wall of frozen soil.

The model has been applied to predict the time required for the formation of the frozen columns (stage 1) and the time required for the formation of the frozen wall (stage 2). In the first stage the temperature field around a freeze pipe, in the frozen region, was determined according the Fourier heat flow equation, assuming a steady-state condition:

$$\frac{d^2T}{dr^2} + \frac{1}{r}\frac{dT}{dr} = 0 \qquad (1)$$

where T = temperature at the distance r. The time required to freeze the column to a radius R was thus obtained from the following:

$$t_I = \frac{R^2 L_I}{4K_1 T_s} \cdot \left[2\ln\left(\frac{R}{r_o}\right) - 1 + \frac{C_1 T_s}{L_I} \right] \qquad (2)$$

where R = radius to the interface frozen-unfrozen soil, r_0 = radius of the freeze pipe, T_s = difference between the temperature at the freeze pipe and the freezing point of water, K_1 = thermal conductivity of the frozen soil, C_1 = frozen soil specific heat, L_I = a function of the latent heat of fusion of the soil water and unfrozen soil specific heat.

Due to the soil heterogeneity, the prediction of the freezing time was carried out considering different scenarios characterized by variations in soil particle size: clay, silt, and sand. In Table 1 the main physical and thermal properties used in the analyses are indicated.

Assuming a temperature of the brine of −28°C the time for the closure of the frozen columns (with a spacing of 1 m) was in the range between 20 days (sand) and 30 days (clay).

The role played by the brine temperature was also investigated: Figure 3 shows the growth of the frozen column in a silty soil, assuming different freeze pipe temperatures.

The freezing time predictions by the simplified model are indeed conservative, nevertheless they gave useful information to plan the freeze pipes lay out and construction stages to be validated by the field tests (see § 5).

Table 1. Main physical and thermal parameters.

Parameter		Sand	Silt	Clay
Dry specific weight	kN/m^3	15.90	15.89	15.87
Water content	%	25.20	25.50	25.80
Saturation degree	–	1.0	1.0	1.0
Frozen soil specific heat	kJ/m^3 K	2.19	2.195	2.20
Unfrozen soil specific heat	kJ/m^3 K	3.061	3.067	3.072
Frozen soil conductivity	kJ/hmK	12.50	10.60	8.70
Unfrozen soil conductivity	kJ/hmK	7.10	6.00	4.90

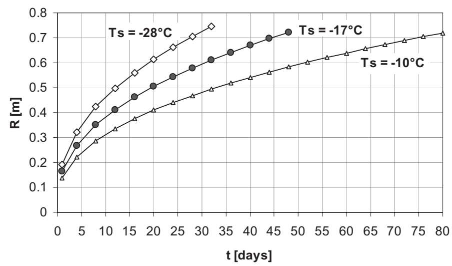

Figure 3. Variation of the radius of the frozen column (isotherm T = 0°C) over time for different values of the freeze pipe temperature, considering the properties of a silty soil.

Considering that the excavation works required the dismantling of the freeze pipes, i.e. these works took place during the thawing process, the estimation of the rate of thawing was the crucial aspect of the design. It had indeed a strong influence on the definition of the construction stages and temperature at which the excavation could start (target temperature, T_o).

Experimental tests on the real site appeared to be the most reliable way for accurately evaluating those aspects and validate the preliminary assumption of a target temperature T_o = −10°C.

Finally, one of the major concern in the design stage was the evaluation of the expansion of the ground due to freezing and the possible effects on tunnel lining. Because of the uncertainties about soil properties and structural behaviour of the tunnel, it was decided to address this issue in the trial field, implementing an accurate system to monitor lining displacements.

5 TRIAL FIELD

5.1 *Geometrical layout*

The trial field was realized inside the tunnel for a length of 48 m. Freeze pipes were installed with a spacing of 1 × 1 m, in a radial arrangement to freeze a 3 m thick soil body below the tunnel invert and around the side walls up to a height of about 2.5 m (Fig. 4). The pipes spacing varies from 1.20 m at the top to 1.35 m at the bottom.

The length of the pipes is in the range from 4.0 m to 4.8 m. The upper section of the pipes, drilled through the existing invert, was insulated to delimitate the target frozen zone of the tube. The total volume of frozen soil was 2640 m^3.

The field test length was subdivided into 4 sections with a length of 12 m each. Each section was equipped with 210 freeze pipes, for the total number of 840 pipes in the trial field.

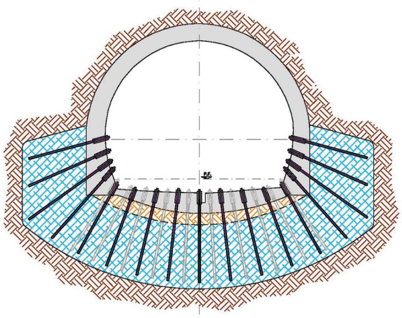

Figure 4. Freeze pipes lay out in the tunnel cross-section.

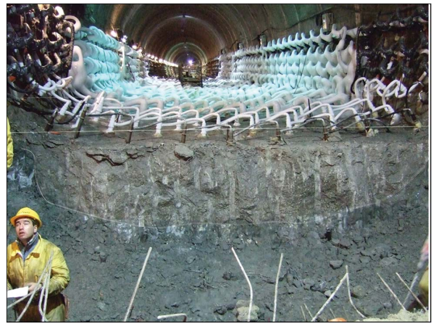

Figure 5. The excavation of the existing invert.

All pipe installations were realized using a preventer system to rapidly close the hole in case of uncontrolled flows of water and/or soil.

The freezing ground application was preceded by injections of water-cement mixtures to reduce the soil permeability and to contrast possible preferential pathways of water especially in the area affected by the water inflow.

5.2 *The refrigeration system*

The refrigeration plant was located inside the tunnel and connected through pipes to the cooling towers realized at the south portal. A freeze plant with a capacity of 500 kW was used. The freeze plant was designed for simultaneously freezing a tunnel length of 48 m. The designed brine supply temperature was −28°C. The delivery system was planned and realized in order to separately feed each row of freeze pipes and to disconnect the pipes installed at the invert, while feeding the pipes at the side walls.

5.3 *The monitoring system*

The monitoring system was developed to be simple and robust in order to work in the complex conditions of a tunnel under construction.

The temperature monitoring system was composed of temperature sensors and data loggers for the automatic data acquisition and recording, allowing real time data processing to control both the freezing and thawing process.

Temperature measurement tubes were installed at well defined distance from the freeze pipes for a total number of 19 for each section: 11 in the invert and 8 at the side walls. Each tube was equipped with 3 sensors, placed at a depth of 0.5 m (upper sensor), 1.5 m (intermediate sensor), 2.5 m (lower sensor) from the top of the tube. The total number of sensors is 57 for each section and 228 in the trial field. The temperature measurement tubes at the side walls were maintained active also during the demolition and reconstruction works to control the thawing process. Temperature sensors were also installed along the delivery system to check the brine temperature.

Monitoring of the tunnel lining behaviour was carried out to control possible movements induced by freezing and thawing as well by excavation. Monitoring program included 3D geodetic optical levelling and invar tape to measure tunnel lining diametrical distortion (convergence). Moreover invar tape was used to monitor side walls movements, verifying possible compressions or extensions.

Ground surface and subsurface movements were also controlled respectively by optical levelling and borehole inclinometers and extensometers.

5.4 *Construction stages*

The sequence of construction stages was the following:

1. freezing of the four 12 m long sections,
2. removal of the freeze pipes only in section n°1 when the target temperature $T_o = -10$°C was measured below the invert and at the side walls,
3. demolition and reconstruction of the invert in section n°1 working for successive stages with a length of 4 m,
4. removal of the freeze pipes in section n°2 when the target temperature $T_o = -10$°C was measured in the frozen soil body,
5. demolition and reconstruction of the invert in section n°2 working for stages with a length of 4 m, and this cycle was repeated for section n°3 and n°4.

During the excavation works the actual shape and structural features of the existing invert were discovered: the invert is characterized by a marked and abrupt reduction of the cross-section size at the side walls connection. In some cases the lack of any structural continuity between the invert and side walls was found out.

5.5 *The monitoring results*

During the trial field 840 freeze pipes were simultaneously active and 228 thermometric sensors were installed in the ground recording data every 8 hours. Monitoring data were an invaluable aid to analyze the freezing process and to define the critical design parameters.

The freezing process is well described by the temperature data plotted versus time. Figure 6 shows the typical measured trend recorded by the sensors below the invert: an initial rapid decrease of temperature is followed by a slowdown trend approaching the freezing point of groundwater. Then, the temperature starts decreasing again but slower than in the previous phase.

At the beginning of the process the freezing rate is in the range 1.3°C/day ÷ 1.7°C/day, decreasing to 0.6 ÷ 0.8°C/day after the freezing point.

During the experimental test, the temperature of the brine flowing into the freeze pipes did not reach the operative value $T = -28$°C, also because of the unsatisfactory insulation of the feeding system. Even if the average brine temperature was $T = -17$°C, the thermal condition $T = 0$°C was attained in about 25 days, while the target temperature $T_o = -10$°C was reached in about 45 days.

Temperature measurements in the ground below the invert were generally homogeneous, while the temperature data in the ground at the side walls were more scattered, showing a coefficient of variation always greater than 0.3 (Fig. 7).

The spatial distribution of the temperature was characterized by a decreasing gradient along the axis of the temperature tubes, due to the radial arrangement of the freeze pipes that caused an increase of the sensor-freeze pipe distance from the

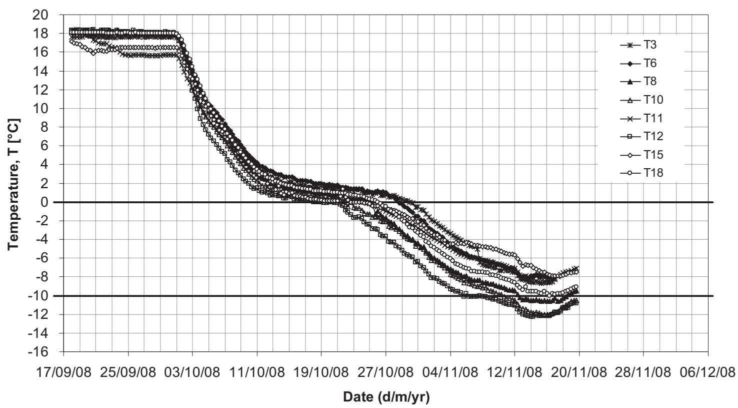

Figure 6. Variation of temperature with time during ground freezing. Data from intermediate sensors below the invert in section n°1.

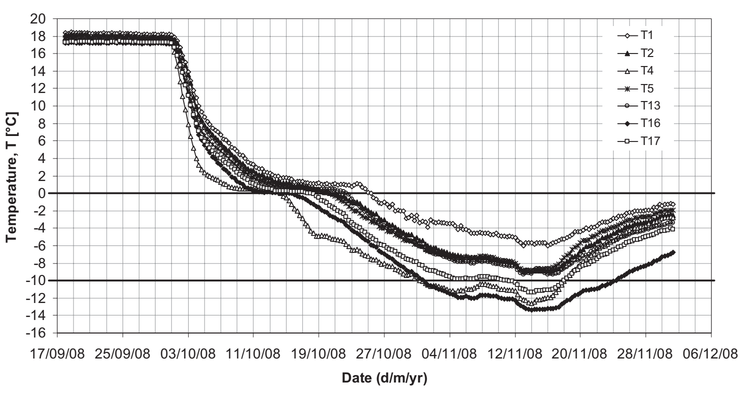

Figure 7. Variation of temperature with time during ground freezing. Data from intermediate sensors at the side walls in section n°1.

top to the bottom. As a consequence, the upper thermometers, that were located closer to the freeze pipes, recorded an apparent faster cooling than the lower sensors. The decreasing radial gradient was in the range between 1.6°C/m and 2.8°C/m.

A different response has been noticed between the sensors installed below soil invert and those installed at the side walls. As a rule, comparing sensors with the same distance from the freezing pipes, the temperatures measured at a given time below the invert were higher than temperatures measured at side walls. The maximum measured difference was 2.7°C.

The thawing process has been monitored for more than 2 weeks for each section. A non linear variation of temperature with time was measured and a faster heat of the ground was observed in the upper sensors (Fig. 8).

Considering a period of time of 7 days the rate of thawing varies in the range from 0.36°C/day to 0.58°C/day with an average value of 0.4°C/day for the sensors below the invert. For the sensors at the side walls the rate varies from 0.38°C/day to 0.81°C/day with an average value of 0.6°C/day.

The freezing process produced slight effects on the tunnel: in sections n°2, 3 and 4 an average convergence of 4 mm was measured (Fig. 9). A different behaviour was observed in section n°1, where

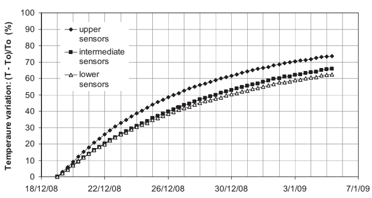

Figure 8. Temperature variations measured in the ground at the side walls during the thawing process. The variation is expressed as a percentage of the temperature to at the beginning of the thawing process.

outward 6 mm displacements of the side walls were recorded.

Movements associated to the excavation stage were more significant; the data were also analysed considering the effect produced by the excavation of the adjacent section (Fig. 9).

5.6 *Field trial results*

The field experimental evidences validated the designed freezing system and helped in defining the time schedule of works and planning the construction site activities.

On the basis of the measured freezing rate, even considering the real brine temperature, the soil response appeared more similar to a sand behaviour, according to the simplified model used in the design stage.

Moreover, two operative temperatures were defined to guide the construction stage. Trial field demonstrated that the demolition and invert reconstruction of a 12 m long section could be completed in a week. Considering that the maximum acceptable average temperature of the thawing phase was calculated to be $T = -5$°C, and the maximum measured rate of thawing was 0.6°C/day, the target temperature at which excavation could start was set $T_o = -10$°C. Thus the design preliminary assumption was confirmed.

However, the dependence of the temperature measurements on the sensor-freeze pipes distance and the different response between invert sensors and side walls sensors highlighted the necessity of a careful analysis of monitoring data. Thus, a procedure for the target temperature evaluation was developed.

This procedure defined the target temperature as the average value $T_o = -10$°C measured in the frozen soil, considering separately data from the side walls sensors and invert sensors; moreover various controls of the quality and representativeness of the data were required (e.g. the coefficient of variation). Additional requirements (e.g. the

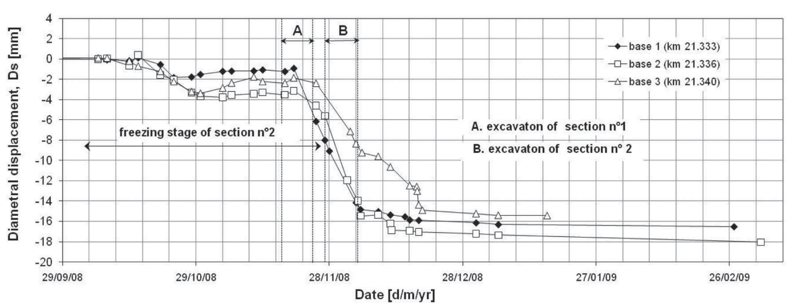

Figure 9. Lining displacements measured during the freezing and excavation stages in tunnel section n°2. (Negative values indicate convergence).

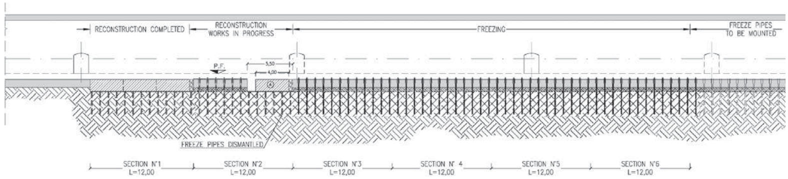

Figure 10. The sequence of tunnel invert reconstruction.

average temperature for each temperature tube T ≤ −7.5°C) were also implemented to ensure about local fails in the formation of the frozen soil.

6 CONSTRUCTION PHASE

The invert reconstruction was carried out for a total tunnel length of 325 m, according to the cyclic sequence that was tested in the trial field (Fig. 10):

1. freezing of four sections in advance,
2. removal of the freeze pipes in the first section when the target temperature in the frozen soil was reached according to the procedure above described,
3. freezing of the fifth section, in order to provide a length of 48 m of freezing soil in advance,
4. demolition and reconstruction of the invert of the first section, advancing for stages with a length of 4 m each.

All the execution stages have been carried out with the constant control of thermal conditions and lining deformations.

The procedure for the target temperature evaluation has proved to be a useful tool for a rapid data analysis to control the thermal requirements for excavation.

Monitoring of the temperatures demonstrated that the thermal condition $T = 0$°C was reached between 16 and 20 days.

Monitoring of the movements inside the tunnel during the freezing stage confirmed that in most cases the tunnel underwent a convergence of some millimetres (2÷5 mm). Outward movements of the side walls have also been recorded. Side walls generally suffered slight extensions. The different pattern of deformation appears indeed to be related to the effectiveness of the structural continuity between the old invert and side walls (e.g. Fig. 5). During the excavation stage the average measured convergence was about 14 mm.

Long term monitoring ensured about the stabilization of induced movements (e.g. Fig. 9) and proved the end of thawing process. In situ investigations (cone penetration tests) were carried out before and after the freezing process: the results demonstrated that mechanical properties were not affected by artificial freezing.

7 CONCLUSIONS

The design of the ground freezing application to the Cassia-Monte Mario tunnel was a challenging task, as it required the prediction of the behaviour of a soil which has suffered modifications from its natural state in the long history of tunnel instabilities and ground improvements. The structural behaviour of the existing lining was also not easily to predict because of the different methods of tunnel execution which were adopted over time.

In this complicated context, real-scale trial fields were an important step of the design stage, giving a deeper insight into the temperature field in the frozen soil body, the rate of freezing and thawing and the associated volume changes. An effective planning of the execution was carried out on the basis of the field testing and the invert reconstruction was successfully completed.

Long term monitoring and in situ tests have allowed to verify the complete exhaustion of the freezing process without any adverse consequences on the existing lining and on the ground surface.

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